MYRTLE GROVE DELTA BUILDING DIVERSION MODELING EFFORT IN SUPPORT OF THE LCA MEDIUM DIVERSION AT MYRTLE GROVE WITH DEDICATED DREDGING PROJECT

DATA COLLECTION, PRELIMINARY DESIGN AND MODELING INITIATIVE

Louisiana Coastal Protection and Restoration Authority

December 12, 2011

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Appendix D: *Water and Sediment Surveys of the Mississippi River Channel Conducted at Myrtle Grove and Magnolia in Support of Numerical Modeling (October 2008 – May 2010), Mead Allison, PhD, April 2011.*

Appendix E: *Myrtle Grove Delta Building Diversion: Numerical Modeling of Hydrodynamic and Sediment Transport in Lower Mississippi near Myrtle Grove River Bend, Ehab Meselhe, PhD, PE, Ioannis Georgiou, PhD, PE, John A McCorquodale, PhD, PE, June 2011.*

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1 INTRODUCTION

1.1 Background

The *Delta Building Diversion at Myrtle Grove (BA-33)* project (see Figure 1-1) was initially proposed to be another large freshwater diversion project in the Barataria Basin in addition to the Davis Pond Diversion project. The Myrtle Grove Diversion project would involve installation of a diversion structure on the west bank of the Mississippi River between Ironton on the south and the Alliance Refinery on the north; a conveyance channel with parallel mainline flood control levees and an outflow channel with guide levees.

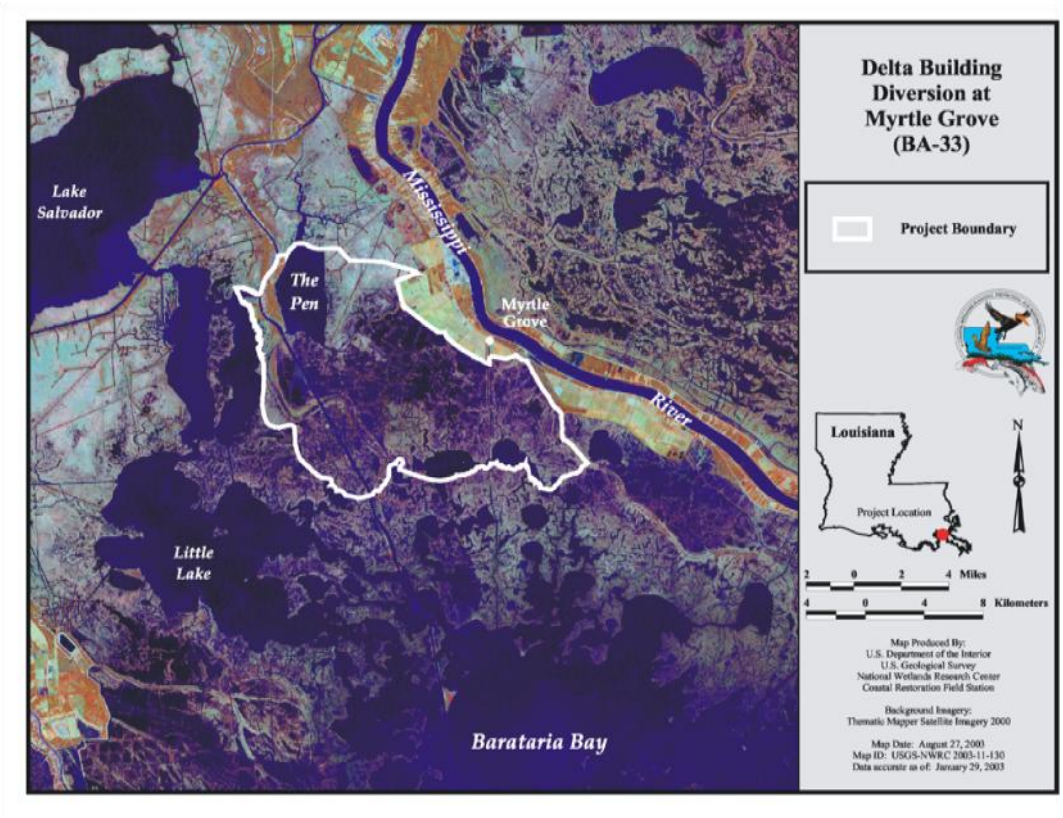

Figure 1-1: Project Location

In November 2008, the project was transferred to LCA authority and recast as a *Medium Diversion at Myrtle Grove with Dedicated Dredging*. This project was authorized by WRDA 2007 – Sec 7006 as a medium diversion (2,500 to 15,000 cfs) with a projected cost of $278.3 million (Oct 2004). However, Sec 7006 (c) (2) allowed for modifications:

> (A) IN GENERAL – In carrying out each project under paragraph (1), the secretary shall carry out such modifications as may be necessary to the ecosystem restoration features identified in the restoration plan –
> (i) to address the impacts of Hurricanes Katrina and Rita on the areas of the project.

According to the LCA project description:

> *The diversion would provide additional sediment and nutrients to nourish highly degraded existing fresh to brackish wetlands in shallow open water areas. This reintroduction would ensure the long-term sustainability of these marshes by increasing plant productivity, thereby preventing future loss. The introduction of sediment to this area would also promote the infilling of shallow open water areas both through deposition and marsh expansion. Dedicated dredging of sediment mined from the Mississippi River would complement this feature. This feature is located in the vicinity of a historic crevasse. The proposed feature would provide up to 13,400 acres of new emergent marsh and prevent the loss of another 6,300 acres of marsh. As one of the five near-term critical restoration features identified in the LCA Study, the Myrtle Grove project addresses the most critical ecological needs of the central Barataria Basin, which is an area where delaying action would result in a "loss of opportunity" to achieve restoration and/or much greater restoration costs. The benefits provided by this project includes: sustainable reintroduction of riverine resources; rebuilding of wetlands in areas at high risk for future loss, the preservation and maintenance of critical coastal geomorphic structures; preservation of critical areas within the coastal ecosystem while preserving the opportunity to begin to identify and evaluate potential long-term solutions.*

In order to examine modifications and how they might improve, or be detrimental to, the benefits and impacts of the project, the Louisiana Coastal Protection and Restoration Authority (CPRA) and several non-profit conservation organizations entered into a unique collaboration to undertake an extensive data-gathering and modeling initiative.

1.2 Objectives

The purpose of this initiative was to bring the best science and modeling into the planning process, to modify the diversion to maximize the capture of sediment and build land, to assess the potential impacts, both positive and negative, of the modified diversion and to answer stakeholder questions. Specific objectives included:

- Evaluate Design Flows
- Evaluate River Side Concerns
- Determine Optimal Location to Maximize Sediment Capture
- Complete a Conceptual Design of the Diversion Structure
- Evaluate Basin Side Impacts
- Complete a Comprehensive Report

1.3 Project Team & Structure of Report

1.3.1 Project Team

A project team approach was taken to fully take advantage of the experience and expertise of several public, Non-Governmental Organizations and private organizations. They included:

- Coastal Protection and Restoration Authority
- Non-Governmental Organizations
    - Environmental Defense Fund
    - National Audubon Society
    - National Wildlife Federation
- Louisiana State University
- University of Louisiana at Lafayette
- University of Texas
- Moffatt & Nichol
- Brown, Cunningham and Gannuch
- C.H. Fenstermaker
- Coastal Restoration Consultants
- Mobile Boundary Hydraulics

1.3.2 Structure of Report

The project was initiated in the summer of 2009 and a Memorandum of Understanding (MOU) was signed between the Environmental Defense Fund (working in conjunction with the National Wildlife Federation and the National Audubon Society) and the Coastal Protection and Restoration Authority of Louisiana. This MOU provided the framework for a data collection and modeling effort that could be utilized to initiate the process of satisfying the Water Resources Development Act of 2007 requirements in order to hasten construction of the Myrtle Grove Diversion.

Initially, Coastal Restoration Consultants and the LSU Department of Civil & Environmental Engineering were contracted to conduct physical model experiments at their small-scale physical model facility. This work was performed in late 2009 with the specific objectives of investigating the effectiveness of a sediment diversion at the authorized Myrtle Grove location and to identify other potential locations for sediment diversions within a reasonable distance from the authorized site.

In conjunction with, and after, this physical modeling effort, BCG Engineering and Consulting, Inc. developed conceptual designs of the diversion structure. These concepts incorporated the alternative diversion locations and channel alignments identified by the project team.

Moffatt & Nichol had previously performed numerical hydrodynamic and salinity modeling of the basin for CPRA as part of the original CWPPRA project. This prior work investigated numerous diversion scenarios in combination with Davis Pond diversions. As part of this new initiative, M&N performed additional model runs for a wide range of diversion magnitudes to determine their effects on water levels and salinities within the basin. Additionally, pulsed operation of different diversion magnitudes was modeled. Based on these results, the project team selected a range of "feasible" diversion flows for further evaluation.

Throughout the efforts described above, an intensive data collection effort was lead by Mead A. Allison, PhD. These efforts included bathymetric surveys to provide a basemap for numerical modeling of the river and to assist in the determination of bedload transport rates. River discharges were measured and suspended and bottom sediment samples collected. Profile data through the water column was collected using CTD (conductivity / temperature / depth) and LISST gages. This data included depth, backscatter, pH, oxygen, beam transmission, conductivity, salinity, temperature and suspended grain sizes.

Upon completion of the above tasks, 3-dimensional numerical modeling of the river was performed by Ehab Meselhe, PhD, PE; Ioannis Georgiou, PhD, PE; and John McCorquodale, PhD, PE. This modeling utilized the collected data for model setup and calibration / verification.

It also incorporated the conceptual designs of the diversion structure and channel. The results of this numerical modeling included the amount / percentage of various sediment sizes that would be diverted from the river, through the structure and channel, and into the Basin under the different diversion scenarios.

M&N then used these results as input to a morphological model of the basin to obtain a general understanding of the potential land-building capacity within the basin. Two alternative alignments and diversion scenarios were modeled for a period of 45 years. Based on the sediment transport patterns and morphological changes predicted by the model, deposition volumes of various depths were calculated. Additionally, consolidation of the underlying material was also accounted for.

This report provides a summary of all these efforts with more detailed reports for each provided in the Appendices.

2 SMALL SCALE PHYSICAL MODELING

2.1 Introduction

Coastal Restoration Consultants (CRC) and the LSU Department of Civil & Environmental Engineering (CEE) were contracted by the Coastal Protection and Restoration Authority to conduct physical model experiments at the small-scale physical model (SSPM) in support of the Myrtle Grove Delta Building Diversion Modeling Effort. The specific objectives of the SSPM experiments were: (1) to investigate the effectiveness of a sediment diversion at the authorized Myrtle Grove location; and (2) identify other potential locations for sediment diversions within a reasonable distance from the authorized site. In addition, the study looked at the relative effectiveness of two different diversion designs. A complete report of these efforts is included in Appendix A.

2.2 Small-Scale Physical Model Capabilities

The present small-scale model was designed for the purpose of making preliminary selection of desirable locations along the lower Mississippi river. While the SSPM is a distorted model, with the horizontal scale 1/12,000 and the vertical scale 1/500 (a distortion of 24), experimental results can provide important insights, especially when the SSPM is employed to compare identical diversion structure dimensions, using the same model and same operating procedures. Any difference in sediment diversion efficiencies is directly related to the structures' physical location and adjacent river current patterns. More quantitative analyses and studies related to structure design should be accomplished with larger scale modelling of sediment diversions from the Mississippi River to the wetlands.

In spite of the high distortion, the SSPM produces a relatively good reproduction of the prototype sand transport. A second point that needs to be emphasized is that this distortion is present everywhere on the SSPM and therefore the model can be used to compare different sites and scenarios, but does not provide the quantitative data necessary for design.

2.3 Overview of Two Schematic Designs of Diversion Structures

The SSPM was used to test two schematic design plans of the water and sediment diversion structures that have been suggested. The two schematic design concepts are:

- Gated broad crested sill by Coastal Restoration Construction (CRC).
- Narrow deep canal by Brown, Cunningham & Gannuch Engineering, Inc. (BCG).

The diversion structure (Figure 2-1) discussed by CRC can be described as a gated broad-crested sill that spans a total distance of approximately one mile along the river. Here, this structure is referred to as the "shallow" diversion. The idea behind placing the diversion structure along the riverbank close to the river water surface level is to take advantage of the secondary currents in large meandering rivers like the Mississippi. Locating the structures in areas where the secondary currents are oriented towards and impinging on the river bank with an appreciable angle, i.e., not parallel to it or moving away from the bank, will add momentum to the mass of water and sediment moving towards the bank and will transport a large portion of the sand load inland during the optimum river discharge of high sediment concentration.

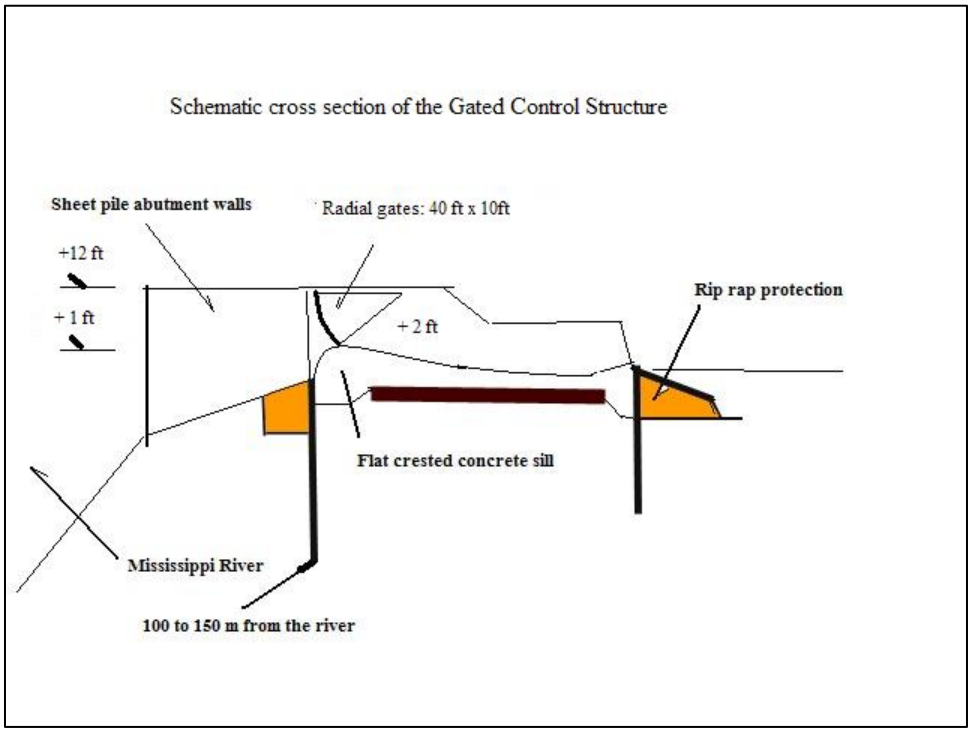

Figure 2-1: Schematic Representation of a Broad Crested Sill Diversion Structure

This shallow diversion structure employs a broad-crested sill with an average crest elevation of approximately + 2.0 ft NAVD88 and a total length of about 1 mile (without considering the pier widths separating the 40 ft x 10 ft gates). The radial gates would be 40 ft wide and 10 ft high, remotely controlled and operated by hydraulic cylinders. The discharge through the diversion can be estimated for different river stages by using assuming that the flow through the open structure is a broad crested weir at an elevation of +2.0 NAVD88 (Figure 2-2).

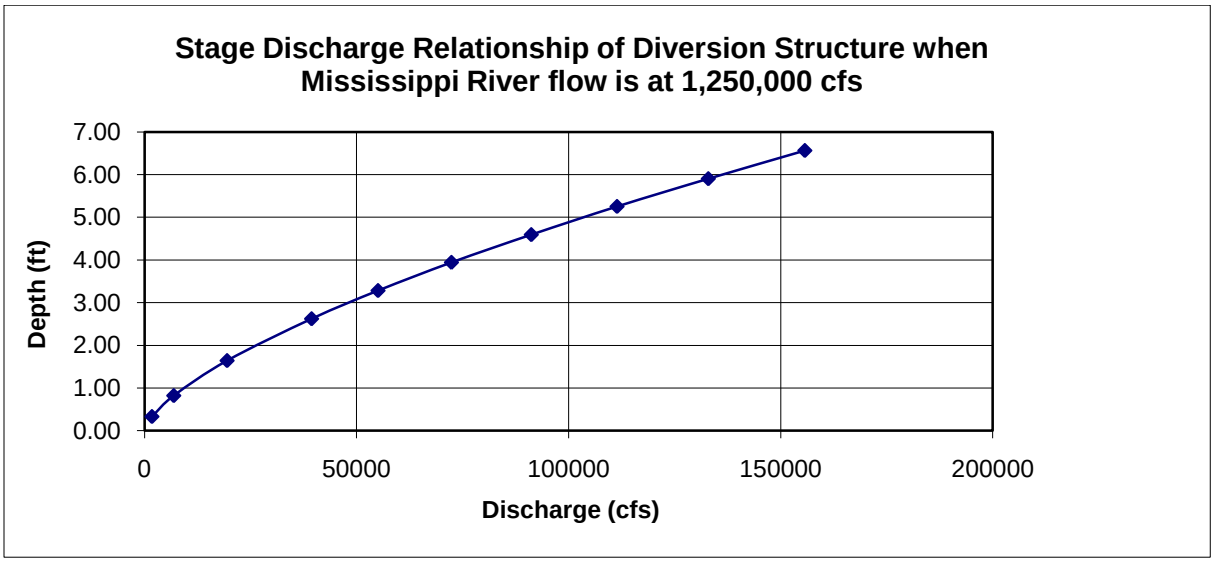

Figure 2-2: Computed Stage Discharge Rating Curve of a Diversion Structure

The deep channel diversion structure proposed by BCG has an inlet design with a channel bottom elevation near the riverbed level to capture the sand load transported by the Mississippi River (see Chapter 3).

2.4 Site Selection

Three separate sites were selected for testing (Figure 2-3). Because the SSPM only extends to ~RM61, the first site, at Myrtle Grove (RM 59), was chosen because that is the nearest location on the SSPM to the site currently being considered by the USCOE as a diversion site. The second site is located near Magnolia (RM 47.5). The third site, located at Woodland (RM 51), was chosen as an intermediate site.

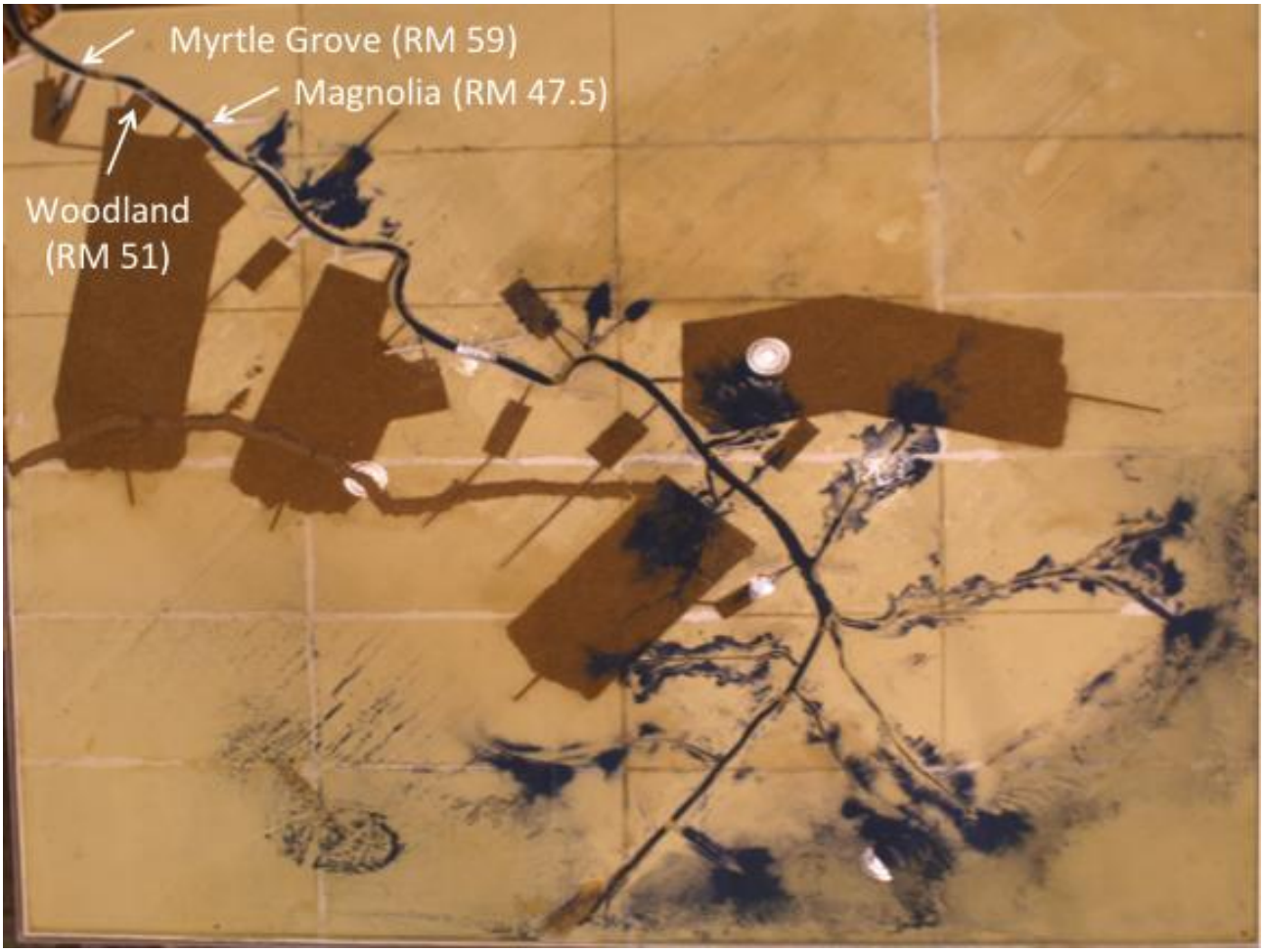

Figure 2-3: Diversion Locations for the SSPM Tests

The Myrtle Grove (RM 59) diversion site is located just downstream from a concave river bend. The channel bottom in this section is very deep and located close to the right descending bank. (It must be noted, however, the actual proposed Myrtle Grove site is located upriver of the bend but this location is too close to the boundary of the SSPM). The Magnolia (RM 47.5) site is located downstream of a convex river bend near a lateral bar while the Woodland location (RM 51) has similar characteristics to Magnolia.

2.5 Testing and Assessment of the Total Yearly Sediment Diversion

All tests were run using the SSPM standard operating procedures. Data collected during each test included river stages; mass of sediment injected, dredged, exiting model; and high-resolution overhead photos and video. This approach allows for more direct comparison between the different diversion locations and structure types.

2.5.1 Myrtle Grove and Magnolia (Shallow, 100 year)

A 100-year simulation was made with two identical shallow diversion structures, one at Myrtle Grove (RM 59) and one at Magnolia (RM 47.5), operating simultaneously. While this operational mode (i.e., both diversion operating simultaneously) is probably not likely to be followed, this configuration was used to optimize the experimental time. It should also be noted that the Myrtle Grove diversion would most likely have a negative impact on the Magnolia diversion, but not the other way. From the beginning, it was obvious that the Magnolia (RM 47.5) diversion structure captured a significantly larger amount of sediment than the Myrtle Grove (RM 59) diversion. High-resolution video was taken which clearly shows significant differences in the flow directions and sediment transport patterns at the two diversion locations. Figure 2-4 shows the surface area covered by the diverted sand deposits at Myrtle Grove (RM 59) and Magnolia (RM 47.5) after 100 years.

It should be noted that because of the minimum entrainment velocities over the deposited sediments outside the main river channel, the areas covered by the sand deposits are relatively smaller than what would be in the prototype. Significantly higher flow turbulence in nature would transport the sands much further and would cover much larger areas.

Figure 2-4: Comparative Sediment Diversion Efficiencies

Sieve analysis of the sediments (Figure 2-5) show that a higher percentage of finer sediment particles are being diverted at Myrtle Grove (RM 59), i.e., more from the surface layer of the river flow. On the other hand, the gradation curve for the Magnolia (RM 47.5) location is nearly identical to the inflow sediment gradation indicating that coarser sediment particles are being diverted from the bottom layers of the river flow, as bed loads are entrained by the secondary currents in this reach of the river.

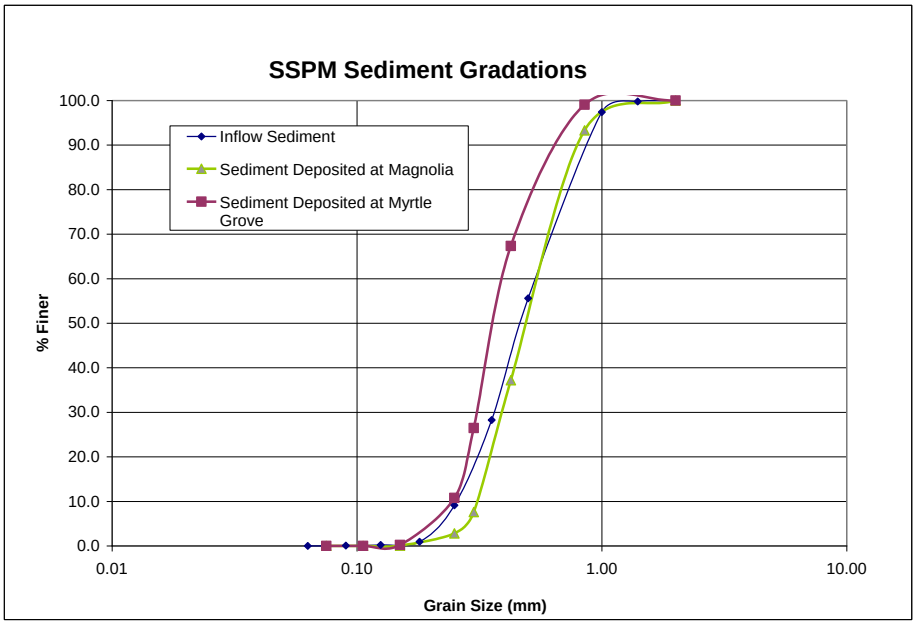

Figure 2-5: Sediment Gradation Curves

2.5.2 Woodland (Shallow, 50 & 100 Year)

Two tests were run using the shallow diversion structure at Woodland (RM 51). The first was conducted immediately following the Myrtle Grove and Magnolia shallow 100 year test and was a 100 year test. Due to time constraints, this experiment was run prior to removal of the model sediment from the Myrtle Grove and Magnolia diversion outfall areas. Thus, merging of the sediment deposits from the Magnolia and Woodland deposition areas make photos of just Woodland inconclusive and prevented independent measurements of the sediment deposited from each of the diversions. The second test was run for 50 years after a reset of the system. All of the standard data was collected in the 2nd test.

2.5.3 Magnolia and Myrtle Grove (Deep, 50 Year)

50 year tests were run on the Magnolia (RM 47.5) and Myrtle Grove (RM 59) diversion locations using the deep diversion structures based on the BCG design. These tests were run using the same procedures as in the previous tests. Qualitatively, it was obvious that the Magnolia (RM 47.5) and Woodland (RM 51) locations diverted more sediment than the Myrtle Grove (RM 47.5) location and that the shallow diversion structures appear to divert larger quantities of sediment than the deep structures.

2.5.4 Sediment Transfer Efficiency Calculations

Annual sediment (sand) diversion efficiencies were calculated based on weights of sediments collected at Myrtle Grove (RM 59) for deep (50 year) and shallow (100 year) tests, the combined Magnolia (RM 47.5) and Woodland (RM 51) location for shallow tests, the Magnolia (RM 47.5) deep test and the 50 year Woodland shallow test. The results are shown in Table 2-1.

Table 2-1: Sediment Transfer Efficiency

Location	Total Weight of Sediment Diverted Annually (Prototype)	Sediment Transfer Efficiency
Myrtle Grove (RM 59) Shallow	309,286 tons	12.8%
Magnolia (RM 47.5) + Woodland (RM 51) Shallow	1,754,072 tons (each site assuming 50% split)	73%
Woodland (RM 51) Shallow	1,323,220	55.1%
Magnolia (RM 47.5) Deep	653,615	27.2%
Myrtle Grove (RM 59) Deep	58,685	2.4

The comparative efficiency of the sediment diversions at Magnolia (RM 47.5) and Woodland (RM 51) locations in relation to Myrtle Grove (RM 59) is 5.7 times greater. The comparative efficiency of the sediment diversion at the 50 year Woodland (RM 51) location in relation to Myrtle Grove (RM 59) is 4.3 times greater. In addition, comparison of the Magnolia (RM 47.5) deep test to the Myrtle Grove (RM 59) deep test showed that the Magnolia location was 11.3 times more efficient. These values confirm the original hypothesis that the Myrtle Grove location (at least the location tested in the SSPM) would not be conducive for efficient delivery of sediment into the diversion structure.

An additional comparison can be made between the deep and shallow diversions. The efficiency of the Magnolia (RM 47.5) shallow relative to the Magnolia (RM 47.5) deep is 2.7 times greater and the Myrtle Grove (RM 59) shallow relative to the Myrtle Grove (RM 59) deep is 5.3 times greater. Given the limitations of the SSPM to replicate complex 3D flow patterns, the results showing that the shallow diversions are more efficient than the deep ones should be viewed as semi-quantitative. Additional larger scale tests would be required to confirm and better understand the results.

2.6 Conclusions

For identical structural arrangements, the SSPM demonstrated that it is possible to divert significant amounts of more sediment by adequate site selection of the diversion structure. Pre-study hypotheses concerning the impact of river channel geometry combined with analysis of the bathymetric survey charts and knowledge of riverine hydrodynamics and sediment transport processes were confirmed. The two sites located downstream of convex river bends near lateral bars, Magnolia (RM 47.5) and Woodland (RM 51), both performed much better than the Myrtle Grove (RM 59) site located downstream of a deep, concave river bend. While not sufficient for site design work, the SSPM results showed that the shallow, wide diversion structures, proposed by CRC, outperformed the deep diversion structures. Note, however, that additional larger-scale physical modeling, analysis of field data and high-resolution 3D numerical modeling should be performed to guide detailed design.

3 CONCEPTUAL STRUCTURE DESIGN

3.1 Background

BCG Engineering and Consulting, Inc. developed conceptual designs of the diversion structure at Myrtle Grove. Complete reports documenting this work are included in Appendix B and Appendix C. A preliminary design flow hydrograph for the Mississippi River at Myrtle Grove was developed and is shown in Figure 3-1. Additionally, the river level within the study reach must be known for any given design discharge in order to size the opening of a diversion structure to be able to convey water and sediments out of the river and into the waters of Barataria Bay. Thus a rating curve, shown in Figure 3-2, was developed based on an average of points plotting historical river discharges and stages.

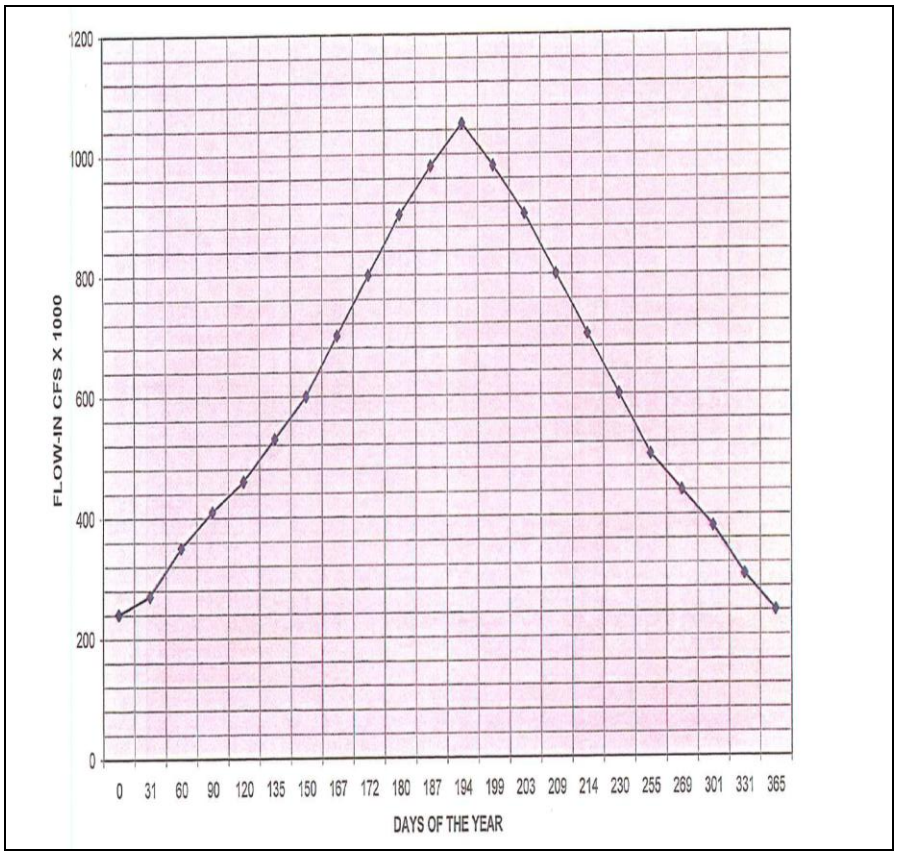

Figure 3-1: Conceptual Design Flow Hydrograph for Mississippi River

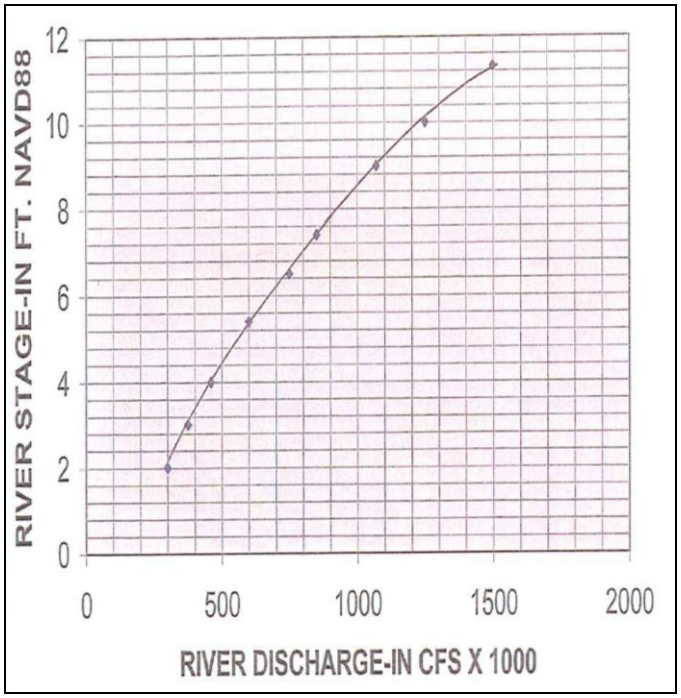

Figure 3-2: Conceptual Design Rating Curve

3.2 Alternative Alignments

Two alternative diversion locations and channel alignments were considered. The first, at RM 60.2 AHP and shown in Figure 3-3, is based on the original USACE concept. The second "modified" alignment, at RM 60.7 and shown in Figure 3-4, was selected based on the results of the physical model tests and basic hydraulic analyses of the River which indicated that it would more efficiently capture and transport sediment from the River to the Barataria Basin.

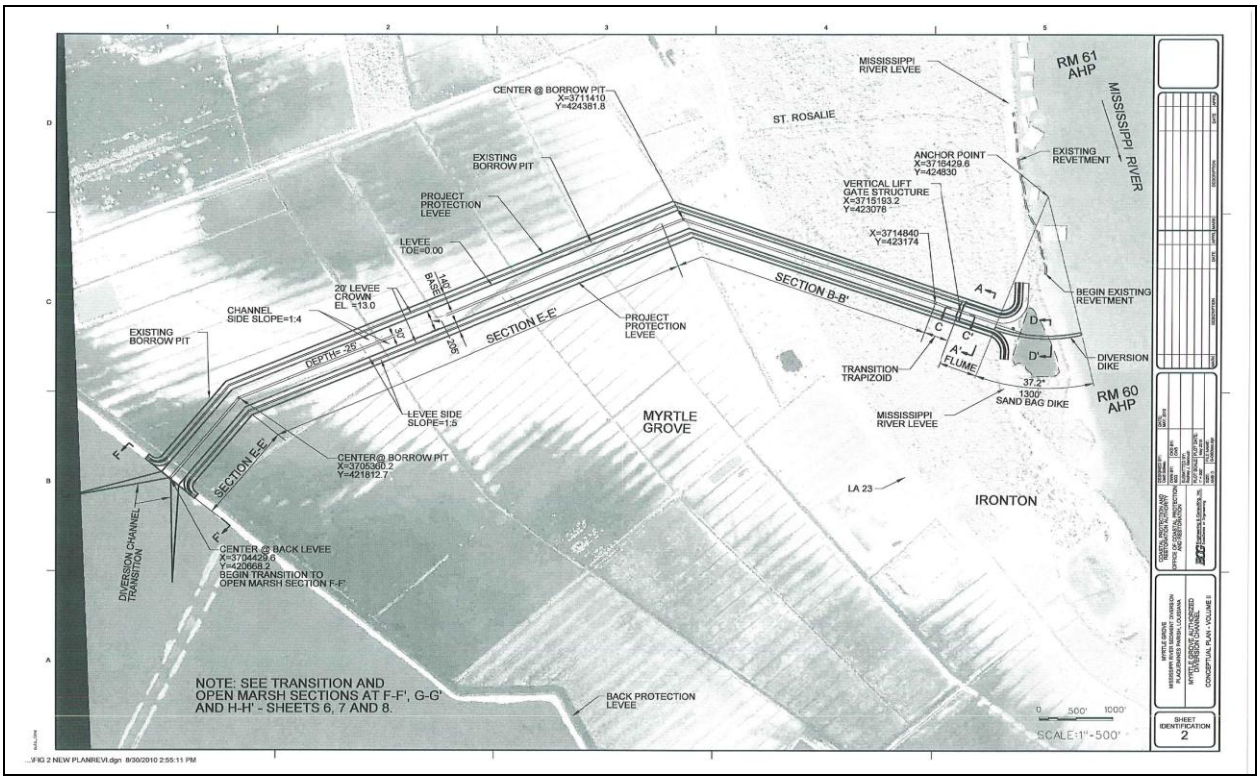

Figure 3-3: Original USACE Alignment

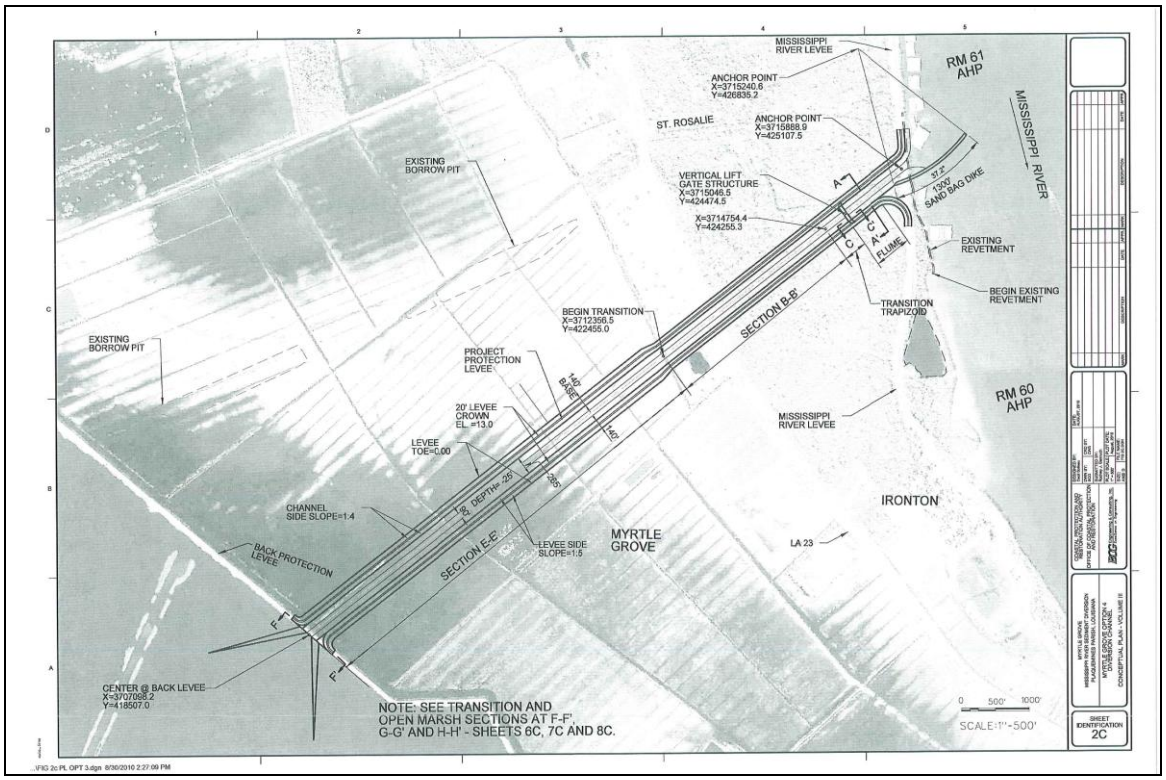

Figure 3-4: Modified Alignment

3.3 Channel Cross-Sections & Structure Design

Given the express purpose of conveying high concentrations of sediments in full suspension above the bed of the diversion channel into the receiving water body, the discharge channel must have as small a wetted perimeter as practical to provide a large hydraulic radius. Additionally, it must be hydraulically smooth to conserve available energy; yet must have a high flow velocity and must cause turbulent flow for good transport capability.

However, hydraulic efficiency and turbulence are generally at cross purposes and therefore counter balance each other, and the fact that this design is for a sediment diversion places additional requirements on the design. This is because the depth of the point bar, -40 feet at RM 60.2 AHP, will govern the shape of the diversion channel and the wetted perimeter. Based on the average annual peak flow of approximately 1.05 million cfs, a river stage of nine feet is expected (neglecting the tidal effects), so a total depth of 44 feet (-35' + 9') over the gate sill would dictate the most efficient cross-section. It is common knowledge that greater hydraulic efficiency can be achieved if the width of a channel is twice the depth, in which case the width would be 2 x 44 feet or 88 feet. However, these dimensions, 44 feet x 88 feet, will provide more than twice the diversion flow being used (15,000 cfs) as the target flow for modular design. However this size channel could be used as a modular target to increase the design flow to 70,000 cfs. Therefore, the basic design for the diversion channel will be a 44 feet x 30 feet concrete flume that will produce adequate turbulence at an average velocity of eleven feet per second.

Basic features of the design of the diversion structure are shown in Figure 3-5 and Figure 3-6. The gates will be five individual 30 feet wide by 10 feet high segments that can be dogged at any desired gate opening to facilitate diversion choices to enhance sediment diversion or freshwater mitigation. River works in the shape of a 1,200 foot deflection dike and free standing sheet pile wall are included to enhance water and sediment entry into the diversion channel. The structure will have a single gate lifting crane to operate the gates. The concrete flume section is then 770 feet in length.

The ultimate size project structure is essentially an enlarged version of the basic design, in that many features/shapes/etc. are similar.

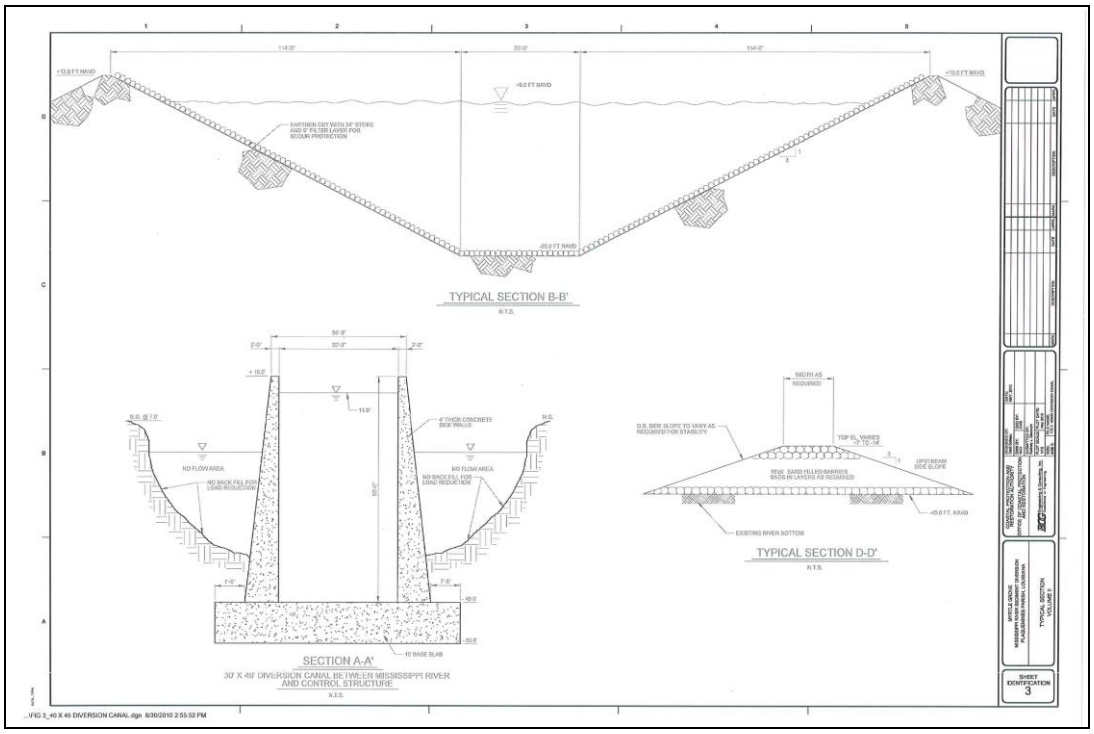

Figure 3-5: Typical Sections

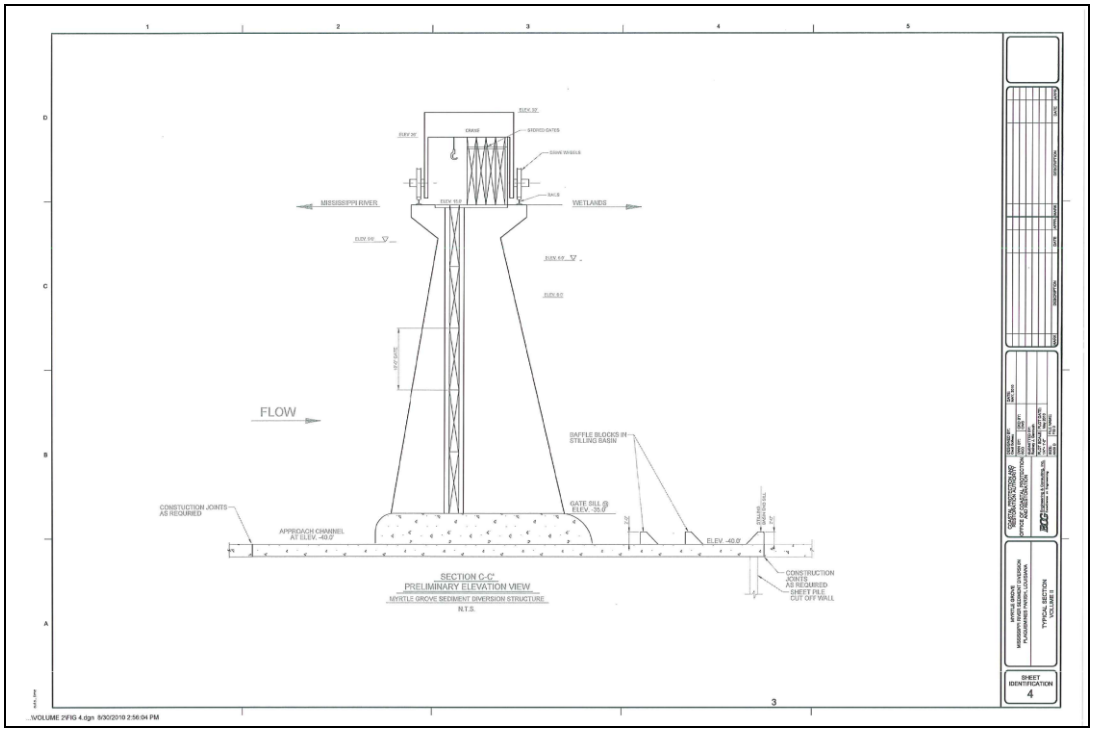

Figure 3-6: Gate Section

3.4 Additional Considerations

This project location will sever a portion of the drainage system currently in existence on Conoco Phillips Company lands and measures will need to be taken to render the drainage system whole again. Furthermore, there are public utilities, water forced mains, a natural gas line and several commercial power lines that will require major relocations and modifications to span across the project channel. In addition, an elevated four-lane bridge and access ramps will be required to accommodate area traffic on State Highway 23.

Federal levees and local back levees exist at the Mississippi River bank and at the National Food and Beverage Company western boundary. This will require that adjacent levees, equal in design criteria to Federal Mississippi River and Tributaries Project Levees, must be provided on each bank of the diversion channel, since these levees will contain Mississippi River waters.

3.5 Budget Level Cost Estimate

The probable construction cost for the 15,000 cfs Myrtle Grove Freshwater and Sediment Diversion Project as located by the USACE at RM 60.2 AHP is approximately $180 million. It should be noted that this estimate does not include any land acquisition costs or engineering and design fees, or any costs for supervision and administration during construction. If the project is enlarged to allow for a 45,000 cfs diversion, then the cost estimate would be approximately $330 million.

4 BASIN SIDE HYDRODYNAMIC & SALINITY MODELING

4.1 Phase I – CWPPRA Modeling

Initially, Moffatt and Nichol (M&N) was retained by Louisiana Department of Natural Resources - LNDR (now Coastal Protection and Restoration Authority - CPRA) to perform alternative modeling for the *Coastal Wetlands Planning, Protection, and Restoration Act (CWPPRA) Delta Building Diversion at Myrtle Grove*. This effort was undertaken to provide the ability to more completely evaluate the effects of the proposed project on a basin-wide scale. The results of this initial effort and others discussed in this chapter are included in the complete report included in Appendix F.

Moffatt & Nichol developed and completed an estuarine hydrodynamic model for evaluating the Barataria Basin effects due to operating both the Myrtle Grove and Davis Pond diversions at varying and increasing flow rates. The model is capable of:

- Simulating existing water movements and salinity distributions within the basin;
- Future use by CPRA to project future hydrodynamic conditions for both with and without implementation of restoration strategies;
- Easily upgraded and / or refining as new data become available and modeling technology and capabilities improve;
- And evaluating the effects of proposed Myrtle Grove project on a basin-wide scale.

4.1.1 Model Description

This previously developed numerical model (see Figure 4-1) was used for this study. Some modifications were made to the original model grid to increase resolution in the vicinity of the Myrtle Grove diversion and to eliminate one discharge boundary condition where limited data was available. Additionally, rainfall runoff from areas within the basin not included in the model grid was added.

A simulation time period of one calendar year was chosen to assess the alternative plans. This simulation period is from August 2002 to July 2003 which included the two primary model calibration periods when extensive data collection efforts were undertaken. It should be noted that two tropical storms (Isidore and Lili) directly impacted the basin during late September and the beginning of October 2002, while Tropical Storm Bill impacted the basin at the end of June and beginning of July 2003.

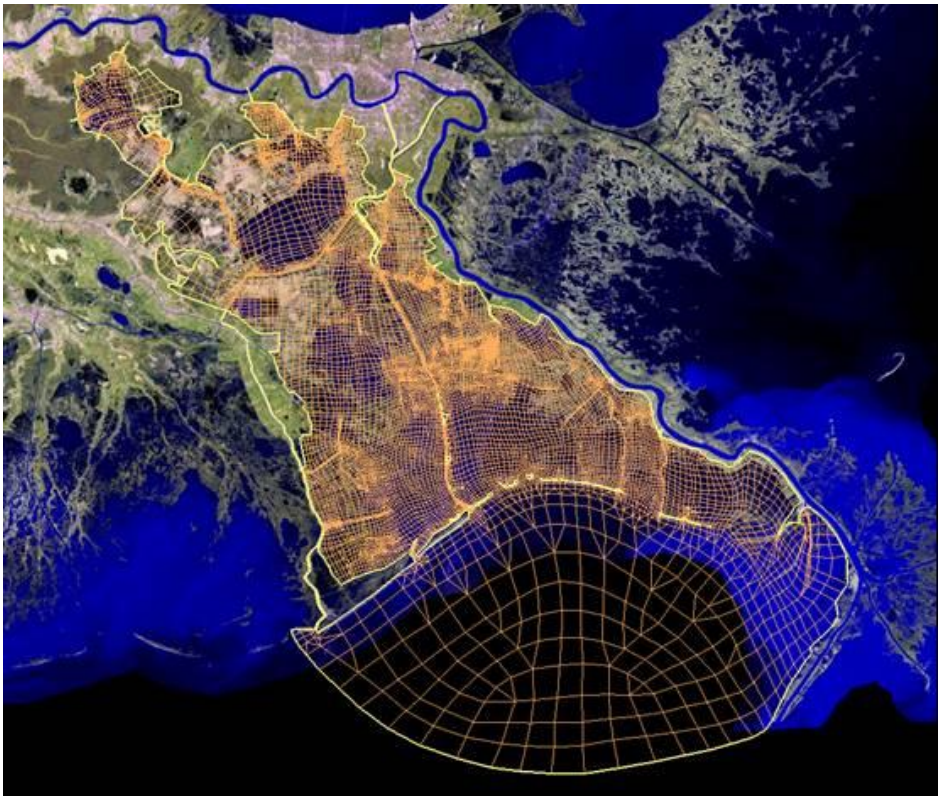

Figure 4-1: Barataria Basin Finite Element Grid

4.1.2 Model Results

For this alternative modeling study, four different Myrtle Grove diversion scenarios were modeled in conjunction with three appropriate diversion regimes at Davis Pond as shown in Table 4-1. These diversion flow rates were provided by the USACE. The baseline condition for salinity change comparison was taken as the "existing" condition runs with only nominal diversions at Myrtle Grove and Davis Pond.

Additionally, three extra runs were made to distinguish the effects of the Davis Pond Diversion from those due to the Myrtle Grove Diversion. For these three runs, nominal discharge boundary conditions at the Myrtle Grove Diversion were applied coupled with the three different Davis Pond Diversion discharge scenarios.

In order to investigate a larger Myrtle Grove diversion effect, another scenario run with double-high Myrtle Grove diversion (diversion flows are twice the flow rates of the high Myrtle Grove scenario) in combination with a high Davis Pond diversion was also performed.

Therefore, a total of seventeen one-year model runs were performed:

- "Existing" Condition (EXCO)
- High Davis Pond & High Myrtle Grove (DHMH)
- High Davis Pond & Medium Myrtle Grove (DHMM)
- High Davis Pond & Low Myrtle Grove (DHML)
- Medium Davis Pond & High Myrtle Grove (DMMH)
- Medium Davis Pond & Medium Myrtle Grove (DMMM)
- Medium Davis Pond & Low Myrtle Grove (DMML)
- Low Davis Pond & High Myrtle Grove (DLMH)
- Low Davis Pond & Medium Myrtle Grove (DLMM)
- Low Davis Pond & Low Myrtle Grove (DLML)
- High Davis Pond & Nominal Myrtle Grove (DHMN)
- Medium Davis Pond & Nominal Myrtle Grove (DMMN)
- Low Davis Pond & Nominal Myrtle Grove (DLMN)
- High Davis Pond & Alternative R1 Myrtle Grove (DHMR1)
- Medium Davis Pond & Alternative R1 Myrtle Grove (DMMR1)
- Low Davis Pond & Alternative R1 Myrtle Grove (DLMR1)
- High Davis Pond & Double-High Myrtle Grove (DHM2H)

After the one year runs were completed, the following data were extracted from the results:

- The average salinities on a monthly, semi-annual (December – May), and an annual basis;
- The average, minimum and maximum water levels on a monthly, semi-annual (December – May), and an annual basis;
- The average and maximum current velocity magnitudes on a monthly, semi-annual (December – May), and an annual basis;
- The 5 ppt and 15 ppt salinity contour lines; and
- Water level and velocity magnitude results from seventeen specific locations throughout the basin (Figure 4-2).

Table 4-1: Discharge Time Series at the Diversions (cfs)

Month	Davis Pond Diversion				Myrtle Grove Diversion				
	Existing	High	Medium	Low	Existing	High	Medium	Low	R1
Jan		8,000	6,000	4,000		16,500	5,300	10	19,881
Feb		10,560	7,920	5,280		18,300	6,400	10	33,063
Mar		10,560	7,920	5,280		19,500	7,700	10	39,546
Apr		10,560	7,920	5,280		19,420	7,500	10	39,546
May		10,560	7,920	5,280		19,200	7,000	10	39,546
Jun	Nominal	10,560	7,920	5,280	Nominal	18,950	4,800	10	10
Jul	10	6,000	4,500	3,000	10	14,840	3,300	2,500	10
Aug		4,000	3,000	2,000		9,740	3,300	1,500	10
Sep		4,000	3,000	2,000		9,550	3,000	10	10
Oct		4,000	3,000	2,000		9,400	3,000	1,000	10
Nov		6,000	4,500	3,000		9,330	3,000	1,000	10
Dec		8,000	6,000	4,000		12,900	4,000	10	19,881

Note: The medium and high flows for the Davis Pond diversion are computed as 150% and 200%, respectively, of the corresponding low flows; for Myrtle Grove diversion R1, the monthly discharges are the maximum values.

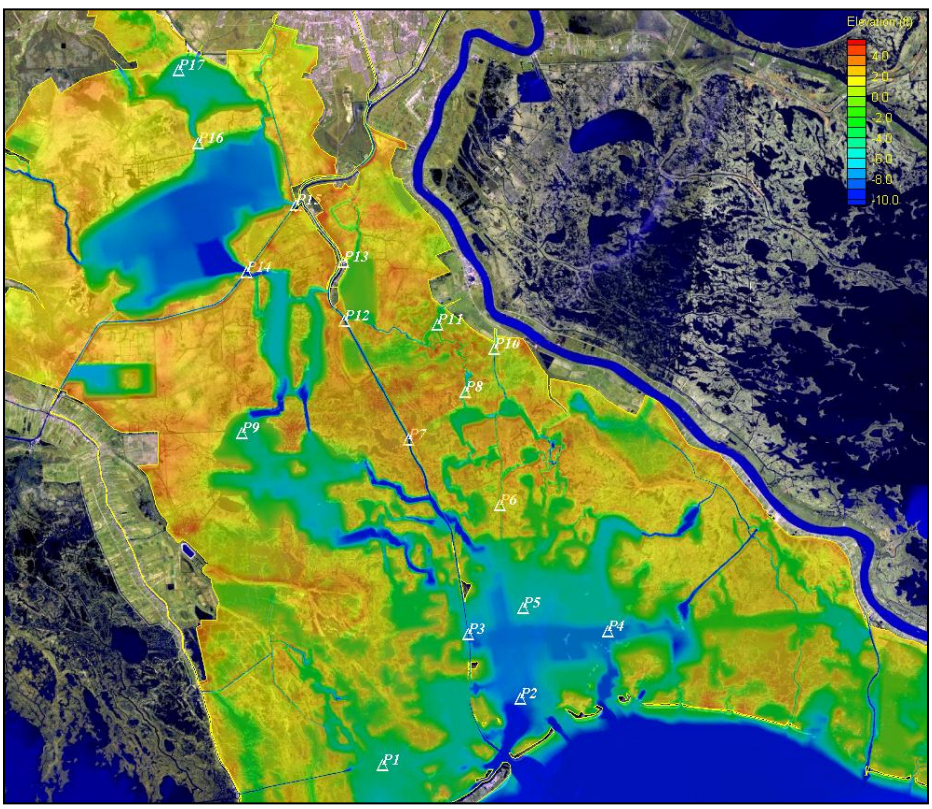

Figure 4-2: Location Map for Hydrodynamics Results

Figure 4-3 through Figure 4-5 present the impact of the proposed Myrtle Grove diversions on the 5 ppt and 15 ppt salinity contours in the basin on an annually averaged basis for the 2002-2003 time frame modeled. These figures indicate that by comparing the results to the nominal Myrtle Grove diversion scenarios:

- In the case of the high Myrtle Grove diversion, the 5 ppt and 15 ppt salinity contour lines would retreat to the south by about 4 miles and 1.5 miles, respectively; further when combined with the high Davis Pond diversion than if there was no diversion at Myrtle Grove and only a high Davis Pond diversion (DHMH-DHMN). The retreats would be about 4.5 miles and 2 miles if combined with a medium Davis Pond diversion (DMMH-DMMN); and the retreats would increase to 6 miles and 2.5 miles combined with a low Davis Pond diversion (DLMH-DLMN). In other words, the impact of a high Myrtle Grove diversion increases with a decrease in the diversion at Davis Pond.

- Under the medium Myrtle Grove diversion scenarios, the 5 ppt and 15 ppt lines would move about half the distance of corresponding high Myrtle Grove diversion cases.

- The low Myrtle Grove diversion shows little impact on the salinity levels in the basin.

- For the R1 diversion case at Myrtle Grove, the annually averaged 5 ppt contour lines retreat to the south less than the high Myrtle Grove cases, and the 15 ppt contours are close to the high Myrtle Grove cases. The reason is that there are no diversions from June to November.

- For the double-high Myrtle Grove diversion case, both the 5 ppt and 15 ppt contours retreat further to the south.

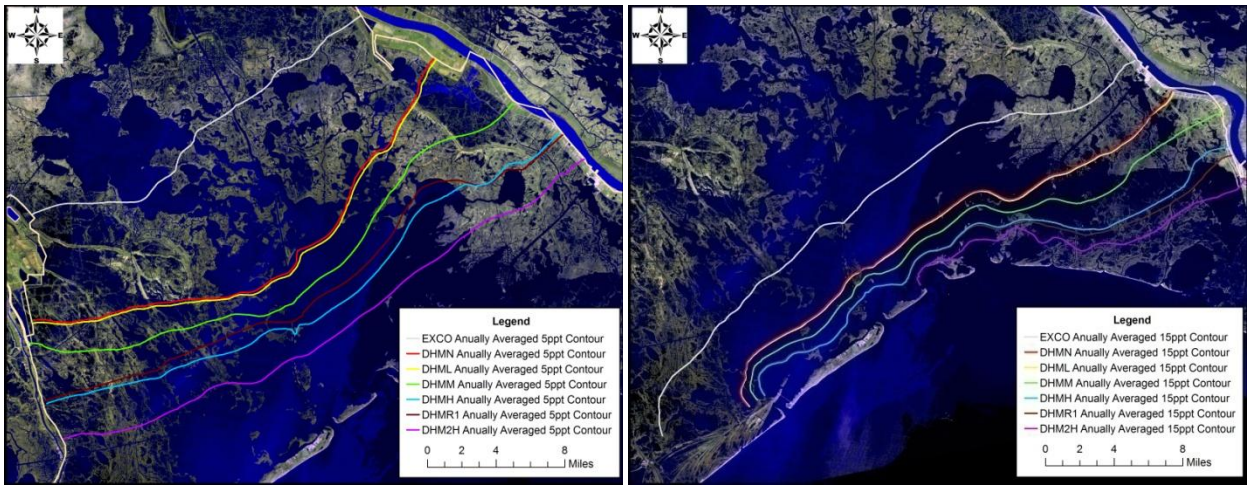

Figure 4-3: Annually Averaged Contour Changes under High Davis Pond Diversion Rate

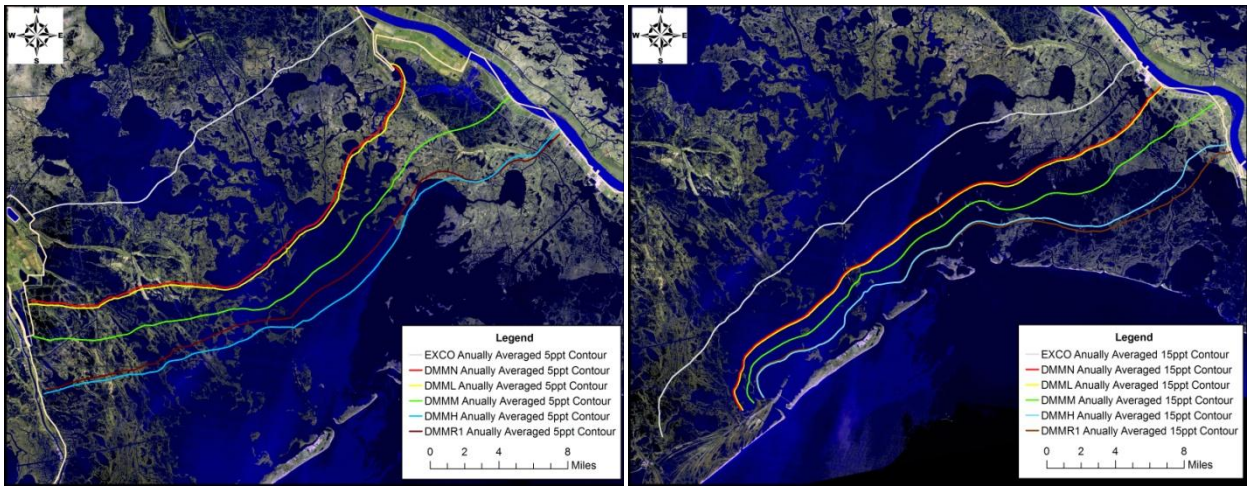

Figure 4-4: Annually Averaged Contour Changes under Medium Davis Pond Diversion

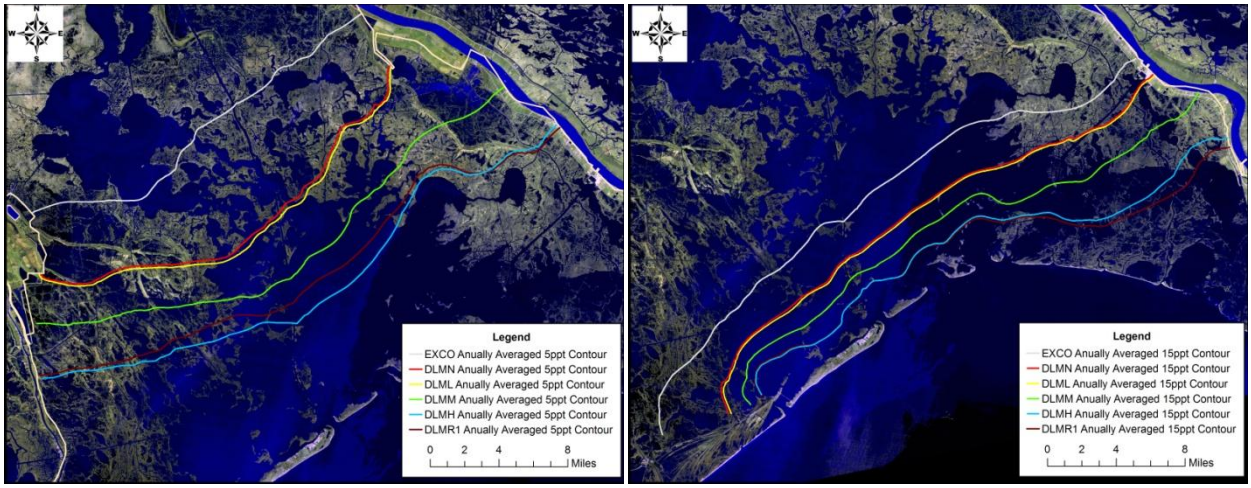

Figure 4-5: Annually Averaged Contour Changes under Low Davis Pond Diversion

The following conclusions were drawn after analyses and review of the alternative modeling results:

- The impacts on salinity levels in the Barataria Basin from the Myrtle Grove project depend on the diversion regimes at Davis Pond. The effects of the Myrtle Grove project are reduced under higher Davis Pond diversion scenarios.
- The Myrtle Grove project under low diversion has negligible impact on salinity levels in the Barataria Basin regardless of the Davis Pond Diversion operational level.
- High Myrtle Grove diversions could reduce annual average salinity levels over 6 ppt depending upon the magnitude of diversions at Davis Pond while medium Myrtle Grove diversions would only reduce the annual average salinity by less than 4 ppt.
- The high Myrtle Grove diversion scenario would push the annual 5 ppt and 15 ppt salinity level contours twice as far southward as the medium Myrtle Grove diversion case; regardless of the magnitude of the different Davis Pond diversion.
- On a semi-annual (December – May) basis, the R1 and double-high Myrtle Grove diversion scenarios push the 15 ppt salinity line to near the backside of the barrier islands except in the immediate vicinity of the passes, and the far eastern section of the basin.
- From a hydrodynamic point of view, on average, the larger diversions from Myrtle Grove and Davis Pond cause significant water surface elevation and current magnitude increases in the regions adjacent to the diversion sites.

4.2 Phase II – Hydrodynamic Pre-Screening Runs

The work described in Phase I was built upon to pre-screen Myrtle Grove diversion flows in order to determine feasible diversion flows for a sediment diversion. The RMA2 model was used to evaluate maximum diversion magnitudes of 15,000 cfs; 45,000 cfs; 75,000 cfs; 150,000 cfs; 240,000 cfs and 300,000 cfs (Figure 4-6) on water level and velocity increases in the basin. The purpose of these runs was to establish the upper limit of diversion magnitudes for further modeling by identifying the flows with significant, insurmountable impacts. Wind and rain were not evaluated in these runs.

The diversion was assumed to be operated from February to July. The Myrtle Grove diversion hydrographs were developed based on the hydrograph from the Mississippi River.

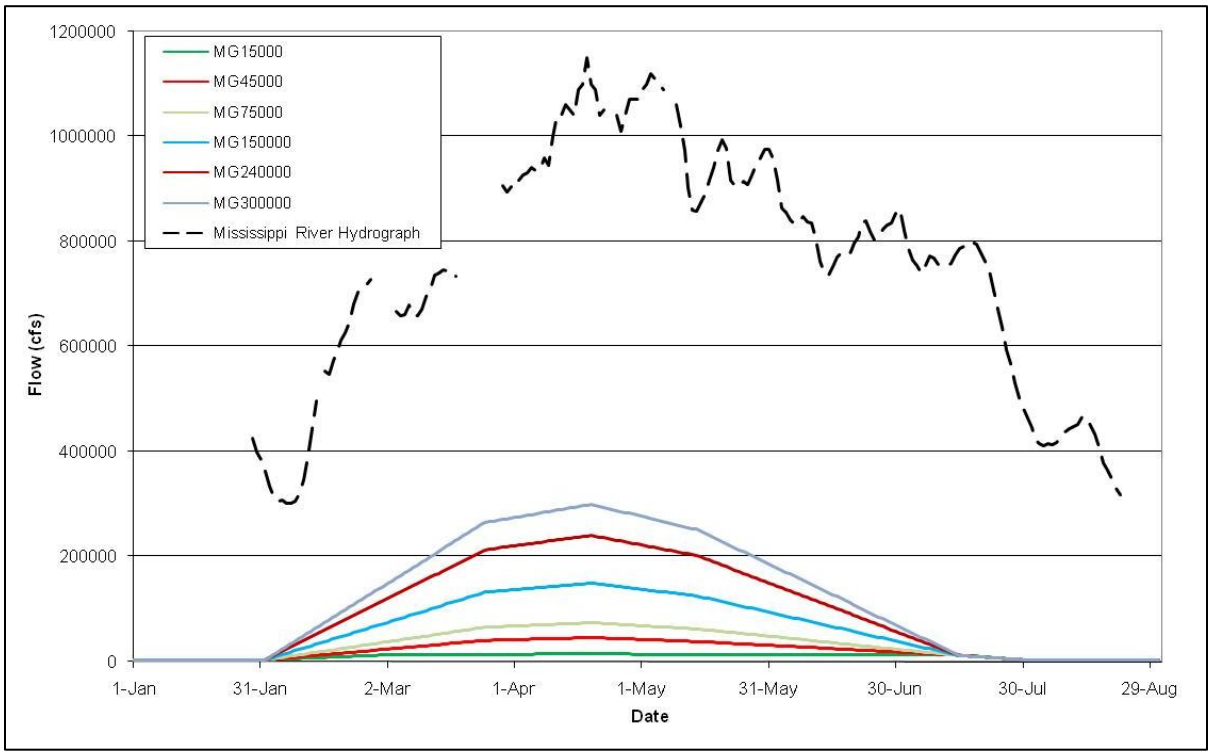

Figure 4-6: Myrtle Grove Diversion Hydrographs for Pre-screening Model Runs

At first, a modeling run with nominal freshwater discharges from the Davis Pond and Myrtle Grove diversions was conducted to act as the baseline condition. The offshore water level boundary condition and GIWW discharge boundary condition are the same as the ones in the Phase I study. Next, the six Myrtle Grove diversion scenarios were modeled in conjunction with a nominal David Pond diversion.

After the model runs finished, the monthly average, maximum and minimum water surface elevations, as well as the monthly average and maximum current velocity magnitude for each diversion scenario for the period of February to July were extracted and compared to the baseline condition. The results from the 17 points in Figure 4-2 were also extracted. Figure 4-7 and Figure 4-8 present the monthly average water surface elevation and velocity magnitude changes relative to the baseline condition caused by different Myrtle Grove diversion magnitudes in April when the diversion reaches its peak.

It is evident that the highest diversions could cause significant flooding problems and severe currents in the regions adjacent to the diversions. On average, near the Town of Lafitte in April when the diversion reaches its peak flow, the 45,000 cfs and 75,000 cfs diversion flows could increase water levels by 1 and 2 ft, respectively; but for the 150,000 cfs and 300,000 cfs diversions, the peak water levels could increase by 3 and 5 ft, respectively. The increased velocity with the larger diversions could cause local scour and damage the existing marshes as well as creating a potential safety hazard.

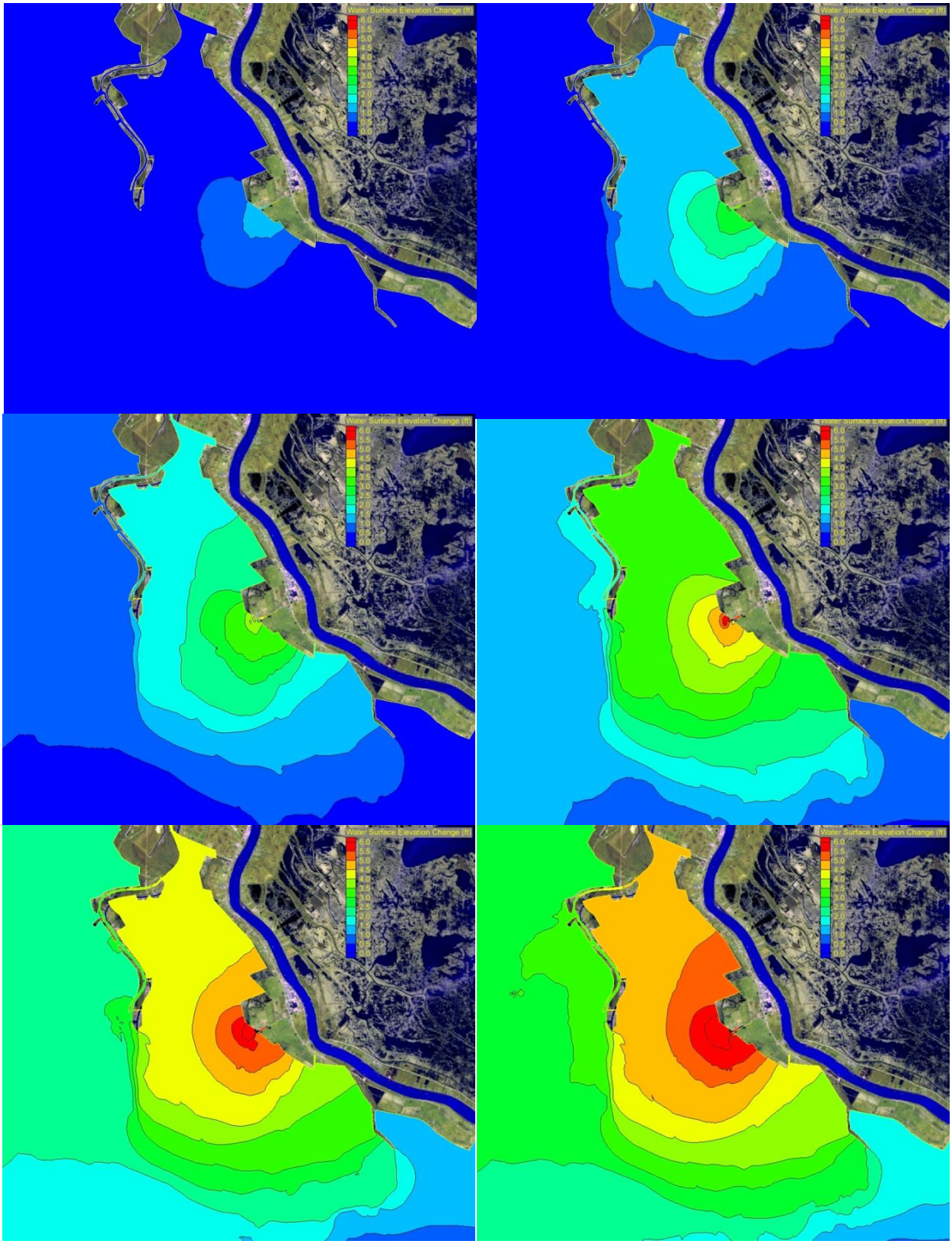

Figure 4-7: Average Water Surface Elevation Change

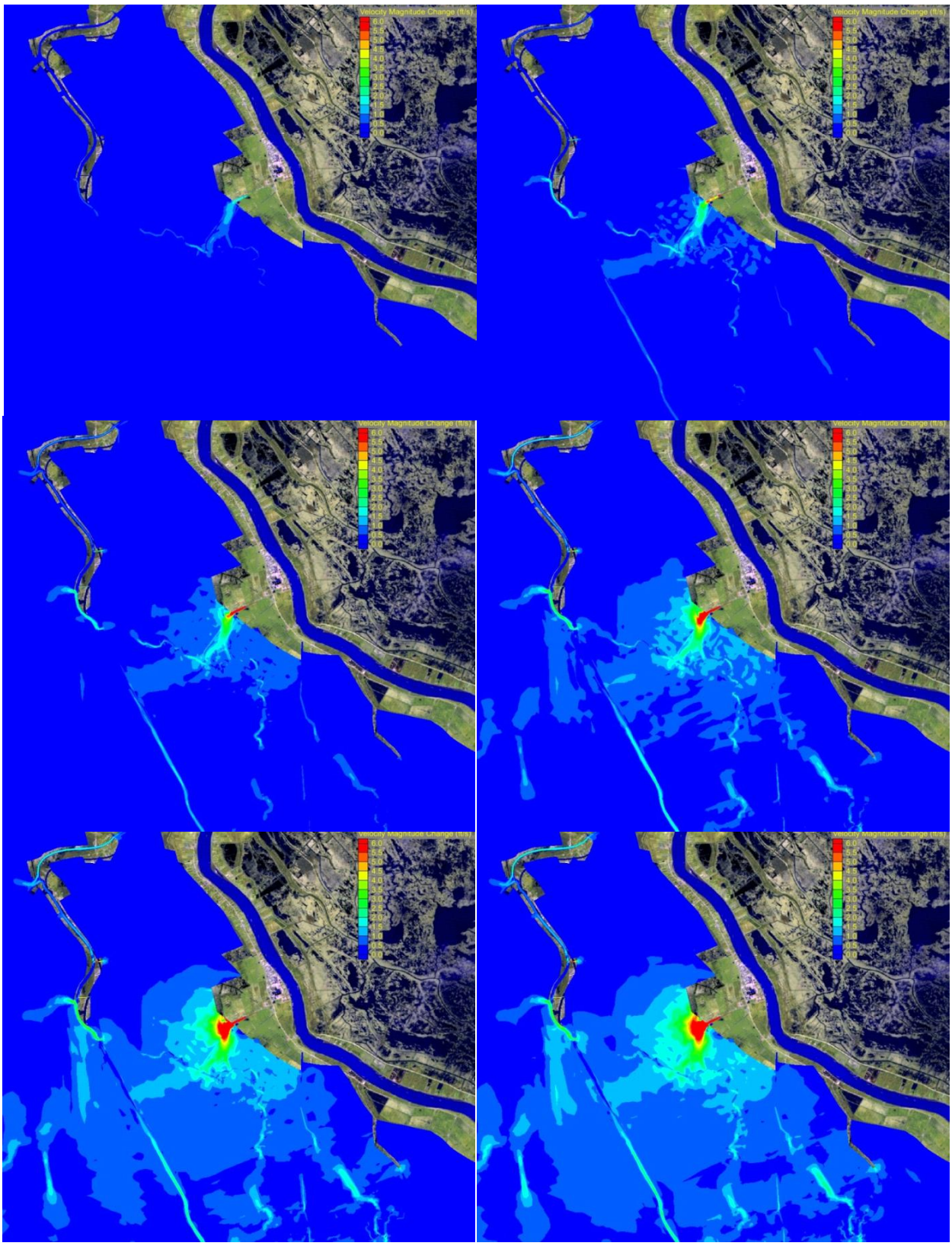

Figure 4-8: Average Velocity Magnitude Change

4.3 Phase III – Pulsing Diversion

Pulsed operation of controlled diversions offers a possible strategy for maintaining a wide range of salinity conditions while at the same time delivering river sediments into a wetland region. For example, if diversions are operated alternating between periods of full discharge and zero discharge, then one might expect surface water salinity in the basin to fluctuate between less saline and more saline conditions, depending on the residence time of water in the basin and the frequency with which the diversion discharge is pulsed. Additionally, the duration of elevated water levels and current velocities might be mitigated if the system responds fairly quickly.

Based on the results from Phase II, the hydrodynamic effects of three pulsing diversions were investigated in this phase of the study. The maximum diversion magnitudes are 15,000 cfs; 45,000 cfs and 75,000 cfs and the pulsing interval was set at two weeks with the diversion operating and two weeks with a minimum diversion flow of 5,000 cfs. Figure 4-9 shows the proposed diversion hydrographs. Wind and rain effects were not considered in these runs.

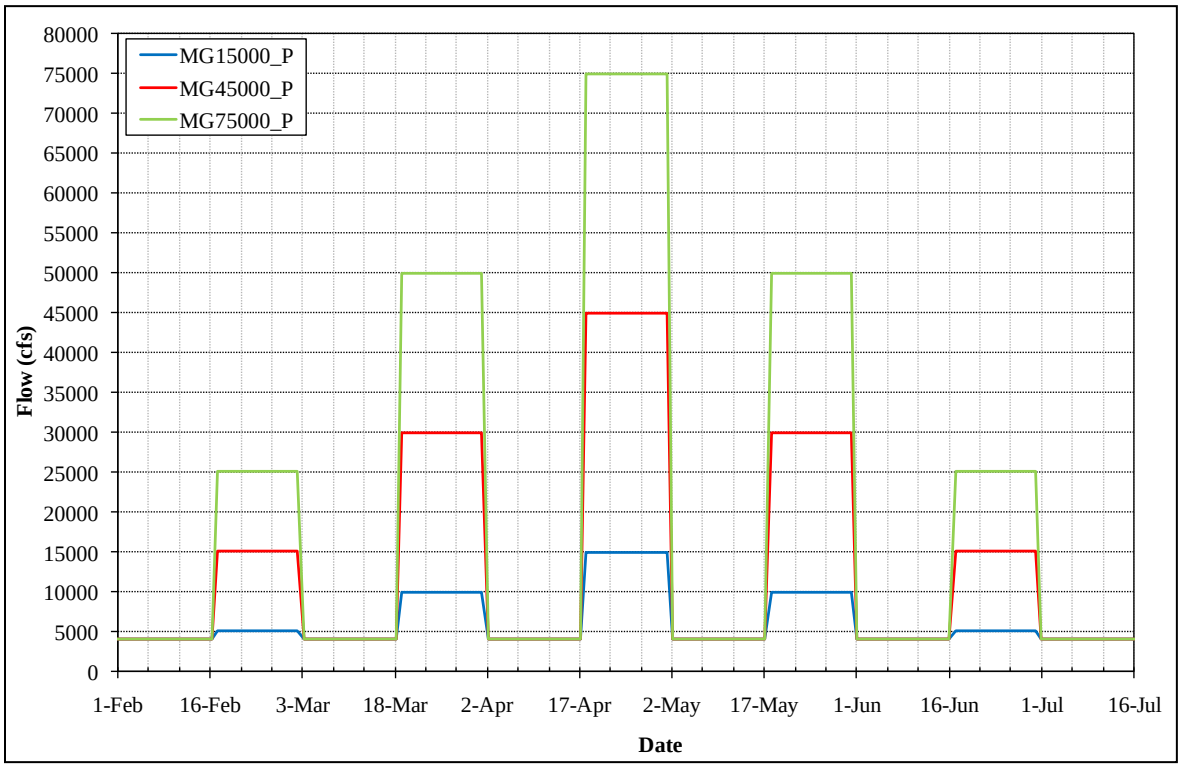

Figure 4-9: Pulsing Diversion Scenarios at Myrtle Grove

The baseline condition was kept the same as used in the Phase II study, and the three pulsing Myrtle Grove diversion scenarios were modeled in conjunction with a nominal Davis Pond diversion. After the model runs finished, the monthly average, maximum and minimum water surface elevations, as well as the monthly average and maximum current velocity magnitudes for each diversion scenario for the period of February to July were extracted and compared to the baseline condition. The results from the 17 points in Figure 4-2 were also extracted.

Figure 4-10 through Figure 4-12 demonstrate the modeled water surface elevation time series for the three pulsing diversion scenarios at three different locations: P8 – North of Round Lake at Bayou Dupont, P11 – South of Diversion Canal at Bayou Dupont, and P12 – Barataria Waterway South of the Town of Lafitte. Figure 4-13 presents the monthly average water surface elevation and velocity magnitude changes relative to the baseline condition caused by different Myrtle Grove diversion magnitudes in April when the diversion reaches its peak.

Review of these results clearly indicates that the system reacts fairly quickly to the diverted flows with water levels and currents rising rapidly upon initiation of the diversion and then decreasing rapidly once the diverted flows are reduced.

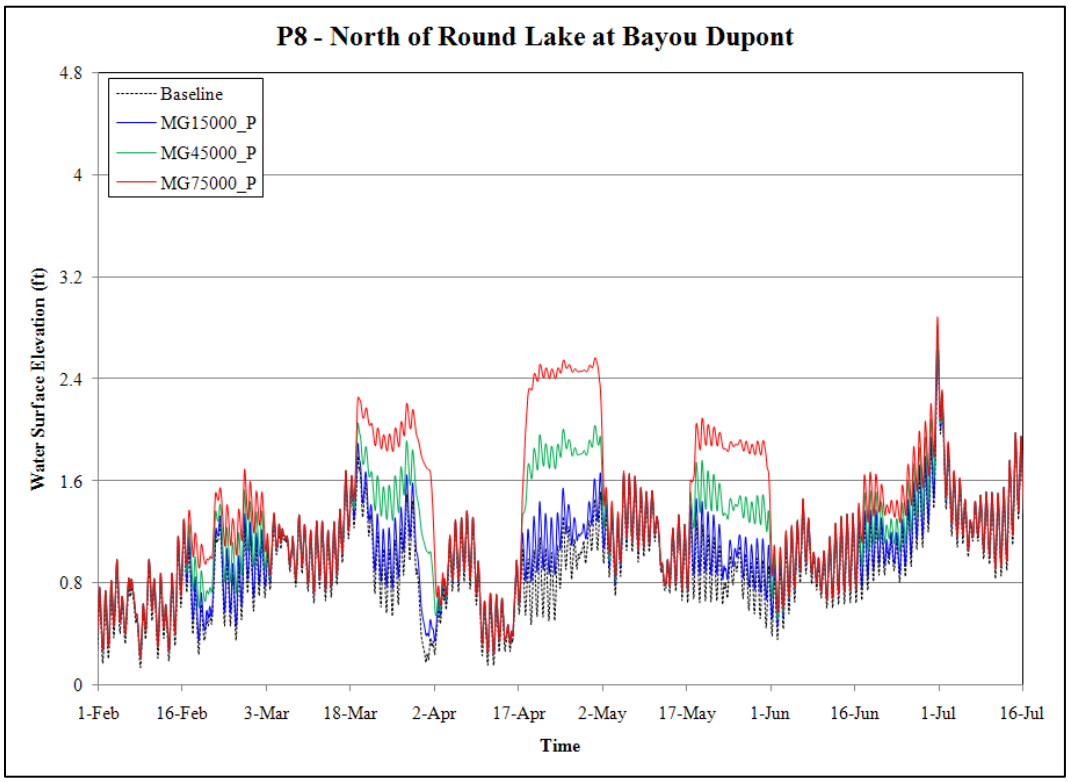

Figure 4-10: Water Surface Elevation near North of Round Lake

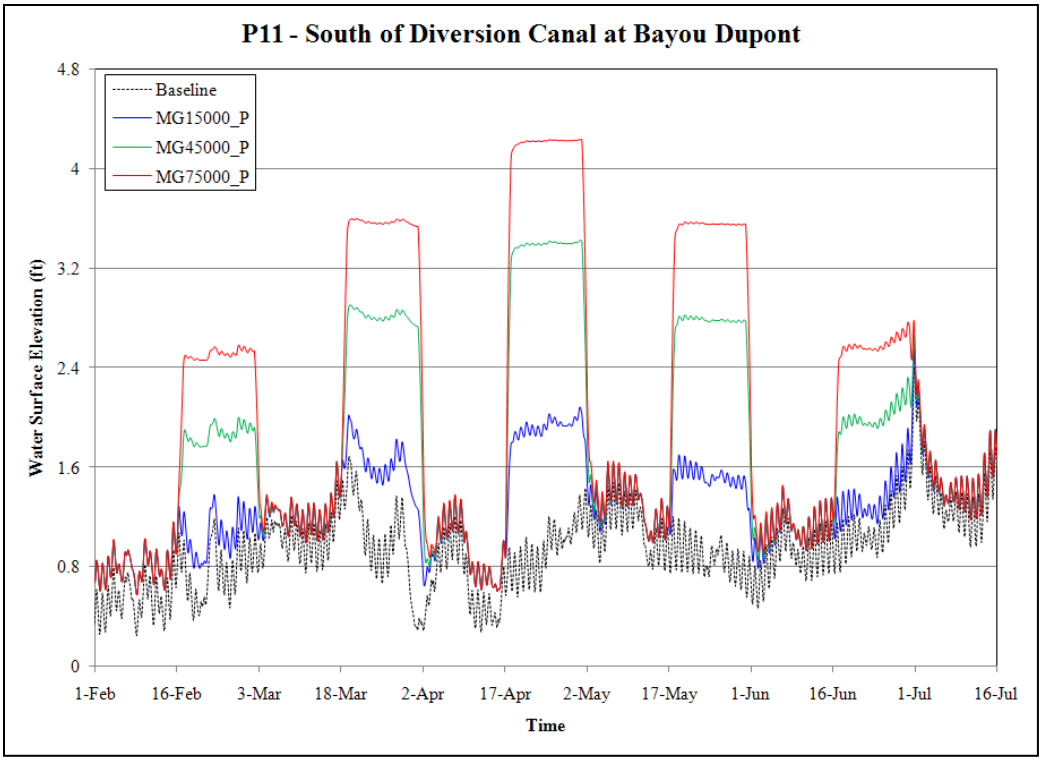

Figure 4-11: Water Surface Elevation near South of Diversion Canal

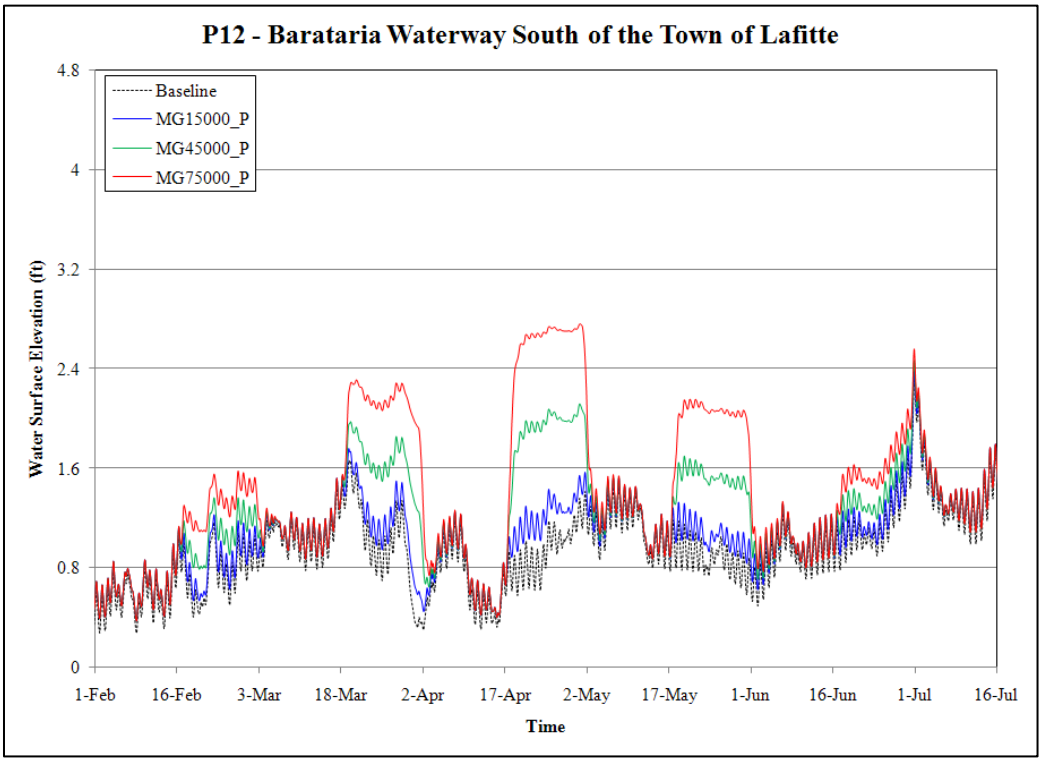

Figure 4-12: Water Surface Elevation on Barataria Waterway

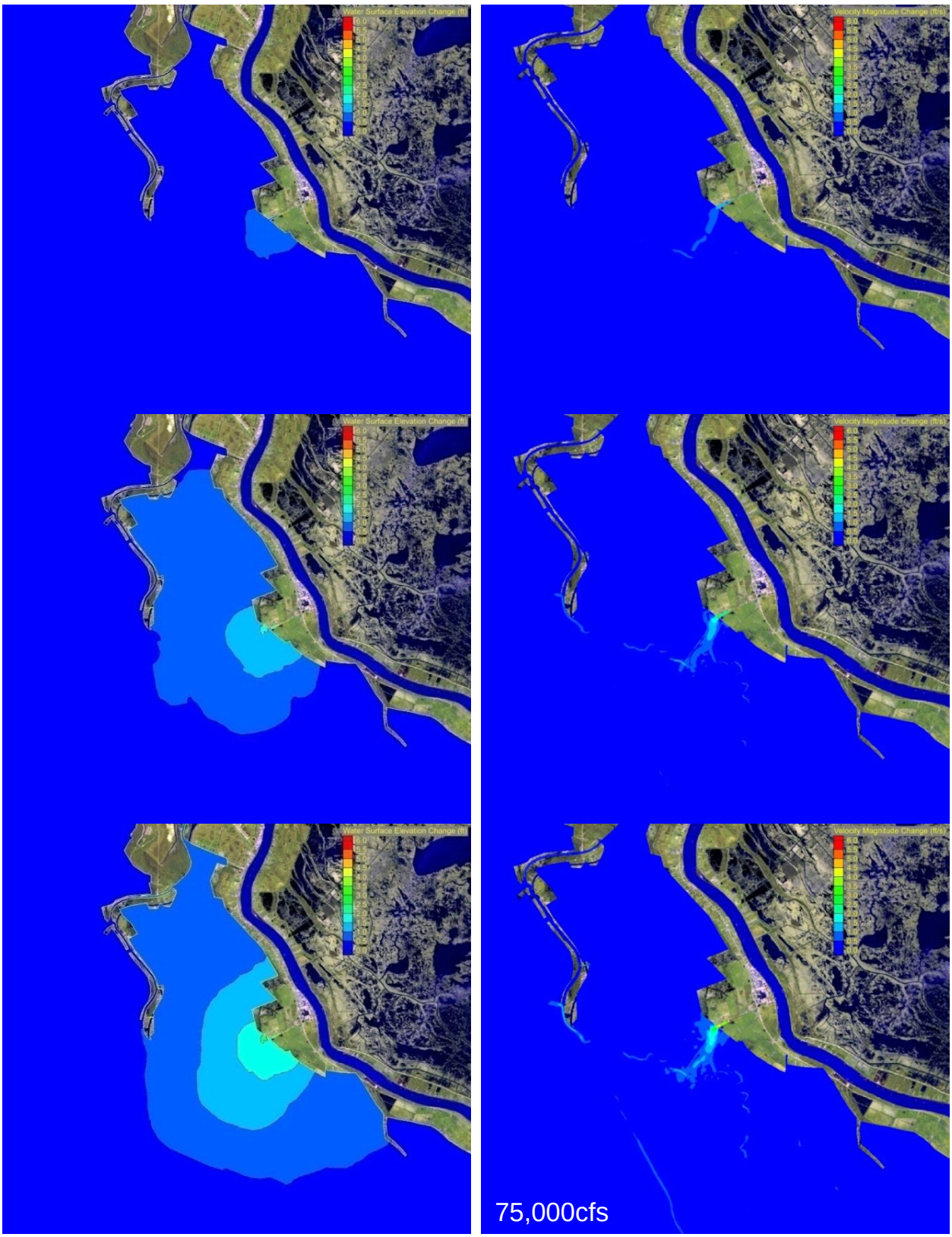

Figure 4-13: Monthly Average Water Surface Elevation and Velocity Magnitude Changes

4.4 Tail Water at Myrtle Grove Diversion

To provide boundary conditions for the diversion structure and channel modeling efforts, hydrodynamic model runs were conducted to estimate the basin stage as a function of the diverted flow at Myrtle Grove. The diverted flow conditions are: 5,000 cfs, 10,000 cfs, 15,000 cfs, 30,000 cfs, and 45,000 cfs. The offshore water level boundary condition was set to a constant value of 0.7ft NAVD88, the mean water level in April 2003. There were no wind, precipitation and rainfall runoff inputs. For each diversion flow condition, four day periods were run to obtain "steady-state" conditions, and the water levels at the discharge point were extracted from the results. Figure 4-14 presents the tail water levels.

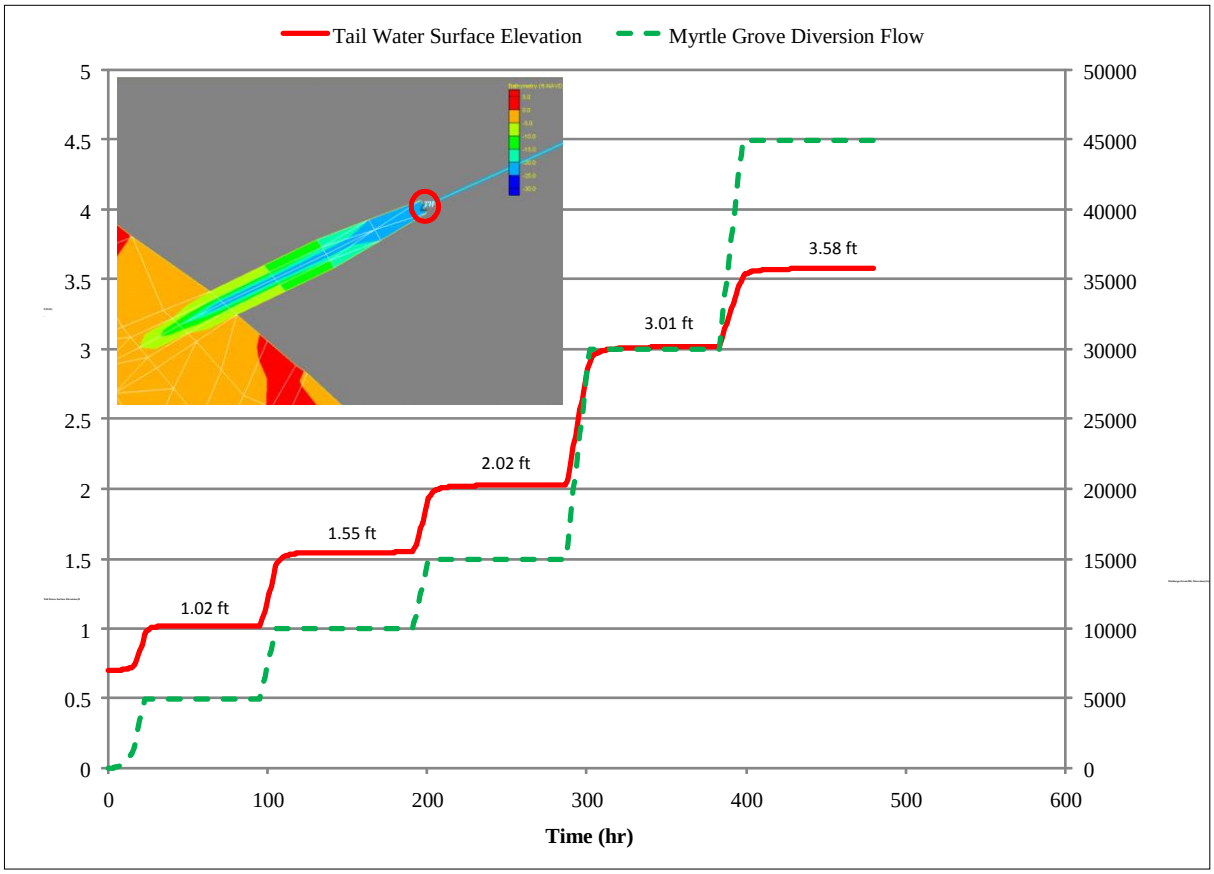

Figure 4-14: Tail Water Levels at Discharge Point

5 DATA COLLECTION

5.1 Introduction

An extensive data collection effort was lead by Mead A. Allison, PhD, Senior Research Scientist, Institute for Geophysics, University of Texas. A complete report documenting these efforts entitled, "Water and Sediment Surveys of the Mississippi River Channel Conducted at Myrtle Grove and Magnolia in Support of Numerical Modeling (October 2008 – May 2010)" is included in Appendix D.

Five main field surveys were carried out to capture a range of Mississippi River water discharges in October 2008, April 2009, May 2009, April 2010 and May 2010. In addition, a Laser In-Situ Scattering and Transmissometry (LISST) instrument was placed on the USGS Belle Chasse platform in August 2009 and operated through January 2010. Data were collected only above the bend at Myrtle Grove in October 2008 and at the Myrtle Grove and Magnolia sites in 2009-2010 (see Figure 5-1 and Figure 5-2). Data were collected under the river discharge conditions shown in Table 5-1.

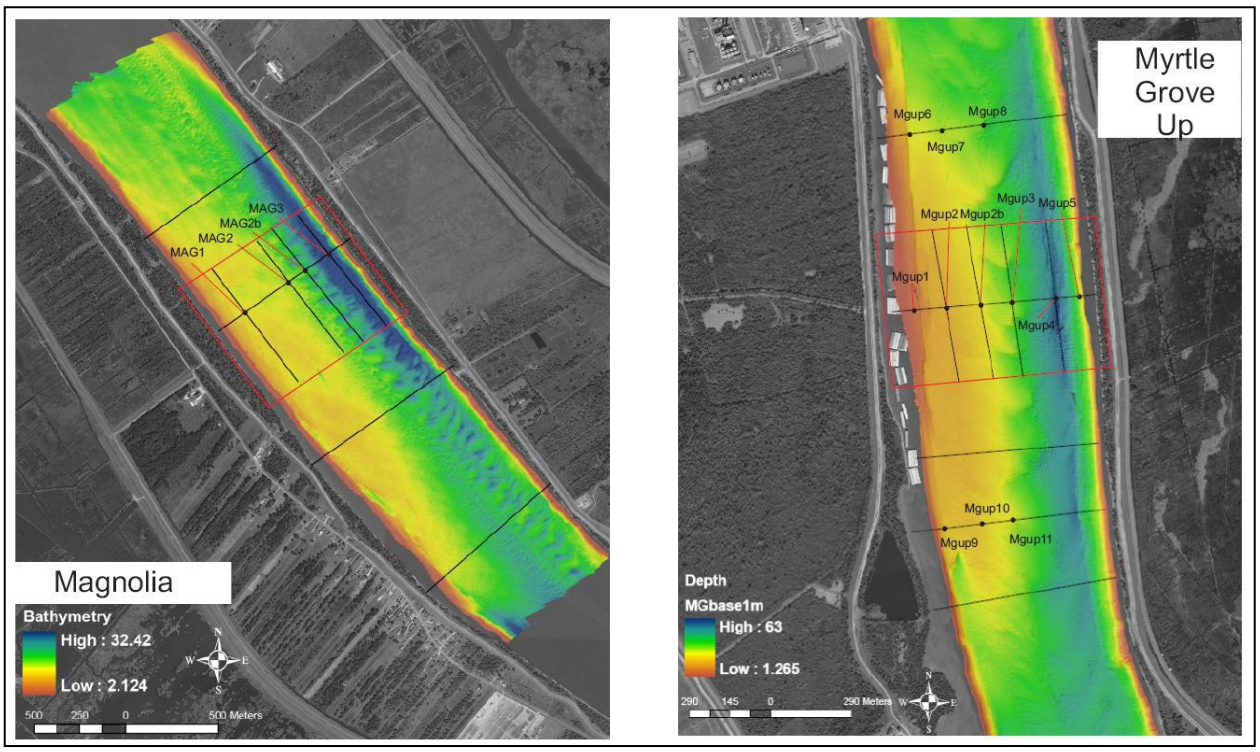

Figure 5-1: Data Collection Stations

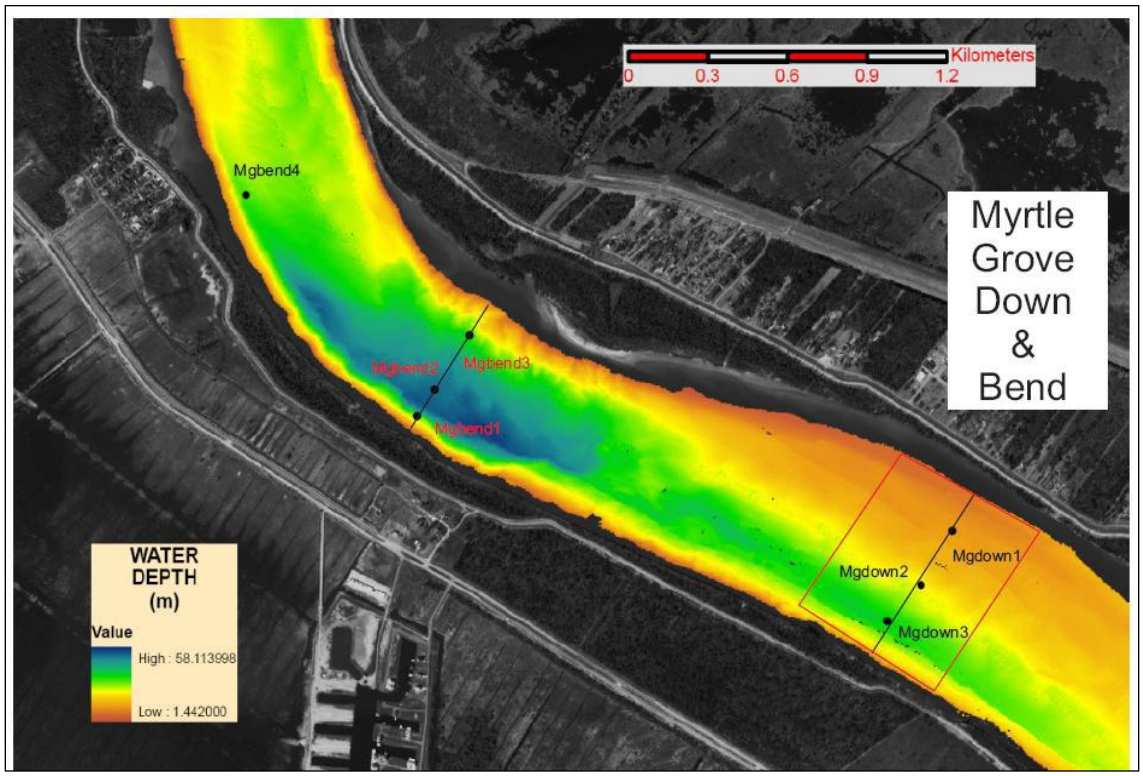

Figure 5-2: Data Collection Stations

Table 5-1: Data Collection Summary

Cruise	Date	Site**	Tarbert Landing Discharge (m^3/sec)*	Tarbert Landing Discharge (ft^3/sec)*	Activity[a]
Oct 2008	10th	MGup	9,968	352,017	Basemap, P-63, bedload, adcp
Oct 2008	11th	MGup	9,854	347,991	Basemap, CTD / LISST, grab, bedload
April 2009	4th	MAG	19,567	691,002	Bedload, adcp, CTD / LISST
April 2009	5th	MAG, MGdown	19,737	697,006	Bedload, P-63, adcp, basemap
April 2009	6th	No Collection	20,020	707,000	Due to weather
April 2009	7th	MGdown, MGup	20,331	717,983	Bedload, CTD / LISST, adcp, P-63, basemap
April 2009	8th	MAG, MGdown, MGbend	20,643	729,001	Grab, bedload, CTD / LISST
April 2009	9th	MGbend	20,954	739,984	P-63, adcp
May 2009	1st	MAG, MGdown, MGbend	22,512	795,004	Adcp, grab
May 2009	2nd	MGup, MGdown, MAG	22,370	789,989	Bedload, adcp, CTD / LISST
May 2009	3rd	MGup, MGdown, MAG	22,200	783,986	Bedload, CTD / LISST
May 2009	4th	MAG, MGup, MGbend	22,200	783,986	P-63, CTD / LISST
May 2009	5th	No Collection	22,342	789,000	Boat engine failure
May 2009	6th	MGup, MGbend, MGdown	22,710	801,996	Grab, P-63
May 2009	7th	MGdown, MGbend	23,220	820,007	Grab, adcp
April 2010	12th	MAG	25,598	903,985	Basemap
April 2010	13th	MAG, MGup	25,485	899,994	Adcp, basemap, CTD
April 2010	14th	MAG, MGup, MGbend, MGdown	24,098	851,013	Adcp, bedload, CTD / LISST
April 2010	15th	MAG, MGup, MGdown	23,956	845,998	P-63, bedload, basemap
April 2010	16th	MGup, MGbend, MGdown	23,616	833,991	Adcp, P-63, bedload
May 2010	11th	MGup, MAG	20,671	729,990	Adcp, P-63, bedload, CTD / LISST
May 2010	12th	MAG, MGup	21,096	744,998	Adcp, basemap, bedload, grab
May 2010	13th	MGup, MAG	21,606	763,009	Basemap, bedload, Adcp, grab
May 2010	14th	MGup, MAG	22,115	780,984	Bedload, basemap, grab, adcp
May 2010	15th	MAG, MGup	22,596	797,970	P-63, adcp, bedload, CTD / LISST

*Water discharge daily averages at Tarbert Landing, MS were obtained at http://www.mvn.usace.army.mil/cgi-gin/wcmanual.pl?01100

**MGup = Myrtle Grove survey area above the bend; MGbend = Myrtle Grove survey area within the bend; MGdown = Myrtle Grove survey area below (downriver) of the bend; MAG = Magnolia

[a]Basemap = multibeam basemap mapping; P-63 = isokinetic water sampling with P-63 point integrative sampler; bedload = multibeam repeat bathymetric mapping for bedload transport measurements; grab = bottom grab sampling; adcp = acoustic doppler current profiler cross-sections of water discharge and backscatter (suspended sediment)

5.2 Multibeam Data Collection and Post-Processing

Multibeam bathymetric surveys were collected for two purposes:

1) To build a bathymetric basemap of the channel floor in the Myrtle Grove and Magnolia regions to provide a detail grid for 3-D numerical modeling, and
2) To calculate bedload transport rates at several sites near Myrtle Grove and Magnolia

The extent of the Myrtle Grove basemap is shown in Figure 5-3.

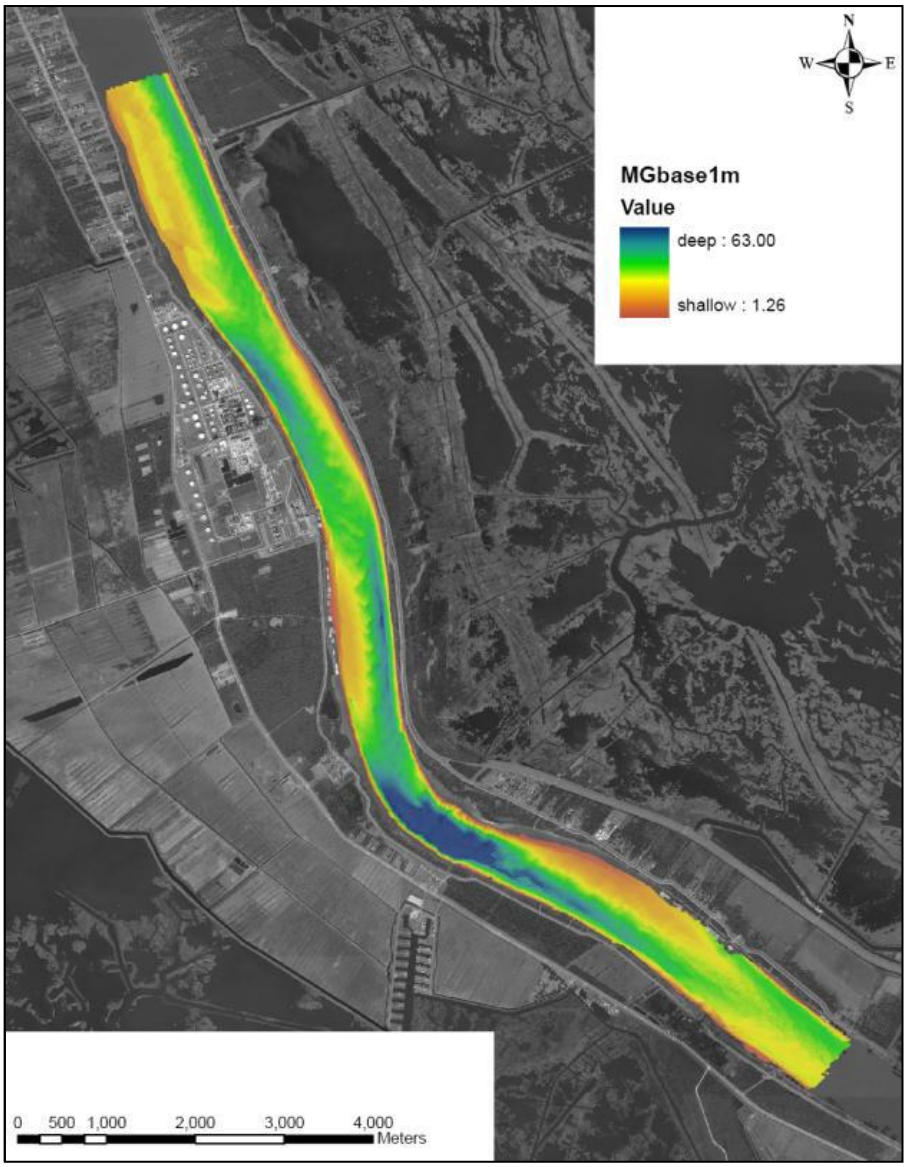

Figure 5-3: Limits of Multibeam Bathymetric Basemap

5.3 Bedload (Bedform) Sediment Transport Rates

Raster grids were constructed using the bathymetric survey results and repeat surveys were then compared; and visual identification of crest locations confirmed that dunes had migrated less than one wavelength between surveys. Volumes of sand deposited were calculated and converted to derive a final bedform translational flux for each river cross-section. Bedload (bedform) flux rate results for each site and cruise are shown in Table 5-2.

Table 5-2: Bedload Mass Flux

Survey Site*	Date	River Mile**	Channel Mass Flux (tonne/day)[a]
MGup (2008)	10/10-11	61.6	1511.4
MAG (2009)	4/4-5	46.6	2718.2
MGdown (2009)	4/7-8	58.0	3190.7
MGup (2009)	4/7-8	61.6	4822.4
MGup (2009)	5/2-3	61.6	6683.5
MGdown (2009)	5/2-3	58.0	5419.3
MAG (2009)	5/2-3	46.6	5942.8
MGup (2010)	4/14-15	61.6	3606.6
MAGb (2010)	4/14-15	46.6	14101.8
MGdown (2010)	4/15-16	58.0	13795.4
MGupAB (2010)	5/11-12	61.6	747.7
MGupBC (2010)	5/12-13	61.6	1353.5
MGupCD (2010)	5/13-14	61.6	1652.6
MGupDE (2010)	5/14-15	61.6	2898.6

*MGup = Myrtle Grove survey area above the bend; MGbend = Myrtle Grove survey area within the bend; MGdown = Myrtle Grove survey area below (downriver) of the bend; MAG = Magnolia

**River mile of the center of the survey reach

[a]Figures are in metric tonne (1 MT = 1,000 kg)

5.4 Water Discharge

Table 5-3 presents cross-sectional water discharge data that was collected in 2008-2010 along a series of transects that included all four study grids (MGup, MGbend, MGdown, MAG) as shown in Figure 5-1 and Figure 5-2.

Table 5-3: Discharges

Survey Site*	Study Dates	River Mile**	Discharge at Tarbert Landing, MS (m^3/sec)	Discharge at Tarbert Landing, MS (ft^3/sec)	Mean Measured Discharge (m^3/sec)	Mean Measured Discharge (ft^3/sec)	Standard Deviation of Discharge (m^3/sec)	Standard Deviation of Discharge (ft^3/sec)
MGup	10/10/08	61.6	9,968	352,017	11,253	397,396	491	17,340
MGup	04/07/09	61.6	20,331	717,983	19,964	705,022	512	18,081
MGabovebend	04/09/09	59.9	20,954	739,984	20,980	740,902	423	14,938
MGbend	04/09/09	59.3	20,954	739,984	18,458	651,838	4,402	155,455
MGbelowbend	04/09/09	58.9	20,954	739,984	18,682	659,749	963	34,008
MGdown	04/05/09	58.0	19,737	697,006	18,886	666,953	1,349	47,639
MGdown	04/09/09	58.0	20,954	739,984	20,862	736,735	1,081	38,175
MAG	04/04/09	46.6	19,567	691,002	18,987	670,520	609	21,507
MGup	05/02/09	61.6	22,370	789,989	21,705	766,505	432	15,256
MGabovebend	05/07/09	59.9	23,220	820,007	19,981	705,622	-	-
MGbend	05/07/09	59.3	23,220	820,007	20,214	713,851	1,325	46,792
MGbelowbend	05/07/09	58.9	23,220	820,007	21,078	744,363	-	-
MGdown	05/01/09	58.0	22,512	795,004	20,697	730,908	956	33,761
MAG	05/01/09	46.6	22,512	795,004	21,092	744,857	996	35,173
MGup	04/13/10	61.6	25,200	889,930	24,767	874,638	914	32,278
MAG	04/13/10	46.6	25,200	889,930	23,861	842,643	842	29,735
MGup	04/16/10	61.6	23,616	833,991	23,797	840,383	206	7,275
MGbend	04/16/10	59.3	23,616	833,991	22,901	808,741	1,444	50,994
MGdown	04/16/10	58.0	23,616	833,991	22,819	805,845	1,699	60,000
MGup	05/11/10	61.6	20,440	721,832	18,770	662,856	459	16,209
MAG	05/11/10	46.4	20,440	721,832	19,343	683,092	1,233	43,543
MGup	05/12/10	61.6	20,860	736,664	19,548	690,331	533	18,823
MAG	05/12/10	46.4	20,860	736,664	19,891	702,444	1,326	46,827
MGup	05/13/10	61.6	21,364	754,463	19,885	702,232	409	14,444
MAG	05/13/10	46.4	21,364	754,463	20,327	717,841	1,416	50,006
MGup	05/14/10	61.6	21,868	772,261	22,332	788,647	619	21,860
MAG	05/14/10	46.4	21,868	772,261	20,443	721,938	941	33,231
MGup	05/15/10	61.6	22,344	789,071	20,572	726,493	858	30,300
MAG	05/15/10	46.4	22,344	789,071	20,701	731,049	1,241	43,826

*MGup = Myrtle Grove survey area above the bend; MGbend = Myrtle Grove survey area within the bend; MGdown = Myrtle Grove survey area below (downriver) of the bend; MAG = Magnolia; MGabovebend = Myrtle Grove survey area between MGup and MGbend; MGbelowbend = Myrtle Grove survey area between MGbend and MGdown.

**River mile of the center of the survey reach

5.5 Isokinetic Suspended Sediment Samples and Suspended Sediment Loads

Suspended sediment point samples were collected with a P-63 isokinetic sampler at various depths in the water column. Suspended sediment loads were calculated from these results and the measured discharges and are presented in Table 5-4.

Table 5-4: Suspended Sediment Loads

Survey Site*	Date (P-63) Date (ADCP)	Ratio**	Corr Sand (tonne/d)	Corr Mud (tonne/d)
MGup	10/10/08 10/10/08	1.000	1,006	115,410
MGup	04/07/09 04/07/09	1.000	97,118	259,344
MGbend	04/09/09 04/09/09	1.000	71,335	224,535
MGdown	04/05/09 04/05/09	1.000	81,592	275,776
MAG	04/05/09 04/04/09	1.009	100,400	262,646
MGup	05/04/09 05/02/09	0.992	28,278	176,734
MGbend	05/06/09 05/07/09	0.978	37,262	172,362
MGdown	05/06/09 05/01/09	1.009	30,502	171,553
MAG	05/04/09 05/01/09	0.986	109,612	174,736
MAG	04/15/10 04/13/10	0.940	44,890	154,364
MGup	04/15/10 04/13/10	0.940	44,597	166,851
MAG	05/11/10 05/11/10	1.000	19,209	468,051
MGup	05/11/10 05/11/10	1.000	49,502	500,229
MAG	05/15/10 05/15/10	1.000	41,558	321,127
MGup	05/15/10 05/15/10	1.000	49,531	274,062

*MGup = Myrtle Grove survey area above the bend; MGbend = Myrtle Grove survey area within the bend; MGdown = Myrtle Grove survey area below (downriver) of the bend; MAG = Magnolia in 2009/April 2010

**Refers to the ratio of water discharge at Tarbert Landing, MS on the P-63 data collection day versus the ADCP data collection day. This is the ratio utilized in the corrected sediment load columns.

5.6 Sediment Bottom Grabs

Sediment bottom sampling was conducted where suspended sediment concentration data was collected except those locations where the depth exceeded 150 feet. Analyses of the samples were performed and the results are presented in Appendix D.

5.7 CTD / LISST Cast Data

Profiles from the CTD (conductivity / temperature / depth) containing the following data was collected:

- Depth (m)
- OBS-3 Backscatter (nephelometric turbidity units – NTU) (see Figure 5-4)
- pH
- Oxygen (ml/L)
- Beam Transmission (fractional %)
- Conductivity (uS/cm)
- Salinity (PSU)
- Temperature (degrees C)

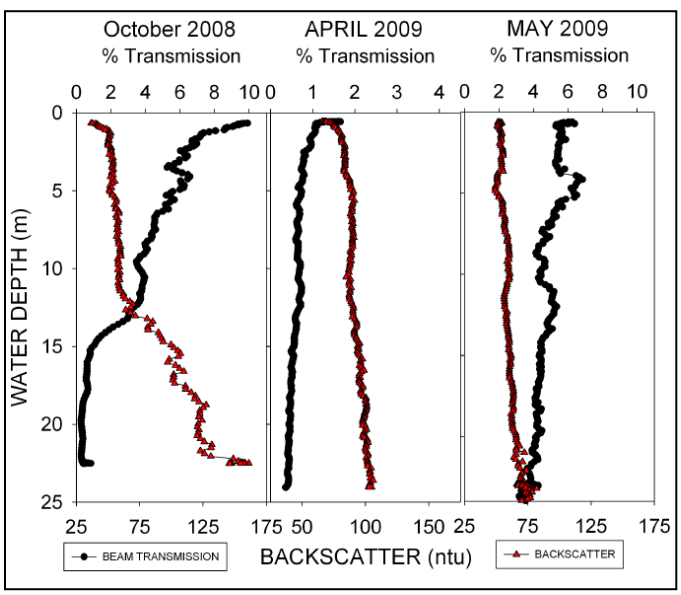

Figure 5-4: Example of Optical Turbidity Data

Additionally, LISST suspended sediment grain size data was collected. Figure 5-5 shows the depth profiles from the same station (MGup) that the turbidity data is shown for in Figure 5-4.

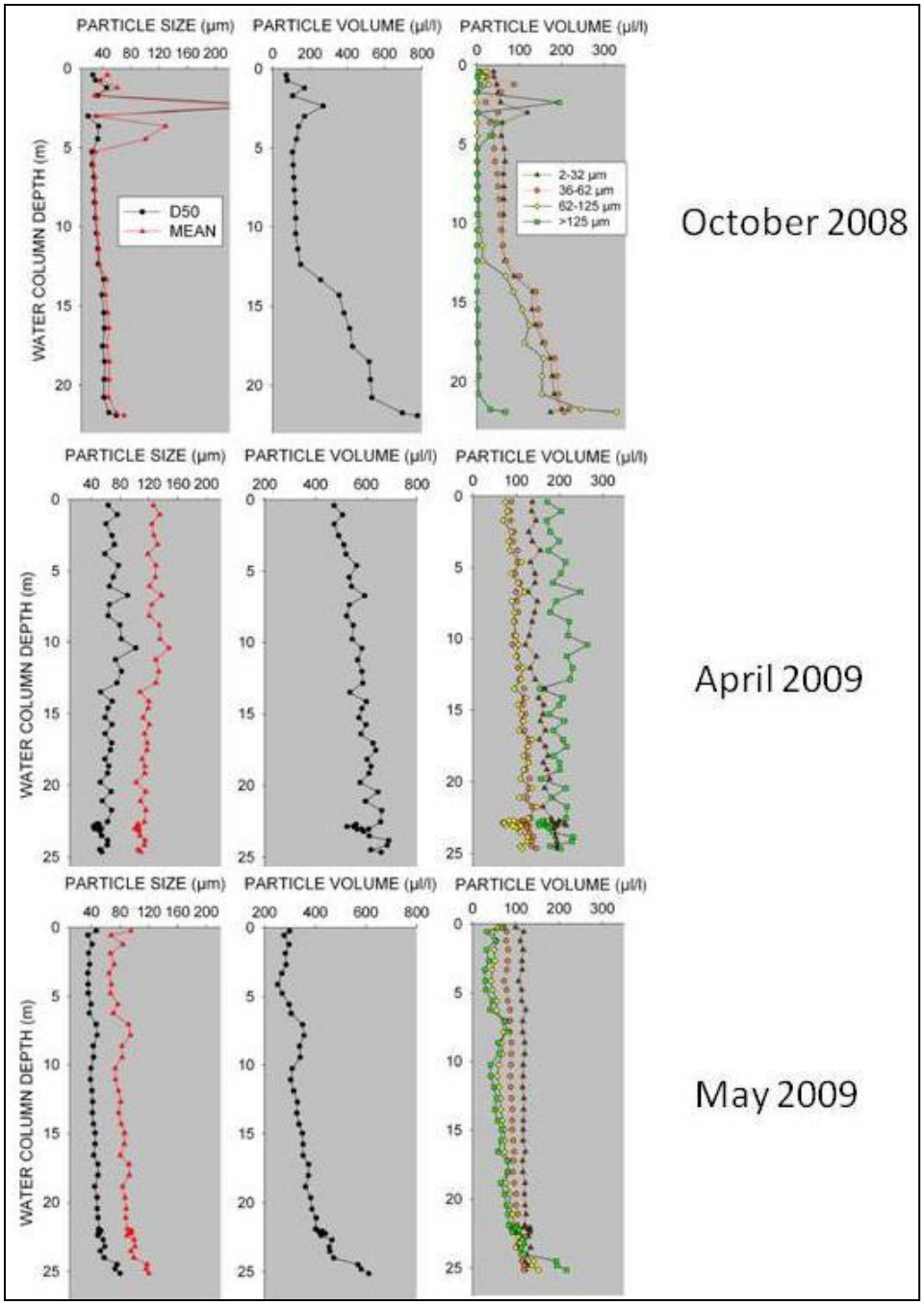

Figure 5-5: Example of LISST Data

6 RIVER-SIDE HYDRODYNAMIC MODELING

Numerical modeling of the hydrodynamics and sediment transport in the Lower Mississippi River near the Myrtle Grove River Bend was performed by Ehab Meselhe, PhD, PE; Ioannis Georgiou, PhD, PE; and John McCorquodale, PhD, PE. A complete copy of their report is included in Appendix E.

6.1 Data Collection

The field data collection effort included two main components, setting up real-time monitoring stations (discussed herein) and gathering boat-based observations (discussed in Chapter 5). The real-time field observations are intended to supplement the boat-based observations and provide an overall understanding of the dynamics of the Lower Mississippi River near the proposed Myrtle Grove Delta Building Diversion Project. The area of interest here is in Plaquemines Parish, Louisiana (Mississippi River Mile 55 to 65 Above Head of Passes). Monitoring stations were established at three locations (Table 6-1 and Figure 6-1). These locations were selected to monitor stage, temperature, turbidity, and conductivity upstream and downstream of the Myrtle Grove Delta Building Diversion Project area.

Table 6-1: Myrtle Grove Diversion Study Area

Station ID	Location	Mississippi River Mile
1	Belle Chasse, LA	Mile 72.8
2	ConocoPhillips Alliance Refinery	Mile 63.2
3	Buras, LA	Mile 24.2

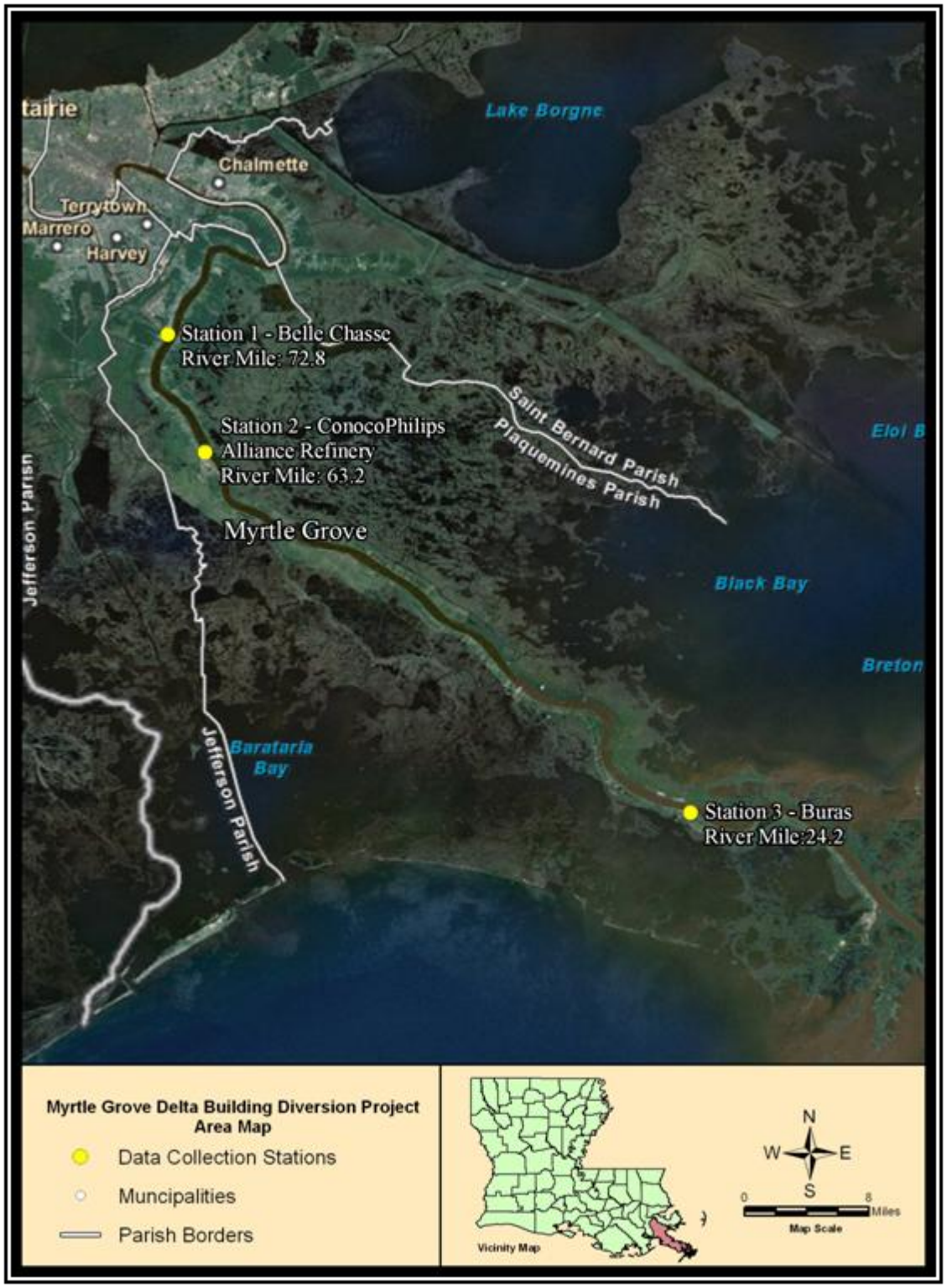

Figure 6-1: Myrtle Grove Diversion Study Area

6.2 Numerical Modeling

Although the main focus of this report is the three-dimensional modeling effort performed at the local scale around the Myrtle Grove project area, it should be noted that a large scale one-dimensional model was also used. The US Army Corps of Engineers code HEC-RAS (Hydrologic Engineering Center – River Analysis System) was used for this one-dimensional effort. The model extends from Tarbert Landing to the Head of Passes. The one-dimensional model was used to develop an overall understanding of the Lower Mississippi River at large temporal and spatial scales. It was also used to provide boundary conditions for the more detailed and local three-dimensional model.

The three-dimensional model was used to develop an understanding of the complex flow field near the Myrtle Grove river bend and examine the interaction between flow and sediment. It is important to identify a model capable of capturing patterns such as the migration of sand near the bar on the west bank (upstream of the Myrtle Grove bend), through the river bend and then across toward the bar on the east bank (downstream of the Myrtle Grove bend).

The FLOW-3D model by Flow Science is suitable for this study as it has the adequate physical processes and computational efficiency. Further, the field observations were adequate to set up and calibrate the model described above.

Overall, the objectives of this modeling effort include:

- Capture the complex three-dimensional flow field in the vicinity of possible diversion sites.
- Quantify the sand load available for diversion under various flow conditions.
- Quantify the fraction of the available sand load that can be diverted for various conveyance channel alignments and configurations.
- Investigate the potential for shoaling and head-cutting as a result of sediment diversions.

6.3 Model Setup

The model domain extended from River Mile (RM) 56.0 (above the head of passes – AHP) to RM 62.7 AHP (Figure 6-2). As shown in Figure 6-3, this area has been surveyed using a high-resolution multi-beam survey. This river reach encompasses the potential location for the sediment diversion site.

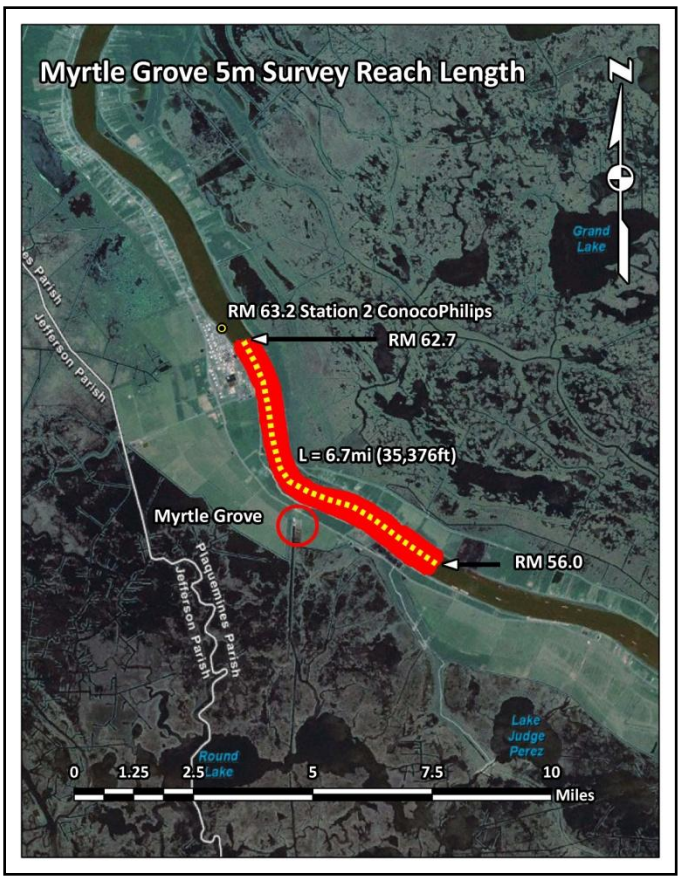

Figure 6-2: FLOW-3D Model Domain

The multi-beam data, collected by a boat, does not capture the shallow areas near the bank lines. Hence it was supplemented by the US Army Corps of Engineers decadal single beam surveys in order to have fully bathymetric information of this river reach. The over bank topographic data was captured by LiDAR data. The three data sets, namely multi beam, single beam and LiDAR data, were combined as shown in Figure 6-4.

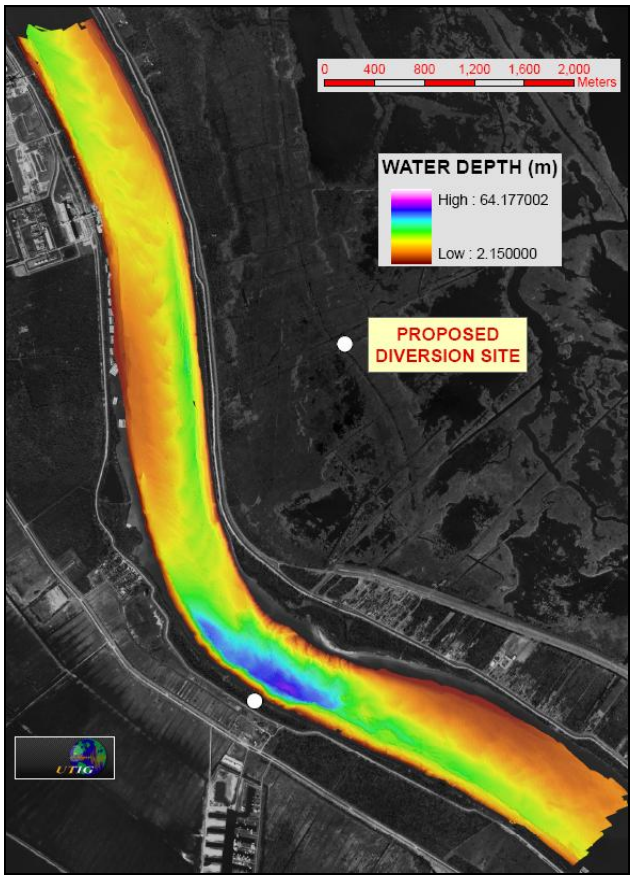

Figure 6-3: Multi-Beam Bathymetry Map

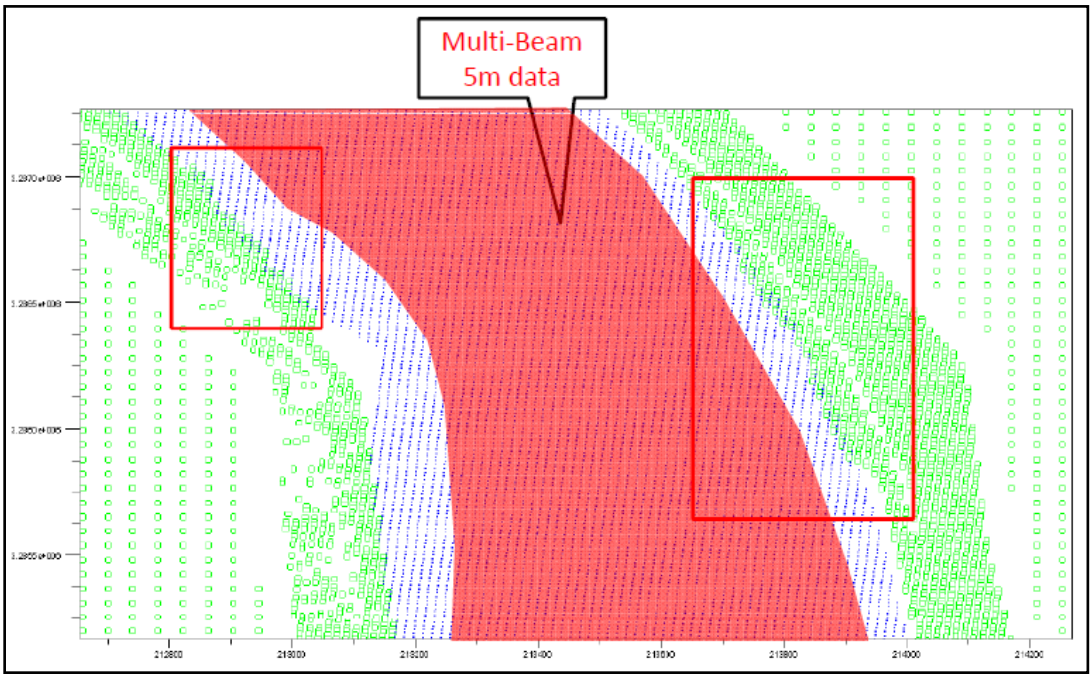

Figure 6-4: Supplementing the Multi-Beam Data with Single-Beam and LiDAR Data (Red points: multi-beam; blue points: single beam; green points: LiDAR)

6.4 Boundary Conditions

The model was calibrated using the field data of April 2009. The water flow rate was used as the upstream boundary condition, while the tail water was used as the downstream boundary condition. The water flow during this field campaign was 700,000 cfs (19821.79 m^3/s). There is no tail water information available at the downstream end of the model domain, and therefore a numerical simulation using HEC-RAS as well as the measurements at the Conoco-Philips (RM63.2) station were used to estimate a tail-water. The tail-water estimate was 6.23 ft-NAVD88 (1.9 m).

The model was validated against the April 2010 events. The water flow during the April 2010 event was 840,000 cfs (23,786 m^3/s). Again HEC-RAS simulations combined with information from the continuous station at Conoco-Philips (RM 63.2) were used to estimate the tail water. For April 2010, tail water in the range of 5.90 to 6.56 ft-NAVD88 (1.8 to 2.0 m) was used.

6.5 Diversion Scenarios

Once calibrated and validated, the model was used to test various diversion designs. The numerical experimentations included various alignments, intake locations, and sizes of the intake channel and diversion structure.

6.5.1 Original (USACE) Alignment (RM 60.2) [OA-RM60.2-15K]

The general layout of the design is shown in Figure 6-5 and Figure 6-6. This design is intended to divert a maximum of 15,000 CFS when the Mississippi River discharge is at 1,000,000 CFS. A HEC-RAS model was used to design the geometry (size and slope) of the diversion channel such that it would carry a flow of 15,000 CFS. It should be noted that the HEC-RAS model did not include (explicitly) the energy losses due to bends along the length of the diversion channel and due to the transitions from one cross section shape/size to another.

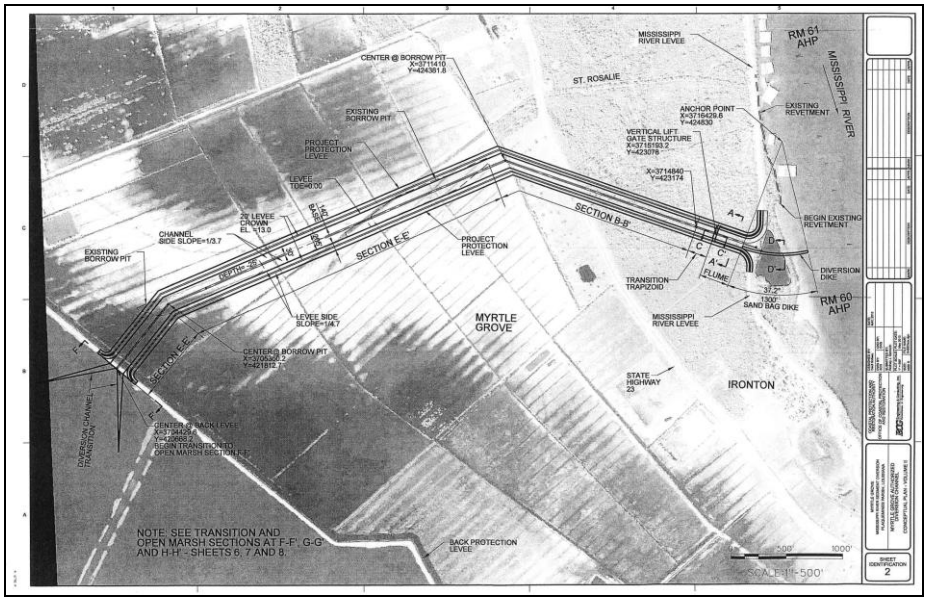

Figure 6-5: Layout of the Original (USACE) Alignment (RM 60.2 – 15 K CFS) [OA-RM60.2-15K]

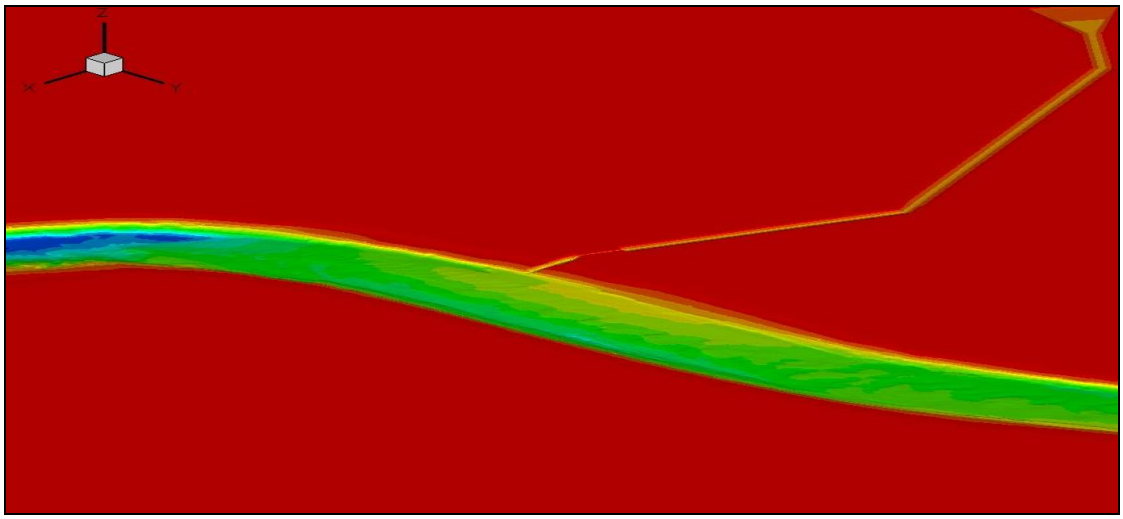

Figure 6-6: Diversion Channel

[OA-RM60.2-15K]

The April 2009 event (same event that was used to calibrate the model) was used to assess the performance of this design. A tail-water elevation of 5 ft-NAVD88 (1.524 m) was set at the downstream end of the diversion channel (basin side). This estimate reflects typical conditions observed in the basin side. The mouth of the intake channel at the river side is set at -40 ft-NAVD88. The channel gradually slopes up. At the release point at the basin side, the channel invert is -25 ft-NAVD88 (-7.62 m). Immediately after the entrance from the river side, the channel has rectangular cross section with a width of 30 ft (9.14 m). The channel then transitions into a trapezoidal cross section (bottom width = 30 ft (9.14 m), and side slopes of 3:1 (H:V)). The channel has two bends as shown in Figure 6-5 and Figure 6-6. The channel cross section transitions into a trapezoidal cross section with bottom width = 30 ft (9.14 m), and a compounded side slope of 3.7:1 and 4.7:1 (H:V). For cross section details see the conceptual design report in Appendix C.

A nested grid (multi-grid) approach was used to capture the flow field within the narrow intake of the diversion channel. Figure 6-7 through Figure 6-9 show the details of the flow field as captured by the numerical model. The energy losses captured by FLOW-3D due to the bends and cross section transitions resulted in a reduction of the diversion channel capacity. For the river conditions used in this case (River Flow = 700,000 cfs (19,821 m^3/s), River tail-water = 6.23 ft-NAVD88 (1.9 m), diversion channel tail water = 5 ft-NAVD88 (1.524 m)), the flow captured by the diversion channel was 11,369 cfs (322 m^3/s).

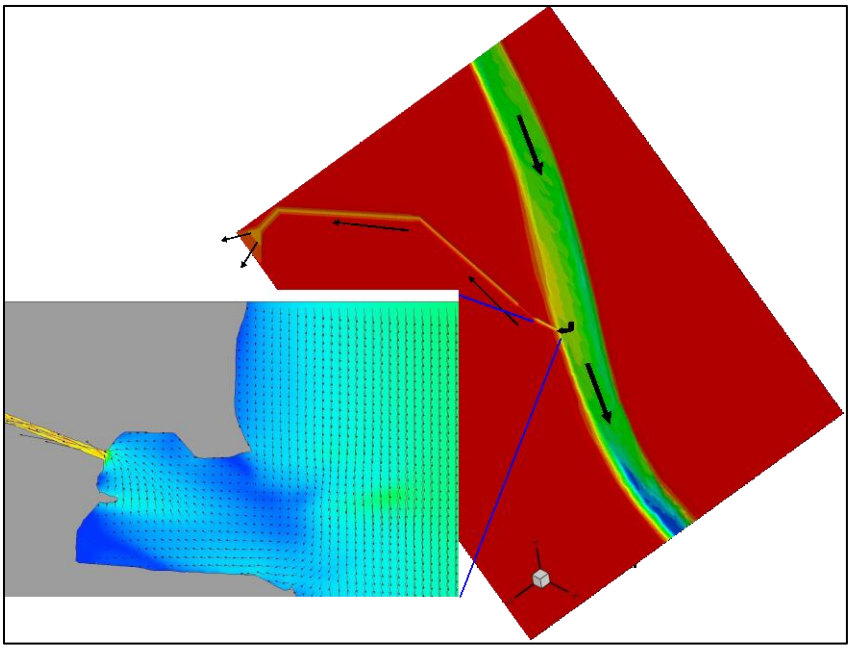

Figure 6-7: Water Drawn into the Intake of the Diversion Channel [OA-RM60.2-15K]

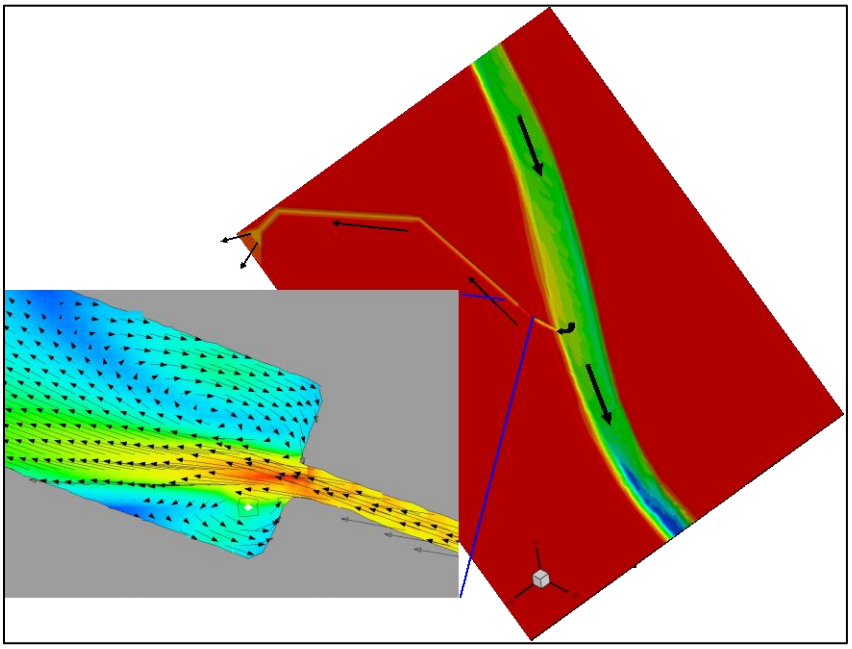

Figure 6-8: Flow Field at Transition Zone [OA-RM60.2-15K]

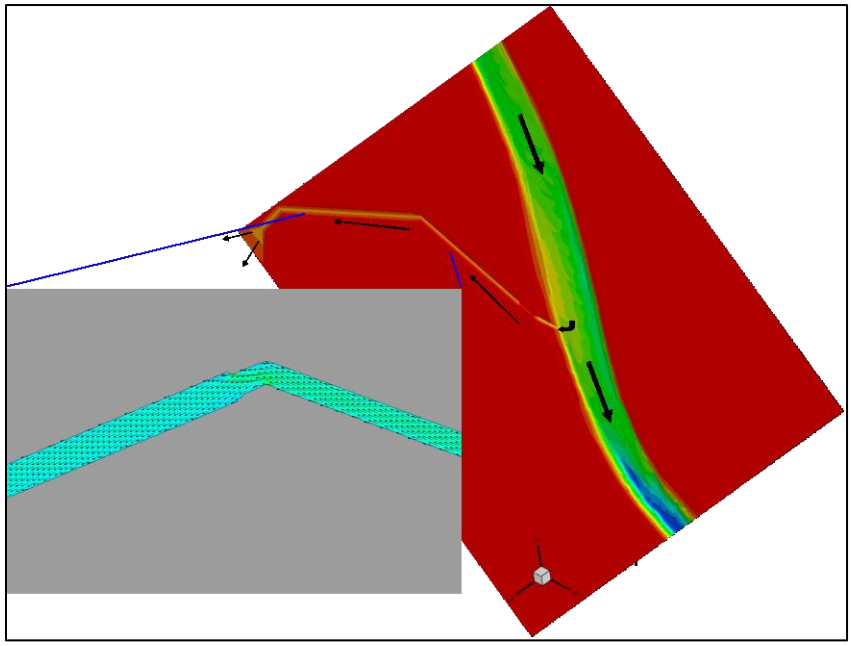

Figure 6-9: Flow Field at the 120 bend

[OA-RM60.2-15K]

As indicated previously, the sediment modeling approach used was suitable to assess the ability of the diversion channel to capture various sediment size classes. The sediment load for every size class that was captured in the diversion channel was calculated, and a summary of the sediment load captured in the diversion channel is shown in Table 6-2.

Table 6-2: Sediment Load (OA-RM60.2-15K)

	Mississippi River (Main Stem)	Diversion Channel OA-RM60.2-15K	Ratio
Water Discharge (m3/s)	19,821	322	1.62
Water Discharge (CFS)	700,000	11,369	1.62
Sediment Load (metric tons/d) - 32 Micron	233,539	2,786	1.19
Sediment Load (metric tons/d) - 63 Micron	10,839	104	0.96
Sediment Load (metric tons/d) - 96 Micron	21,816	144	0.66
Sediment Load (metric tons/d) - 125 Micron	34,437	133	0.39
Sediment Load (metric tons/d) - 250 Micron	23,460	2	0.01
Total 63 - 250 Micron Load (metric tond/d)	90,554	383	0.42

The main objective of this diversion project is to build land. Thus, it is a goal to maximize the sediment load captured in the diversion channel. It is also critical to maximize the ratio of sediment load to water, or Sand/Water Ratio (SWR), captured in the diversion channel to minimize (or completely eliminate) the potential of shoaling downstream of the diversion intake. The following simple indicator was used to assess the performance of each diversion design tested in this study where SWR is the Sand/Water Ratio.

SWR = (Sand Load Diverted/Sand Load in the River) /
(Water Discharge Diverted/Water Discharge in the River).

As shown in Table 6-2, this design extracted 1.62% of the main river water discharge, while it only extracted 1.19% of the main river silt load and 0.42% of the main river sand (size range 63 – 250 microns) load. As such, the SWR is 0.26. This is not a healthy ratio and is indicative of a poor efficiency of this design to capture an adequate amount of sediment.

6.5.2 Modified Alignment (RM 60.7) [MA-RM60.7-45K]

A modified alignment was proposed to increase the diversion's ability to capture sediment and improve the sediment-water ratio in the diversion channel as compared to the sediment-water ratio in the main stem of the river. The intake of the diversion channel was moved upstream approximately 2,640 ft (800 m). This move placed the intake at the tail end of the sand bar on the right-descending bank. Further, bends were removed from the alignment (see Figure 6-10), reducing the energy losses along the length of the diversion channel.

Several designs were tested for this alignment. The first test was for a diversion channel with a capacity of 45,000 cfs (1,274 m3/s). This design was tested with and without the earthen guide dike to assess its ability to enhance the performance of the diversion. Other tests include a diversion channel with a capacity of 15,000 and 75,000 cfs. For these tests, the channel was designed using HEC-RAS.

The first test performed for this alignment was the 45,000 cfs with the guide dike in place. The bottom width of the trapezoidal channel was 90 ft (27.4 m) and the side slopes were 3:1 (H:V). The channel transitioned to a trapezoidal channel with a bottom width of 90 ft (27.4 m) and a compounded side slope.

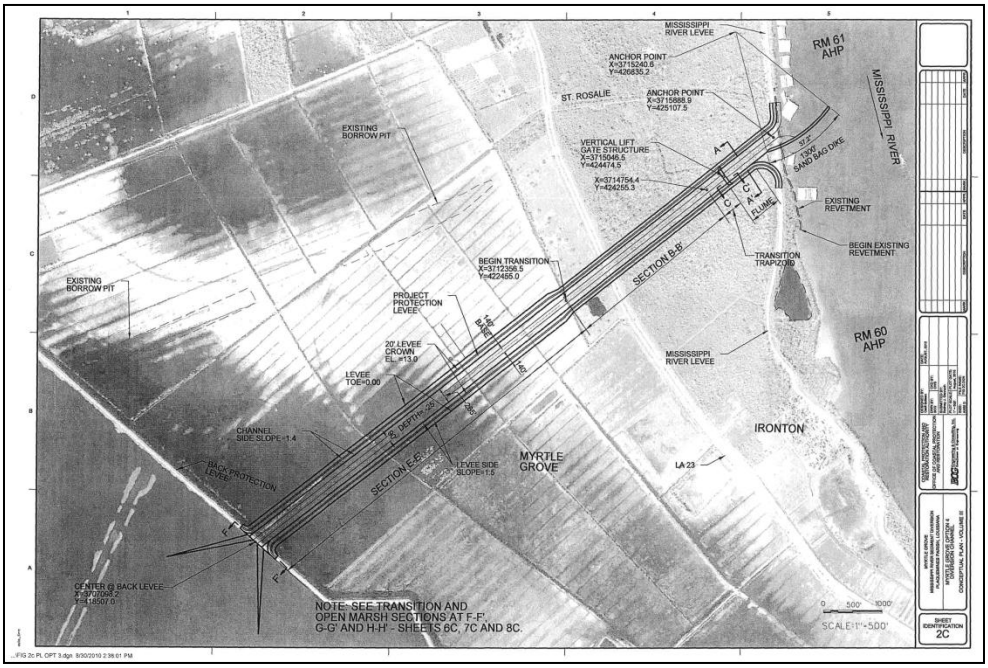

Figure 6-10: Layout of Modified Alignment (RM 60.7) [MA-RM60.7-45K]

The same river conditions that were used for the Original Alignment were used here for the Modified Alignment (River Flow = 700,000 CFS (19,821 m^3/s), River tail-water = 6.23 ft-NAVD88 (1.9 m), diversion channel tail water = 5 ft-NAVD88 (1.524 m). As mentioned before, for a nominal discharge of 1,000,000 cfs in the river, the design capacity of the diversion channel is 45,000 cfs. However, under the flow conditions used in this test (river flow of 700,000 cfs), the flow captured by the diversion channel was 33,735 CFS (955 m^3/s). A summary of the sediment load captured in the diversion channel is shown in Table 6.2.

As shown in Table 6-3, this design extracted 4.82% of the main river water discharge and 6.55% of silt, and 4.14% of sand (size range 63 – 250 microns). This performance is much improved (SWR = 0.85) compared to the Original Alignment.

Table 6-3: Sediment Load (MA-RM60.7-45K)

	Mississippi River (Main Stem)	Diversion Channel MA-RM60.7-45K	Ratio
Water Discharge (m3/s)	19,821	955	4.82
Water Discharge (CFS)	700,000	33,735	4.82
Sediment Load (metric tons/d) - 32 Micron	233,539	15,306	6.55
Sediment Load (metric tons/d) - 63 Micron	10,839	663	6.12
Sediment Load (metric tons/d) - 96 Micron	21,816	1,230	5.64
Sediment Load (metric tons/d) - 125 Micron	34,437	1,637	4.75
Sediment Load (metric tons/d) - 250 Micron	23,460	218	0.93
Total 63 - 250 Micron Load (metric tond/d)	90,554	3,748	4.14

During the simulation of the Modified Alignment design, it was observed that the earthen dike at the intake of the diversion created a large eddy in the main stem of the river and a sediment buildup on the downstream side of the diversion. To avoid this problem, the dike was removed, and a simulation was performed without the dike in place. This simulation was labeled "ND" for "No Dike" (ND-RM60.7-45K). Table 6-4 provides a summary of the sediment load captured in the diversion channel without the dike. Removing the dike slightly lowered the water discharge to 4.73% of the total river discharge. The dike created a stagnation point and built up the water stage at its upstream face, creating a favorable head-differential and improving the total water discharge in the diversion channel. However, this advantage is not significant as can be seen by comparing the with- and without-dike simulations, 4.82% and 4.73% respectively.

Table 6-4: Sediment Load (ND-RM60.7-45K)

	Mississippi River (Main Stem)	Diversion Channel ND-RM60.7-45K	Ratio
Water Discharge (m3/s)	19,821	937	4.73
Water Discharge (CFS)	700,000	33,075	4.73
Sediment Load (metric tons/d) - 32 Micron	233,539	13,819	5.92
Sediment Load (metric tons/d) - 63 Micron	10,839	619	5.71
Sediment Load (metric tons/d) - 96 Micron	21,816	1,150	5.27
Sediment Load (metric tons/d) - 125 Micron	34,437	1,675	4.86
Sediment Load (metric tons/d) - 250 Micron	23,460	528	2.25
Total 63 - 250 Micron Load (metric tond/d)	90,554	3,972	4.39

The ND-RM60.2-45K alignment extracted slightly lower silt load; 5.92% of the main river silt load compared to 6.55% obtained with the MA-RM60.2-45K. However, the ND-RM60.2-45K alignment improved the sand load (range of 63 – 250 microns) extracted in the diversion channel compared to the MA-RM60.2-45K alignment, namely 4.39% and 4.14%, respectively. For this design the SWR is 0.93.

Overall, the dike does not appear to be a critical component of the design. In fact, it is detrimental for some size classes. Hence, it is recommended to remove the dike, given the negative impacts it has on the flow field in the river and the lower capture of larger sediment grain sizes.

6.5.3 Additional Runs

Two additional simulations with design diversion discharges of 15,000 and 75,000 cfs using the modified alignment and no dike (ND-RM60.7-15K, and ND-RM60.7-75K) were performed using the same flow conditions of all the other diversion scenarios. As such, direct comparisons can be made among all the diversion scenarios. Table 6-5 and Table 6-6 show the results of these two simulations. The results clearly show that the performance of the modified alignment is better than the original alignment.

The ND-RM60.7-15K diversion delivered approximately 150% silt load compared to the OA-RM60.2-15K diversion. The level of improvement increased for the larger sediment size classes. The SWR for the 15,000 cfs diversion improved from 0.26 for the original alignment to 0.60 for the modified alignment. However, it should be noted that even with the modified alignment, the SWR is low. To minimize shoaling and other undesirable impacts, a SWR value of at least 1 is needed. The results indicate that the larger the amount of water diverted, the higher the value of SWR. The 75K diversion extracted 9.77% of sand the river while only extracting 8.7% of the river water, i.e. SWR of 1.12. This positive ratio (higher than 1) is encouraging as it implies little potential problems with shoaling in the river as a results of the diversion.

Table 6-5: Sediment Load (ND-RM60.7-15K)

	Mississippi River (Main Stem)	Diversion Channel ND-RM60.7-15K	Ratio
Water Discharge (m3/s)	19,821	361	1.82
Water Discharge (CFS)	700,000	12,733	1.82
Sediment Load (metric tons/d) - 32 Micron	233,539	4,189	1.79
Sediment Load (metric tons/d) - 63 Micron	10,839	188	1.74
Sediment Load (metric tons/d) - 96 Micron	21,816	335	1.53
Sediment Load (metric tons/d) - 125 Micron	34,437	420	1.22
Sediment Load (metric tons/d) - 250 Micron	23,460	44	0.19
Total 63 - 250 Micron Load (metric tond/d)	90,554	987	1.09

Table 6-6: Sediment Load (ND-RM60.7-75K)

	Mississippi River (Main Stem)	Diversion Channel ND-RM60.7-75K	Ratio
Water Discharge (m3/s)	19,821	1,725	8.70
Water Discharge (CFS)	700,000	60,918	8.70
Sediment Load (metric tons/d) - 32 Micron	233,539	24,789	10.61
Sediment Load (metric tons/d) - 63 Micron	10,839	1,156	10.67
Sediment Load (metric tons/d) - 96 Micron	21,816	2,357	10.81
Sediment Load (metric tons/d) - 125 Micron	34,437	3,726	10.82
Sediment Load (metric tons/d) - 250 Micron	23,460	1,607	6.85
Total 63 - 250 Micron Load (metric tond/d)	90,554	8,847	9.77

6.6 Conclusions

A three-dimensional numerical model was developed for the Myrtle Grove sediment diversion project. The model was calibrated and validated against detailed field measurements of velocity and sediment concentrations.

The FLOW-3D suite of models has the adequate physical processes and was selected for this study. The model results compared well with the field observations. The complex flow features observed in the field, such as the strong eddy at the Myrtle Grove bend and the complex sediment concentration vertical profiles, were reproduced by the model.

The model was used to assess the performance of various diversion alignments and sizes. Two alignments were tested. The Original Alignment did not perform well in terms of the water-to-sediment ratio in the diversion channel compared to in the main stem of the Mississippi River. The performance of this alignment was impacted by the location of the intake and the orientation of the alignment (adverse angle to the flow direction in the river). The intake is located on the outside of the Myrtle Grove bend (right descending bank), where the sand material has already started to migrate from the right to the left descending bank. This alignment design also contained two bends that caused energy losses, reducing the water and sediment carrying capacity of the design. A summary of the diversion scenarios is shown in Table 6-7.

Table 6-7: Sediment Load Summary

	Mississippi River (Main Stem)	Diversion Channel OA-RM60.2-15K	Diversion Channel ND-RM60.7-15K	Diversion Channel MA-RM60.7-45K	Diversion Channel ND-RM60.7-45K	Diversion Channel ND-RM60.7-75K
Water Discharge (m3/s)	19,821	322	361	955	937	1,725
Water Discharge (CFS)	700,000	11,369	12,733	33,735	33,075	60,918
Sediment Load (metric tons/d) - 32 Micron	233,539	2,786	4,189	15,306	13,819	24,789
Sediment Load (metric tons/d) - 63 Micron	10,839	104	188	663	619	1,156
Sediment Load (metric tons/d) - 96 Micron	21,816	144	335	1,230	1,150	2,357
Sediment Load (metric tons/d) - 125 Micron	34,437	133	420	1,637	1,675	3,726
Sediment Load (metric tons/d) - 250 Micron	23,460	2	44	218	528	1,607
Total 63 - 250 Micron Load (metric tond/d)	90,554	383	987	3,748	3,972	8,847
Sediment/Water Ration (SWR)		0.26	0.60	0.85	0.93	1.12

The Modified Alignment moved the diversion channel intake to RM60.7. This allowed for capturing the sand material coming off the sand bar before it starts the migration towards the left descending bank. Further, the Modified Alignment had a favorable angle relative to the flow direction in the river. Hence, it produced a much more favorable water-to-sediment ratio. As seen in Table 6-7, for the 15,000 cfs diversion, the modified alignment improved the SWR to 0.60 compared to the 0.26 for the original alignment. Further, it can be seen that the larger the diversion size, the higher the SWR. Comparing the 15,000 cfs, 45,000 cfs, and 75,000 cfs diversions for the modified alignment (all without the guide dike), the SWR improved from 0.60 to 0.93 to 1.12, respectively.

It was observed that the dike component of the Modified Alignment design had an adverse effect on the flow field in the river (large eddy and potential sediment buildup on the downstream side). Thus, the dike was removed and the performance of the diversion was tested without it (run ND-RM60.7-45K). The results indicate that the dike does not enhance the performance of the diversion significantly. In fact, it is detrimental for some of the larger sediment size classes. As seen in Table 6-7, removing the dike improved the SWR from 0.85 to 0.93. Therefore, it is recommended to remove the dike from the design, considering the advantage of avoiding problems in the river and reduction in project cost due to removing a component

Additional intake designs should be considered to improve the SWR. Since the upper layers of the water column carry the least amount of sand, it is recommended that an orifice-like entrance be tested. Such a design will draw water from the deeper and more sand rich layers. Significant improvement to the SWR will be cost effective if it successfully removes (or at least minimizes) any shoaling and navigation concerns.

It should be noted that the analysis performed thus far focused on optimizing the design of the intake structure in terms of size and location. Further three-dimensional and one-dimensional analyses are required to address the long-term and large-scale potential shoaling problems that may result from constructing the diversion.

7 BASIN-SIDE MORPHOLOGICAL MODELING

7.1 Introduction

The Delft3D modeling system was used by M&N to obtain a general understanding of the potential land-building capacity within the Barataria Basin of the Myrtle Grove Sediment Diversion. Appendix F contains a complete report discussing this effort. The Mississippi River 3D Model developed by others and discussed in Chapter 6 provided sediment loadings through the diversion under different discharge conditions and was used as inputs for the Delft3D model. The Delft system was then applied to model the sediment transport and morphological changes due to the sediment diversion over a period of 45 years. This allows for a better informed decision as to whether enough sediment can be diverted at Myrtle Grove and delivered a significant distance into the Barataria Basin to build land; or whether the primary benefit of this diversion will remain the introduction of freshwater into the basin.

7.2 Model Mesh Development

Delft3D uses curvilinear, structured boundary fitted grids. Due to the complex geometry of the Barataria Basin, it is impractical to use just one fine resolution grid for this land building modeling purpose. Domain decomposition is a technique implemented in Delft3D to divide a model domain into several smaller sub-domains. The subdivision is based on the horizontal and vertical model resolution required for adequately simulating physical processes. Then the computations can be carried out separately on these sub-domains. The communication between the sub-domains takes place along internal open boundaries or so-called dd-boundaries.

Figure 7-1 shows the model grids developed for this project. There are a total of 12 sub-domains. C2 represents the Barataria Waterway up to the intersection with the GIWW, which is represented by the C4 grid. I1 is the transition link between C2, C4 and C5 (extension from the GIWW after the intersection with the Barataria Waterway). O1 is the offshore sub-domain where water level boundaries are applied. M1 is the region north of the GIWW, and M2 is the region west of the Barataria Waterway. Five sub-grids represent the area east of the Barataria Waterway. M62a and M62b are two very fine resolution grids representing the areas adjacent to the Myrtle Grove diversion in order to have detailed sediment transport and morphological change patterns. The other three sub-domains are M5, M61 and M63. The Delft model bathymetry is shown in Figure 7-2.

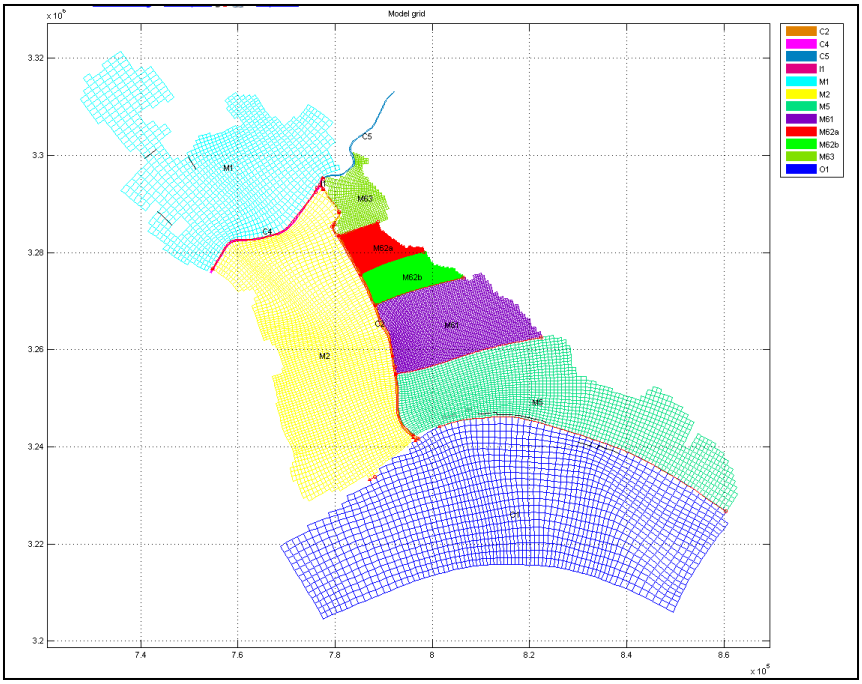

Figure 7-1: Delft3D Model Grid

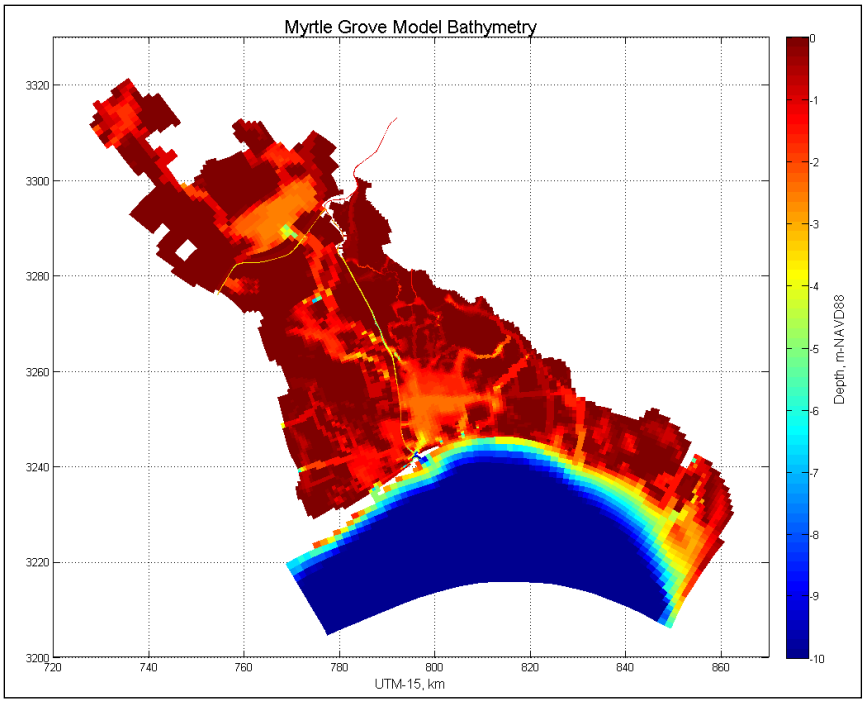

Figure 7-2: Delft3D Model Bathymetry

7.3 Hydrodynamic Model Calibration

As stated previously, the flow module provides the hydrodynamic basis for other modules such as water quality, ecology, waves and morphology. Therefore, it is important to ensure that the hydrodynamics of the basin can be accurately represented by the developed model. The model was calibrated against measured data (see Appendix F).

7.4 Sediment Transport and Morphological Modeling

The calibrated hydrodynamic model was on-line coupled with the sediment transport and morphology modules to quantitatively simulate the fate of the diverted sediments from the Myrtle Grove diversion.

7.5 Alternative Scenarios

The original USACE channel alignment for the Myrtle Grove diversion consisted of a channel with two bends and having the inflow structure located at River Mile 60.2 AHP. The project team, though, identified alternative alignments that might better capture sediment from the Mississippi River. The first, designated as the "Modified USACE" alignment consisted of a channel with only one bend and having the inflow structure located at River Mile 60.7 AHP. This alignment enters the Barataria Basin at the same location in the back levee as the original USACE channel.

The second alternative, designated "new alignment," also had its inflow structure located at River Mile 60.7 AHP but consisted of a straight channel that entered the Basin slightly south of the other alternatives. The team's opinion was that this new alignment would best capture sediment from the River. (Note – this new alignment is referred to as the "modified" alignment in Chapter 6).

In order to "bracket" the possible results, two alternative scenarios were selected to be modeled. They were the original USACE channel alignment with a maximum 15,000 cfs diversion flow and the new alignment with a maximum 45,000 cfs diversion flow.

A 3-dimensional model of the River, diversion structure and outfall channel was developed by others and run for these two alternatives. A river flow of 700,000 cfs was assumed since this matched with that which occurred when the sediment data collection efforts were undertaken. Based on the conceptual diversion structure, this resulted in an actual peak diversion of 11,369 cfs for the first alternative and 33,735 cfs for the second one. Table 7-1 and Table 7-2 present the sediment load in the two diversion alternatives determined by this numerical modeling done by others.

Table 7-1: Sediment Load in USACE Diversion Alignment (15,000 cfs)

Central Size in Microns	<=2	32	63	96	125	250	>250
Size Range in Microns	<=2	2 -63	63-79	79-113	113-187	187-250	>250
Mean Fractional Sediment Load in River (metric tons / day)	25,805	233,539	10,839	21,816	34,437	23,460	6,564
Water Flow in Diversion (%)	1.62 (River = 700,000 cfs, Diversion = 11,369 cfs)						
Sediment Load in Diversion / River (%)	1.62	1.19	0.96	0.66	0.39	0.01	n/a
Sediment Concentration / River (%)	100	73.64	59.48	40.82	23.76	0.53	n/a
Sediment Load in Diversion (metric tons / day)	418	2,786	104	144	133	2	n/a

Table 7-2: Sediment Load in New Diversion Alignment (45,000 cfs)

Central Size in Microns	<=2	32	63	96	125	250	>250
Size Range in Microns	<=2	2 -63	63-79	79-113	113-187	187-250	>250
Mean Fractional Sediment Load in River (metric tons / day)	25,805	233,539	10,839	21,816	34,437	23,460	6,564
Water Flow in Diversion (%)	4.825 (River = 700,000 cfs, Diversion = 33,735 cfs)						
Sediment Load in Diversion / River (%)	n/a	6.554	6.117	5.639	4.754	0.929	n/a
Sediment Concentration / River (%)	n/a	135.83	126.78	116.87	98.53	19.25	n/a
Sediment Load in Diversion (metric tons / day)	n/a	15,306	663	1,230	1,637	218	n/a

For the DELFT numerical modeling of the land-building potential in the Basin, three sediment sizes were modeled; 32, 63 and 96 microns. The sediment load for the 125 microns sand fraction was added to the load for the 96 micron sand while the clay particles (<2 microns) and coarse sand fractions (>=250 microns) were not included in this analysis.

The assumed hydrograph for the diversion flows consisted of a peak diversion flow for the month of April with all three sediment sizes. For the remaining months, only silt sediments (32 microns) were assumed to be carried with the same concentration as in the peak flow. Table 7-3 and Table 7-4 present the monthly diversion flows, sediment concentrations and sediment loads for the two alternatives modeled.

Table 7-3: Diversion and Sediment Parameters for USACE Alignment (15,000 cfs)

Month	Diversion Flow (cfs)	Sediment Concentration (kg/m^3)			Sediment Load (metric tons/day)		
		32 µm	63 µm	96 µm	32 µm	63 µm	96 µm
January	5,000	0.1	-	-	1,223	-	-
February	5,000	0.1	-	-	1,223	-	-
March	9,000	0.1	-	-	2,202	-	-
April	11,400	0.1	0.0037	0.01	2,789	103	279
May	9,000	0.1	-	-	2,202	-	-
June	5,000	0.1	-	-	1,223	-	-
July	5,000	0.1	-	-	1,223	-	-
August	5,000	0.1	-	-	1,223	-	-
September	5,000	0.1	-	-	1,223	-	-
October	5,000	0.1	-	-	1,223	-	-
November	5,000	0.1	-	-	1,223	-	-
December	5,000	0.1	-	-	1,223	-	-
Total Load: 565,615 metric tons (462,373 cubic yards[1])							

1. Assuming the dry sediment density is 1600 kg/m^3.

Table 7-4: Diversion and Sediment Parameters for New Alignment (45,000 cfs)

Month	Diversion Flow (cfs)	Sediment Concentration (kg/m^3)			Sediment Load (metric tons/day)		
		32 µm	63 µm	96 µm	32 µm	63 µm	96 µm
January	10,000	0.186	-	-	4,551	-	-
February	10,000	0.186	-	-	4,551	-	-
March	18,000	0.186	-	-	8,191	-	-
April	33,735	0.186	0.008	0.0347	15,306	663	2,867
May	18,000	0.186	-	-	8,191	-	-
June	10,000	0.186	-	-	4,551	-	-
July	10,000	0.186	-	-	4,551	-	-
August	10,000	0.186	-	-	4,551	-	-
September	10,000	0.186	-	-	4,551	-	-
October	10,000	0.186	-	-	4,551	-	-
November	10,000	0.186	-	-	4,551	-	-
December	10,000	0.186	-	-	4,551	-	-
Total Load: 2,316,545 metric tons (1,893,704 cubic yards[1])							

1. Assuming the dry sediment density is 1600 kg/m^3.

7.6 Consolidation of Underlying Sediments

An important issue that had to be considered was the consolidation of the existing soils in the Barataria Basin due to the weight of newly diverted sediments. Data for the BA-39 and BA-48 Bayou Dupont Marsh Restoration Projects was provided by CPRA. This data was developed by compiling marsh fill settlement estimates for several borings taken for the geotechnical evaluations completed for these projects. It was clear from this data that the underlying material will consolidate significantly due to the weak organic soils near the surface. Where sandy soils are also present, they provide drainage paths for the water thereby increasing the consolidation rate.

An analysis of this data indicated an approximate time varying rate of consolidation as a percentage of the deposition of new material. It was thus assumed that the consolidation would be equal to 19% of the depth of deposition per year in Year 1; 4.75% of the initial depth of deposition per year in Year 2; 1.75% per year in Years 3-5; 0.65% per year in Years 6-10; 0.35% per year in Years 11-15; 0.25% per year in Years 16-25 and 0.1% per year thereafter.

7.7 Results

7.7.1 Without Consolidation

Initially, two runs were performed without considering consolidation of the underlying soils. It is noteworthy that these modeling results also do not include the effects of subsidence, sea level rise or the adaptation of the marsh to such effects. It is also important to note that these modeling results do not account for the decrease in efficiency of the diversion channel as the land on the basin end of the channel builds up; thereby reducing the potential head differential between the basin and the Mississippi River. Maintenance of this area, consisting of excavation / dredging, will be required to achieve the total potential volume of deposition shown in these results. Hence the largest magnitudes of deposition depth and bed elevations shown in these figures will not be reached as this material would be removed and placed elsewhere. The results from these model runs are included in Appendix F.

7.7.2 With Consolidation

Next, the same two runs were performed but accounting for consolidation of the underlying soils. Again, it is noteworthy that these modeling results do not include the effects of subsidence, sea level rise or the adaptation of the marsh to such effects. Also, as previously noted, they do not account for the decrease in efficiency of the diversion channel over time.

Figure 7-3 presents the initial bed elevation assumed in the model. Figure 7-4 presents the modeling results for the total depth of material deposited after 45 years respectively for the 15,000 cfs USACE alignment alternative with consideration of underlying soil consolidation. Figure 7-5 presents the modeling results for the resulting bed elevation after 45 years. Table 7-5 presents the deposition volume for each 0.5 ft deposition depth interval after 45 years.

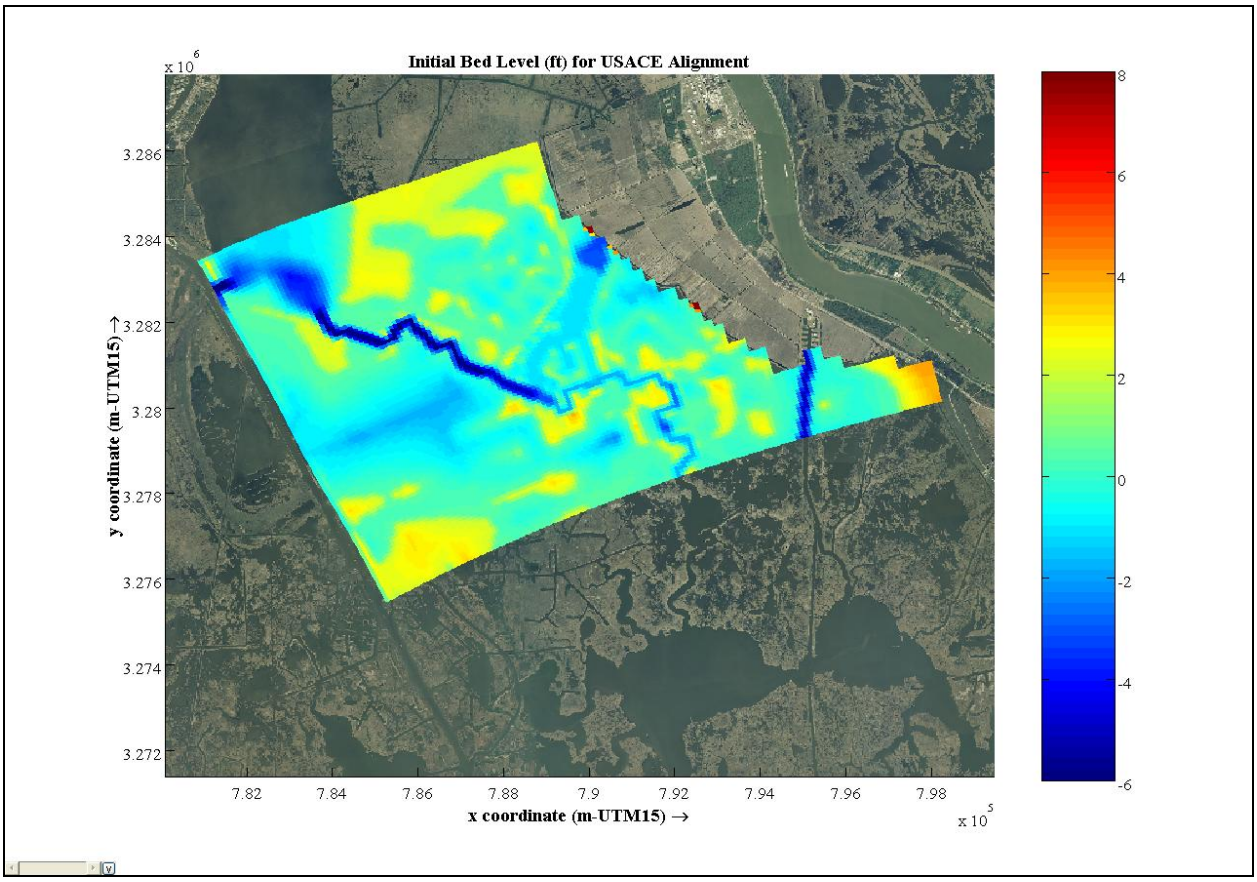

Figure 7-3: Initial Bed Level (ft)

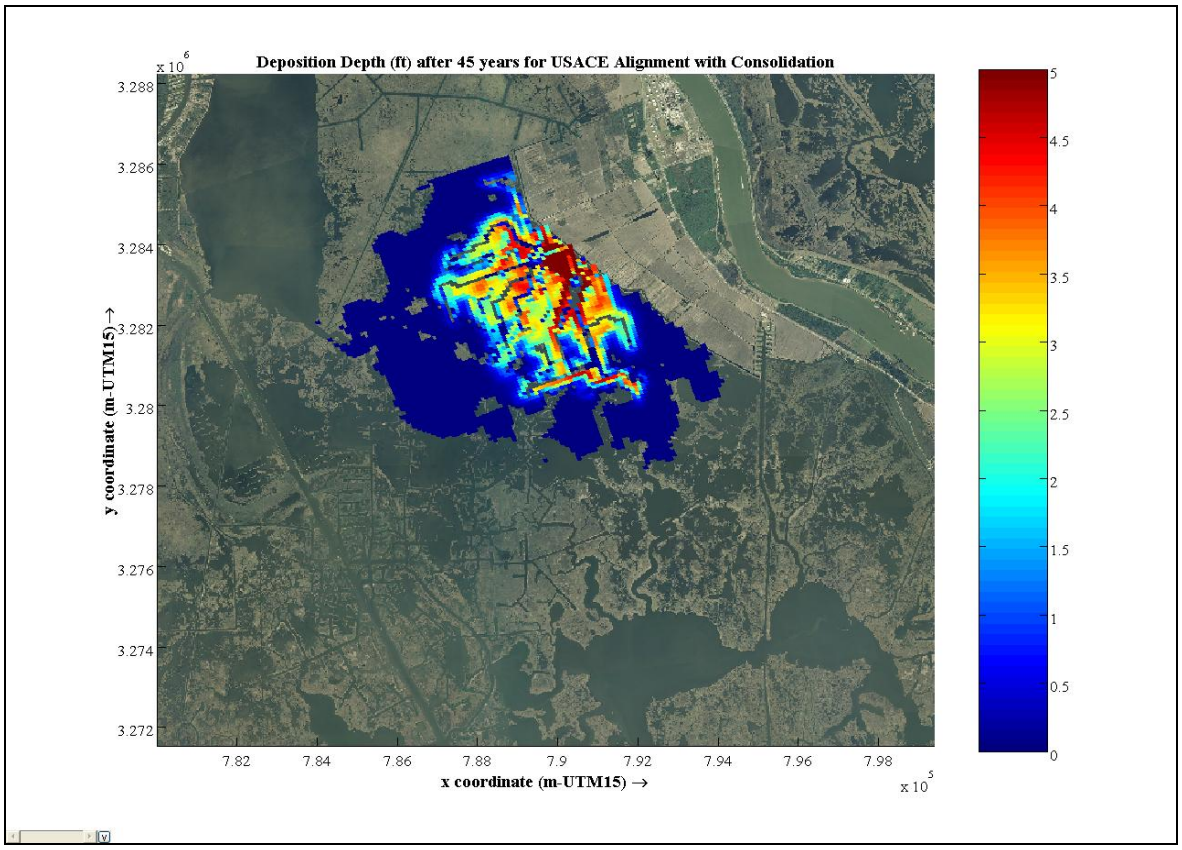

Figure 7-4: Deposition Depth (ft) after 45 years for USACE Alignment (15,000 cfs) with Consolidation

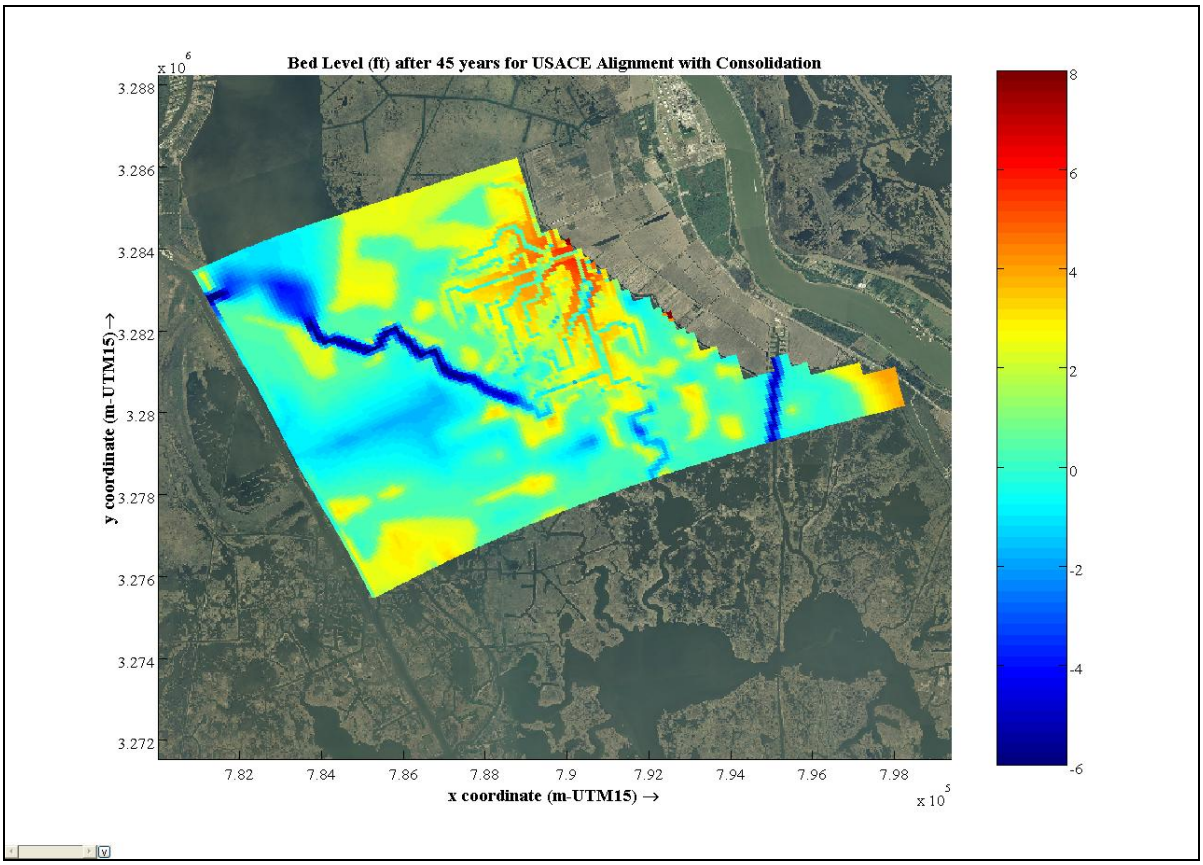

Figure 7-5: Bed Level (ft) after 45 years for USACE Alignment (15,000 cfs) with Consolidation

Table 7-5: Deposition Results after 45 years for USACE Alignment (15,000 cfs) with Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	3,327,969
0.5 – 1.0 ft	2,653,784
1.0 – 1.5 ft	2,163,814
1.5 – 2.0 ft	1,712,050
2.0 – 2.5 ft	1,318,932
2.5 – 3.0 ft	932,771
3.0 – 3.5 ft	582,574
3.5 – 4.0 ft	301,876
4.0 – 4.5 ft	168,732
4.5 – 5.0 ft	108,870
> 5.0ft	278,055
Total	13,549,428

Figure 7-6 presents the modeling results for the total depth of material deposited after 45 years for the 45,000 cfs new alignment alternative with consideration of underlying soil consolidation. Figure 7-7 presents the modeling results for the resulting bed elevation after 45 years.

Table 7-6 presents the deposition volume for each 0.5 ft deposition depth interval after 45 years.

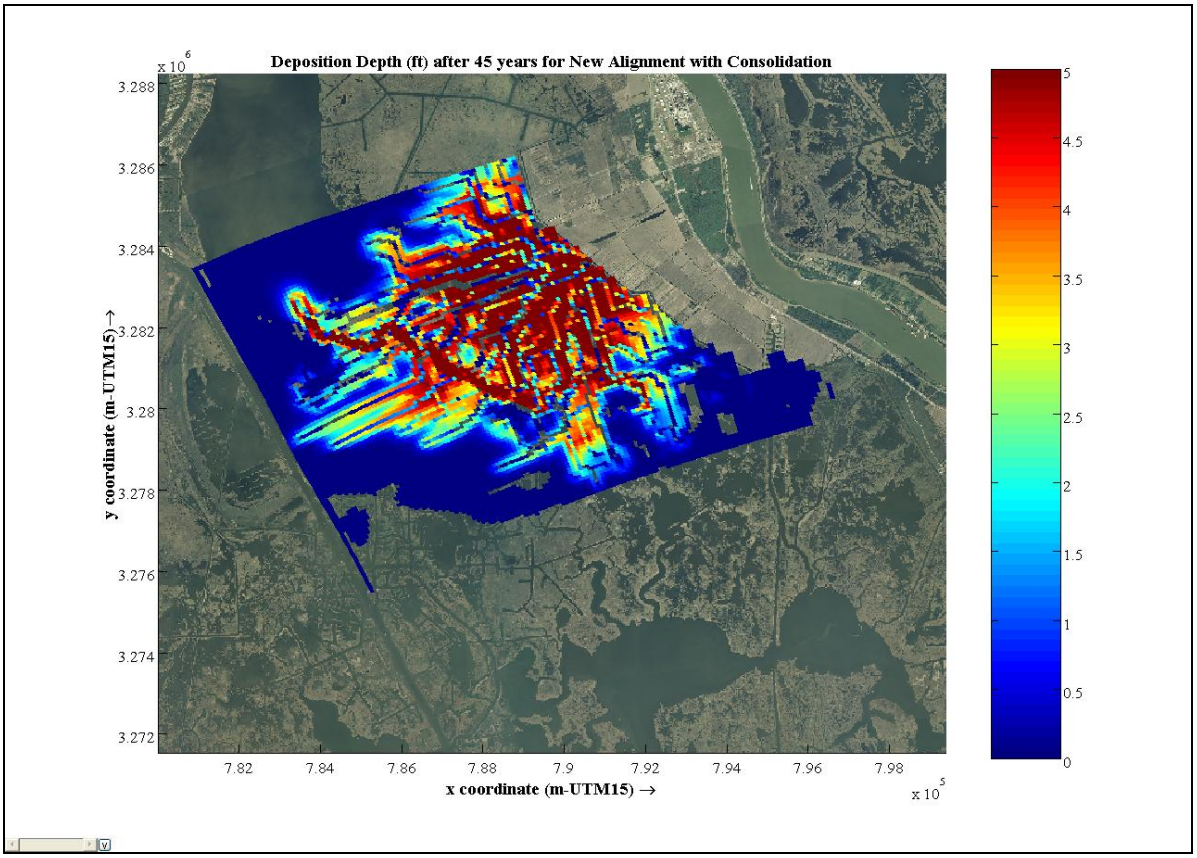

Figure 7-6: Deposition Depth (ft) after 45 years for New Alignment (45,000 cfs) with Consolidation

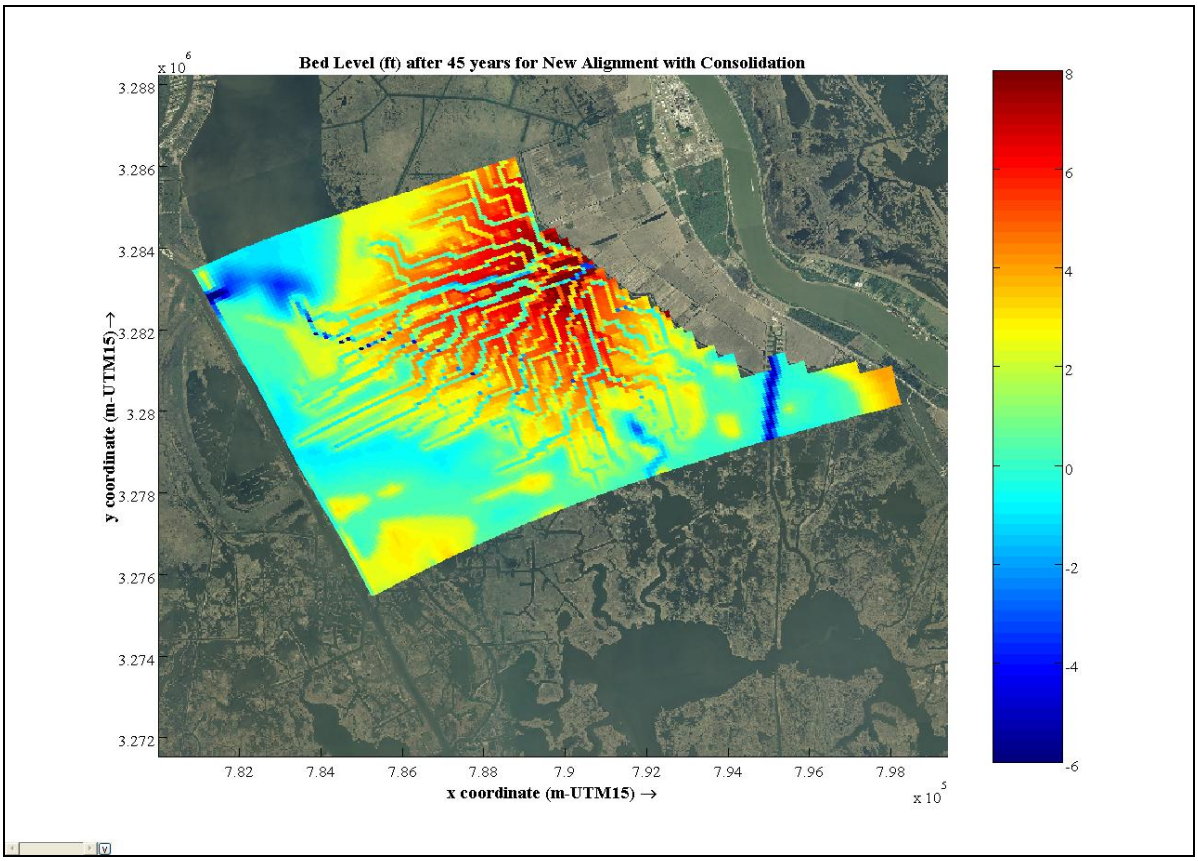

Figure 7-7: Bed Level (ft) after 45 years for New Alignment (45,000 cfs) with Consolidation

Table 7-6: Deposition Results after 45 years for New Alignment (45,000 cfs) with Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	9,845,054
0.5 – 1.0 ft	8,269,320
1.0 – 1.5 ft	7,068,785
1.5 – 2.0 ft	6,051,881
2.0 – 2.5 ft	5,166,077
2.5 – 3.0 ft	4,239,346
3.0 – 3.5 ft	3,414,589
3.5 – 4.0 ft	2,743,833
4.0 – 4.5 ft	2,202,083
4.5 – 5.0 ft	1,663,561
> 5.0ft	3,877,226
Total	54,541,756

7.8 Conclusions

Table 7-7 presents a summary of the total depositional volumes after 45 years for the two alternatives for both the without and with consolidation scenarios.

Table 7-7: Summary of Deposition Results after 45 years

Alignment	Discharge	Consolidation	Deposition
USACE	15,000 cfs	No	~20.5 million cy
USACE	15,000 cfs	Yes	~13.5 million cy
New	45,000 cfs	No	~79.4 million cy
New	45,000 cfs	Yes	~54.5 million cy

It indicates that the new alignment is much more efficient than the original one. After accounting for consolidation, while the discharge is increased by 3 times, the volume deposited increases by over 4 times.

Additionally, the graphical results indicate that, over 45 years, a classic "delta" will form and the bed level will be increased significantly within the area up to about 3 miles from where the discharge channel enters the basin. However, the effects further away quickly diminish with little impact observed at the Barataria Waterway.

This model, though, could not be "calibrated" to a similar project in this area, so the spatial results are mainly qualitative, but the total volumes are accurate as they are based on the results of the riverine modeling. Furthermore, it is evident that significant deposition of the coarser materials will occur at the channel / basin interface and higher resolution modeling (both numerical and physical) of this area will be required to determine how best to manage this area to ensure the operational efficiency of the diversion.

8 SUMMARY & CONCLUSIONS

8.1 Small Scale Physical Modeling

Coastal Restoration Consultants (CRC) and the LSU Department of Civil & Environmental Engineering (CEE) conducted physical model experiments at the small-scale physical model (SSPM). The specific objectives of the SSPM experiments were: (1) to investigate the effectiveness of a sediment diversion at the authorized Myrtle Grove location; and (2) identify other potential locations for sediment diversions within a reasonable distance from the authorized site. In addition, the study looked at the relative effectiveness of two different diversion designs.

Annual sediment (sand) diversion efficiencies were calculated based on weights of sediments collected at Myrtle Grove (RM 59) for deep (50 year) and shallow (100 year) tests, the combined Magnolia (RM 47.5) and Woodland (RM 51) location for shallow tests, the Magnolia (RM 47.5) deep test and the 50 year Woodland shallow test. The comparative efficiency of the sediment diversions at Magnolia (RM 47.5) and Woodland (RM 51) locations in relation to Myrtle Grove (RM 59) is 5.7 times greater. The comparative efficiency of the sediment diversion at the 50 year Woodland (RM 51) location in relation to Myrtle Grove (RM 59) is 4.3 times greater. In addition, comparison of the Magnolia (RM 47.5) deep test to the Myrtle Grove (RM 59) deep test showed that the Magnolia location was 11.3 times more efficient. These values confirm the original hypothesis that the Myrtle Grove location (at least the location tested in the SSPM) would not be conducive for efficient delivery of sediment into the diversion structure.

An additional comparison was made between the deep and shallow diversions. The efficiency of the Magnolia (RM 47.5) shallow relative to the Magnolia (RM 47.5) deep is 2.7 times greater and the Myrtle Grove (RM 59) shallow relative to the Myrtle Grove (RM 59) deep is 5.3 times greater. Given the limitations of the SSPM to replicate complex 3D flow patterns, though, these results showing that the shallow diversions are more efficient than the deep ones should be viewed as semi-quantitative. Additional larger scale tests would be required to confirm and better understand the results.

Furthermore, for identical structural arrangements, the SSPM demonstrated that it is possible to divert significant amounts of more sediment by adequate site selection of the diversion structure. Pre-study hypotheses concerning the impact of river channel geometry combined with analysis of the bathymetric survey charts and knowledge of riverine hydrodynamics and sediment transport processes were confirmed. The two sites located downstream of convex river bends near lateral bars, Magnolia (RM 47.5) and Woodland (RM 51), both performed much better than the Myrtle Grove (RM 59) site located downstream of a deep, concave river bend. While not

sufficient for site design work, the SSPM results showed that the shallow, wide diversion structures, proposed by CRC, outperformed the deep diversion structures. Note, however, that additional larger-scale physical modeling, analysis of field data and high-resolution 3D numerical modeling should be performed to guide detailed design.

8.2 Conceptual Structure Design

BCG Engineering and Consulting, Inc. developed conceptual designs of the diversion structure. These concepts incorporated the alternative diversion locations and channel alignments identified by the project team. A preliminary design flow hydrograph for the Mississippi River at Myrtle was developed as well as a rating curve based on an average of points plotting historical river discharges and stages.

Two alternative diversion locations and channel alignments were considered. The first, at RM 60.2 AHP is based on the original USACE concept. The second "modified" alignment, at RM 60.7 was selected based on the results of the physical model tests and basic hydraulic analyses of the River which indicated that it would more efficiently capture and transport sediment from the River to the Barataria Basin

Basic features of the design of the diversion structure include five individual gates which will be 30 feet wide by 10 feet high segments that can be dogged at any desired gate opening to facilitate diversion choices to enhance sediment diversion or freshwater mitigation. River works in the shape of a 1,200 foot deflection dike and free standing sheet pile wall are also included to enhance water and sediment entry into the diversion channel. Also, the structure will have a single gate lifting crane to operate the gates.

The depth of the point bar, -40 feet at RM 60.2 AHP, governs the shape of the diversion channel and the wetted perimeter. Based on the average annual peak flow a total depth of 44 feet over the gate sill dictates the most efficient cross-section and hence the width would be 2 x 44 feet or 88 feet. These dimensions, 44 feet x 88 feet, will provide more than twice the diversion flow being used (15,000 cfs) as the target flow for modular design. However this size channel could be used as a modular target to increase the design flow to 70,000 cfs. Therefore, the basic design for the diversion channel is a 44 feet x 30 feet concrete flume.

This project location will sever a portion of the drainage system currently in existence on Conoco Phillips Company lands and measures will need to be taken to render the drainage system whole again. Furthermore, there are public utilities, water forced mains, a natural gas line and several commercial power lines that will require major relocations and modifications to span across the project channel. In addition, an elevated four-lane bridge and access ramps will be required to accommodate area traffic on State Highway 23.

Federal levees and local back levees exist at the Mississippi River bank and at the National Food and Beverage Company western boundary. This will require that adjacent levees, equal in design criteria to Federal Mississippi River and Tributaries Project Levees, be provided on each bank of the diversion channel, since these levees will contain Mississippi River waters.

The probable construction cost for the 15,000 cfs Myrtle Grove Freshwater and Sediment Diversion Project as located by the USACE at RM 60.2 AHP is approximately $180 million. It should be noted that this estimate does not include any land acquisition costs or engineering and design fees, or any costs for supervision and administration during construction. If the project is enlarged to allow for a 45,000 cfs diversion, then the cost estimate would be approximately $330 million.

8.3 Basin Side Hydrodynamic & Salinity Modeling

Moffatt & Nichol developed and completed an estuarine hydrodynamic model for evaluating the Barataria Basin effects due to operating both the Myrtle Grove and Davis Pond diversions at varying and increasing flow rates. A total of seventeen one-year model runs for various combinations of diversions were performed.

The following conclusions were drawn after analyses and review of the alternative modeling results:

- The impacts on salinity levels in the Barataria Basin from the Myrtle Grove project depend on the diversion regimes at Davis Pond. The effects of the Myrtle Grove project are reduced under higher Davis Pond diversion scenarios.
- The Myrtle Grove project under low diversion has negligible impact on salinity levels in the Barataria Basin regardless of the Davis Pond Diversion operational level.
- High Myrtle Grove diversions could reduce annual average salinity levels over 6 ppt depending upon the magnitude of diversions at Davis Pond while medium Myrtle Grove diversions would only reduce the annual average salinity by less than 4 ppt.
- The high Myrtle Grove diversion scenario would push the annual 5 ppt and 15 ppt salinity level contours twice as far southward as the medium Myrtle Grove diversion case; regardless of the magnitude of the different Davis Pond diversion.
- On a semi-annual (December – May) basis, the highest Myrtle Grove diversion scenarios push the 15 ppt salinity line to near the backside of the barrier islands except in the immediate vicinity of the passes, and the far eastern section of the basin.

This work was built upon to pre-screen Myrtle Grove diversion flows in order to determine feasible diversion flows for a sediment diversion. The model was used to evaluate maximum diversion magnitudes of 15,000 cfs; 45,000 cfs; 75,000 cfs; 150,000 cfs; 240,000 cfs and 300,000 cfs on water level and velocity increases in the basin. The purpose of these runs was to establish the upper limit of diversion magnitudes for further modeling by identifying the flows with significant, insurmountable impacts. Wind and rain were not evaluated in these runs.

The diversion was assumed to be operated from February to July. It was evident that the highest diversions could cause significant flooding problems and severe currents in the regions adjacent to the diversions. On average, near the Town of Lafitte in April when the diversion reaches its peak flow, the 45,000 cfs and 75,000 cfs diversion flows could increase water levels by 1 and 2 ft, respectively; but for the 150,000 cfs and 300,000 cfs diversions, the peak water levels could increase by 3 and 5 ft, respectively. The increased velocity with the larger diversions could cause local scour and damage the existing marshes as well as creating a potential safety hazard.

Pulsed operation of controlled diversions offers a possible strategy for maintaining a wide range of salinity conditions while at the same time delivering river sediments into a wetland region. Additionally, the duration of elevated water levels and current velocities might be mitigated if the system responds fairly quickly. Therefore, the hydrodynamic effects of three pulsing diversions were investigated. The maximum diversion magnitudes were 15,000 cfs; 45,000 cfs and 75,000 cfs and the pulsing interval was set at two weeks with the diversion operating and two weeks with a minimum diversion flow of 5,000 cfs. Review of these results clearly indicates that the system reacts fairly quickly to the diverted flows with water levels and currents rising rapidly upon initiation of the diversion and then decreasing rapidly once the diverted flows are reduced.

Finally, to provide boundary conditions for the diversion structure and channel modeling efforts, hydrodynamic model runs were conducted to estimate the basin stage as a function of the diverted flow at Myrtle Grove. For each diversion flow condition, four day periods were run to obtain “steady-state” conditions, and the water levels at the discharge point were extracted from the results.

8.4 Data Collection

An intensive data collection effort was lead by Mead A. Allison, PhD. These efforts included bathymetric surveys to provide a basemap for numerical modeling of the river and to assist in the determination of bedload transport rates. River discharges were measured and suspended and bottom sediment samples collected. Profile data through the water column was collected using CTD (conductivity / temperature / depth) and LISST gages. This data included depth, backscatter, pH, oxygen, beam transmission, conductivity, salinity, temperature and suspended grain sizes.

8.5 River Side Hydrodynamic Modeling

Three-dimensional numerical modeling of the river was performed by Ehab Meselhe, PhD, PE; Ioannis Georgiou, PhD, PE; and John McCorquodale, PhD, PE. This modeling utilized the collected data for model setup and calibration / verification. It also incorporated the conceptual designs of the diversion structure and channel. The three-dimensional model was used to develop an understanding of the complex flow field near the Myrtle Grove river bend and examine the interaction between flow and sediment. The model domain extended from River Mile (RM) 56.0 (above the head of passes – AHP) to RM 62.7 AHP. This river reach encompasses the potential location for the sediment diversion site.

The model was used to assess the performance of various diversion alignments and sizes. Two alignments were tested. The Original Alignment did not perform well in terms of the water-to-sediment ratio in the diversion channel compared to in the main stem of the Mississippi River. The performance of this alignment was impacted by the location of the intake and the orientation of the alignment (adverse angle to the flow direction in the river). The intake is located on the outside of the Myrtle Grove bend (right descending bank), where the sand material has already started to migrate from the right to the left descending bank. This alignment design also contained two bends that caused energy losses, reducing the water and sediment carrying capacity of the design.

The Modified Alignment moved the diversion channel intake to RM60.7. This allowed for capturing the sand material coming off the sand bar before it starts the migration toward the left descending bank. Further, the Modified Alignment had a favorable angle relative to the flow direction in the river. Hence, it produced a much more favorable water-to-sediment ratio.

It was observed that the dike component of the Modified Alignment design had an adverse effect on the flow field in the river (large eddy and potential sediment buildup on the downstream side). Thus, the dike was removed and the performance of the diversion was tested without it. The results indicated that the dike does not enhance the performance of the diversion significantly. In

fact, it is detrimental for some of the sediment size classes. It is recommended to remove the dike from the design, considering the advantage of avoiding problems in the river and a reduction in project cost due to removing this component.

The results also show that the 75,000 cfs diversion had a favorable sediment/water ratio (9.77% of sand from the river and 8.7% of the river water). This favorable SWR ratio (higher than 1) is encouraging as it implies very little to no potential problems with shoaling in the river as a result of the diversion.

Additional intake designs should be considered to improve the SWR. Since the upper layers of the water column carry the least amount of sand, it is recommended that an orifice-like entrance be tested. Such design will draw water from the deeper and more sand rich layers Significant improvement to the SWR will be cost effective if it successfully removes (or at least minimizes) any shoaling and navigation concerns.

8.6 Basin Side Morphological Modeling

The Delft3D modeling system was used by M&N to obtain a general understanding of the potential land-building capacity within the Barataria Basin of the Myrtle Grove Sediment Diversion. The Mississippi River 3D Model developed by others provided sediment loadings through the diversion under different discharge conditions and was used as inputs for the Delft3D model. The Delft system was then applied to model the sediment transport and morphological changes due to the sediment diversion over a period of 45 years.

The original USACE channel alignment for the Myrtle Grove diversion consisted of a channel with two bends and having the inflow structure located at River Mile 60.2 AHP. The second alternative, designated “new alignment,” also had its inflow structure located at River Mile 60.7 AHP but consisted of a straight channel that entered the Basin slightly south of the other alternatives. The team’s opinion was that this new alignment would best capture sediment from the River. In order to “bracket” the possible results, two alternative scenarios were selected to be modeled. They were the original USACE channel alignment with a maximum 15,000 cfs diversion flow and the new alignment with a maximum 45,000 cfs diversion flow.

For the DELFT numerical modeling of the land-building potential in the Basin, three sediment sizes were modeled; 32, 63 and 96 microns. The sediment load for the 125 microns sand fraction was added to the load for the 96 micron sand while the clay particles (<2 microns) and coarse sand fractions (>=250 microns) were not included in this analysis.

The assumed hydrograph for the diversion flows consisted of a peak diversion flow for the month of April with all three sediment sizes. For the remaining months, only silt sediments (32 microns) were assumed to be carried with the same concentration as in the peak flow.

An important issue that had to be considered was the consolidation of the existing soils in the Barataria Basin due to the weight of newly diverted sediments. Data for the BA-39 and BA-48 Bayou Dupont Marsh Restoration Projects was provided by CPRA. This data was developed by compiling marsh fill settlement estimates for several borings taken for the geotechnical evaluations completed for these projects. It was clear from this data that the underlying material will consolidate significantly due to the weak organic soils near the surface. Where sandy soils are also present, they provide drainage paths for the water thereby increasing the consolidation rate.

An analysis of this data indicated an approximate time varying rate of consolidation as a percentage of the deposition of new material. Initially, two runs were performed without considering consolidation of the underlying soils. It is noteworthy that these modeling results also do not include the effects of subsidence, sea level rise or the adaptation of the marsh to such effects. It is also important to note that these modeling results do not account for the decrease in efficiency of the diversion channel as the land on the basin end of the channel builds up; thereby reducing the potential head differential between the basin and the Mississippi River. Maintenance of this area, consisting of excavation / dredging, will be required to achieve the total potential volume of deposition shown in these results. Hence the largest magnitudes of deposition depth and bed elevations shown in the results will not be reached as this material would be removed and placed elsewhere. Next, the same two runs were performed but accounting for consolidation of the underlying soils.

The results shown in Table 8-1 indicate that the new alignment is much more efficient than the original one. After accounting for consolidation, while the discharge is increased by 3 times, the volume deposited increases by over 4 times.

Table 8-1: Summary of Deposition Results after 45 years

Alignment	Discharge	Consolidation	Deposition
USACE	15,000 cfs	No	~20.5 million cy
USACE	15,000 cfs	Yes	~13.5 million cy
New	45,000 cfs	No	~79.4 million cy
New	45,000 cfs	Yes	~54.5 million cy

Additionally, the results indicate that, over 45 years, a classic “delta” will form and the bed level will be increased significantly within the area up to about 3 miles from where the discharge channel enters the basin. However, the effects further away quickly diminish with little impact observed at the Barataria Waterway.

This model, though, could not be “calibrated” to a similar project in this area, so the spatial results are mainly qualitative, but the total volumes are accurate as they are based on the results of the riverine modeling. Furthermore, it is evident that significant deposition of the coarser materials will occur at the channel / basin interface and higher resolution modeling (both numerical and physical) of this area will be required to determine how best to manage this area to ensure the operational efficiency of the diversion.

APPENDIX A

SMALL SCALE PHYSICAL MODELLING

PROGRESS REPORT ON SEDIMENT DIVERSION TESTING ON LOWER MISSISSIPPI RIVER

September 28, 2011

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Project Objective

Coastal Restoration Consultants (CRC) and the LSU Department of Civil & Environmental Engineering (CEE) were contracted by the Coastal Protection and Restoration Authority (CPRA) to conduct physical model experiments at the small-scale physical model (SSPM) in support of the Myrtle Grove Delta Building Diversion Modeling Effort. The specific objectives of the SSPM experiments were: (1) to investigate the effectiveness of a sediment diversion at the authorized Myrtle Grove location; and (2) identify other potential locations for sediment diversions within a reasonable distance from the authorized site. In addition, the study looked at the relative effectiveness of two different diversion designs.

Small-scale Physical Model Capabilities

The present small-scale model was designed for the purpose of making preliminary selection of desirable locations along the lower Mississippi river (Sogreah, 2003; BCG, 2004). While the SSPM is a distorted model, with the horizontal scale 1/12,000 and the vertical scale 1/500 (a distortion of 24), experimental results can provide important insights, especially when the SSPM is employed to compare identical diversion structure dimensions, using the same model and same operating procedures. Any difference in sediment diversion efficiencies is directly related to the structures' physical location and adjacent river current patterns. More quantitative analyses and studies related to structure design should be accomplished with larger scale modelling of sediment diversions from the Mississippi River to the wetlands.

A cursory analysis of the vertical distribution of the Mississippi sand and equivalent synthetic model sediment particles (sizes given in Table 1) was performed so as to understand and compare their impacts on the model sediment diversion.

Table 1 : Prototype and Model Sediment Properties

Particle diameter distribution	Mississippi River sand diameter (mm)	Model synthetic material diameter (mm)
d_{10}	0.08	0.25
d_{50}	0.12-0.14	0.38 – 0.45
d_{90}	0.25	0.80

The vertical distribution of the Mississippi River sand was computed using the average shear velocity based on average river water surface slope in 1973 between Head of Passes gauging station and at a point 58 miles upstream (about 9.5 miles upstream of Point A La Hache Gauge) and an average depth of flow about 21.00m or 68.88 ft (with average river bed elevation around – 60.0 ft NAVD88 at Magnolia (RM 47.5)). Table 2 shows the comparison of sediment concentrations at the water surface based on Rouses' graph of vertical distribution of suspended sand particles.

Table 2 : Surface Sediment Concentrations in Prototype and Model River

Particle diameter distribution	Mississippi River surface concentration at Magnolia (RM 47.5)	SSPM surface concentration at Magnolia (RM 47.5)
d_{10}	20%	35%
d_{50}	5%	20%
d_{90}	Trace at the water surface	5%

It was found that the sediment concentration at the water surface on the model is about 15% higher for the particle sizes of d_{10} and $d_{50,}$ and it is 5% more for d_{90}. This comparison is approximate and should be considered as such, but does indicate that in spite of the high distortion, the SSPM produces a relatively good reproduction of the prototype sand transport. A second point that needs to be emphasized is that this distortion is present everywhere on the SSPM and therefore the model can be used to compare different sites and scenarios, but does not provide the quantitative data necessary for design. This analysis, along with the detailed calibration and validation studies performed previously and reported to OCPR, demonstrate that the SSPM can provide some real and important decision-making information that may be as reliable many of the 2D and 3D numerical sediment transport models often used in Louisiana and elsewhere.

Overview of Two Schematic or Conceptual Designs of Diversion Structures

The SSPM was used to test two schematic design plans of the water and sediment diversion structures that have been suggested. The two schematic design concepts are:

- Gated broad crested sill by Coastal Restoration Construction (CRC).
- Narrow deep canal by Brown, Cunningham & Gannuch Engineering, Inc. (BCG).

The diversion structure (Fig.1) discussed by CRC can be described as a gated broad-crested sill that spans a total distance of approximately one mile along the river. Here, this structure is referred to as the "shallow" diversion. The idea behind placing the diversion structure along the riverbank close to the river water surface level is to take advantage of the secondary currents in large meandering rivers like the Mississippi. Locating the structures in areas where the secondary currents are oriented towards and impinging on the river bank with an appreciable angle, i.e., not parallel to it or moving away from the bank, will add momentum to the mass of water and sediment moving towards the bank and will transport a large portion of the sand load inland during the optimum river discharge of high sediment concentration.

It is recommended that these structures be gated so that the diversion discharge and its sediment distribution may be regulated and when necessary may be completely shut off. As more than 80% of the total sediment load is fine silt and clay, it is important that as much of these fines as possible be retained to contribute in the restoration of the wetlands. With an un-gated structure, the fines are generally flushed out during the receding flood, thereby cancelling the overall efficiency of the land building efforts.

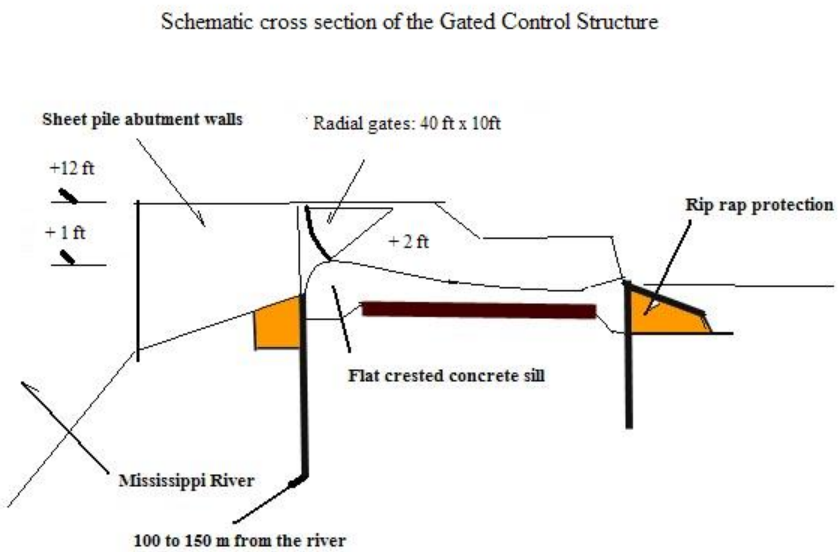

Figure. 1 – Schematic Representation of a Broad Crested Sill Diversion Structure as proposed by CRC

This shallow diversion structure employs a broad-crested sill with an average crest elevation of approximately + 2.0 ft NAVD88 and a total length of about 1 mile (without considering the pier widths separating the 40 ft x 10 ft gates). The radial gates would be 40 ft wide and 10 ft high, remotely controlled and operated by hydraulic cylinders. The discharge through the diversion can be estimated for different river stages by using assuming that the flow through the open structure is a broad crested weir at an elevation of +2.0 NAVD88 (Figure 2). Assuming that the Mississippi River water surface elevation at the project site is + 8.5 ft NAVD88 for a river discharge of 1,250,000 cfs (which would correspond to a head level of 6.5 feet of water over the structure) the corresponding flow through the structure would be approximately 150,000 cfs (Figure 2). The discharge at other river stages can be estimated using the relationship in Figure 2.

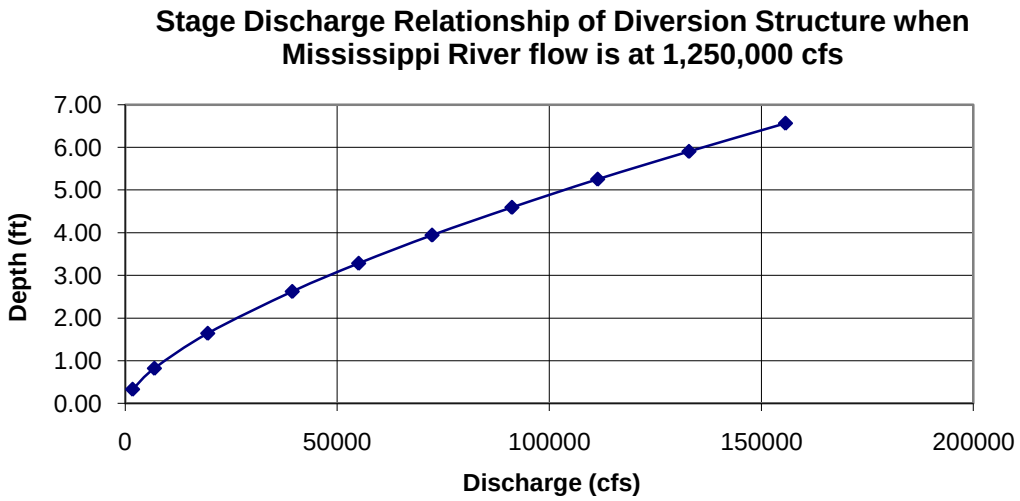

Figure. 2 – Computed Stage Discharge Rating Curve of a Diversion Structure With a Broad Crested Sill at elevation + 2.0 ft NAVD88 (CRC Design).

The deep channel diversion structure proposed by BCG in the Summer of 2009 has an inlet design with a channel bottom elevation near the riverbed level to capture the sand load transported by the Mississippi River. That sediment diversion structure design consists of a sheet pile sediment deflector designed to direct sediment into the structure and a 160 ft x 45 ft concrete flume with its invert set at elevation – 40.0 ft NAVD88 and the slab foundation at – 50.0 ft NAVD88. The discharge capacity of the BCG design (deep diversion) at a river discharge of 1,250,000 cfs is 75,000 cfs (Figure 3). Four vertical lift gates, each composed of four 10.5 ft x 40 ft individual panels, control the discharge and flow shut off. In order to divert 150,000 cfs, the invert width might have to expand to 340 ft. and the number of gates and gantry cranes doubled.

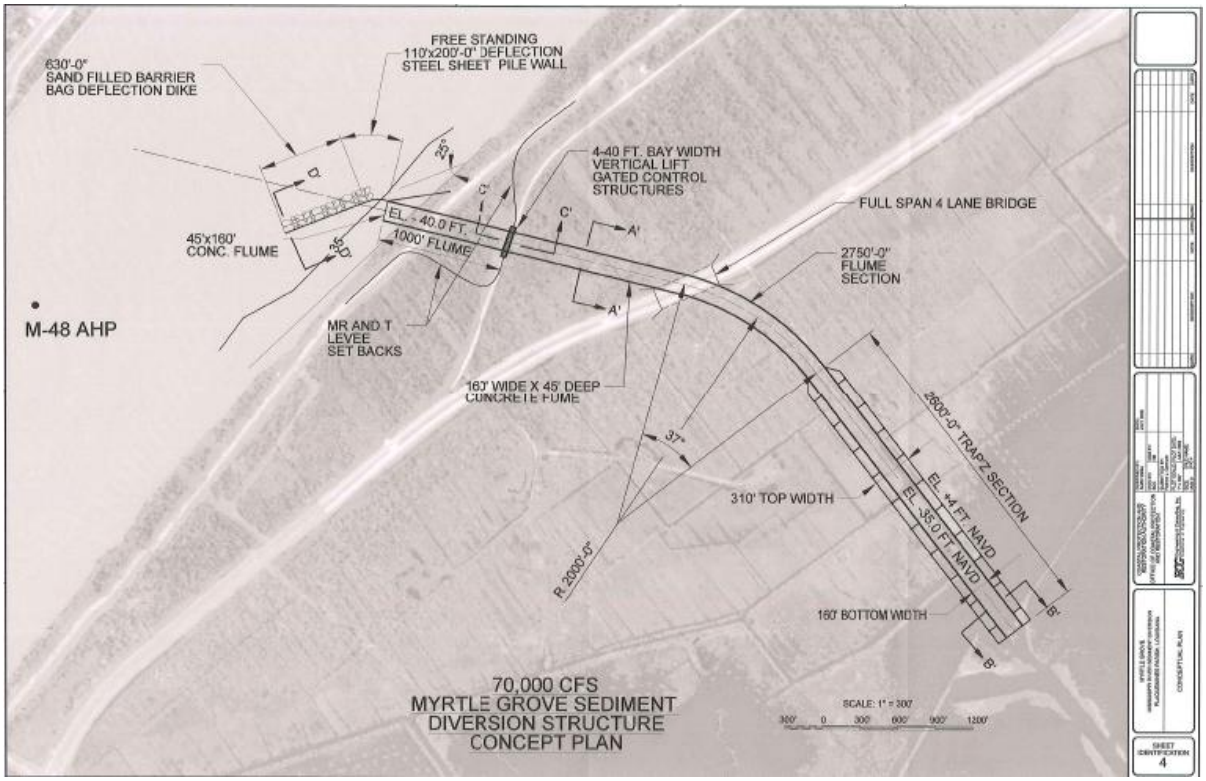

Figure. 3 - Schematic Design of Deep Channel Diversion Structure.

Figure 4. Pictures showing the two diversion structures tested at Myrtle Grove; wide shallow proposed by CRC (left), deep proposed by BCG (right).

Site Selection

Three separate sites were selected for testing (Figure 5). Because the SSPM only extend to ~ RM61, the first site, at Myrtle Grove (RM 59), was chosen because that is the nearest location on the SSPM to the site currently being considered by the U.S. Army Corps of Engineers (USACE) as a diversion site. The second site is located near Magnolia (RM 47.5). The third site, located at Woodland (RM 51), was chosen as an intermediate site.

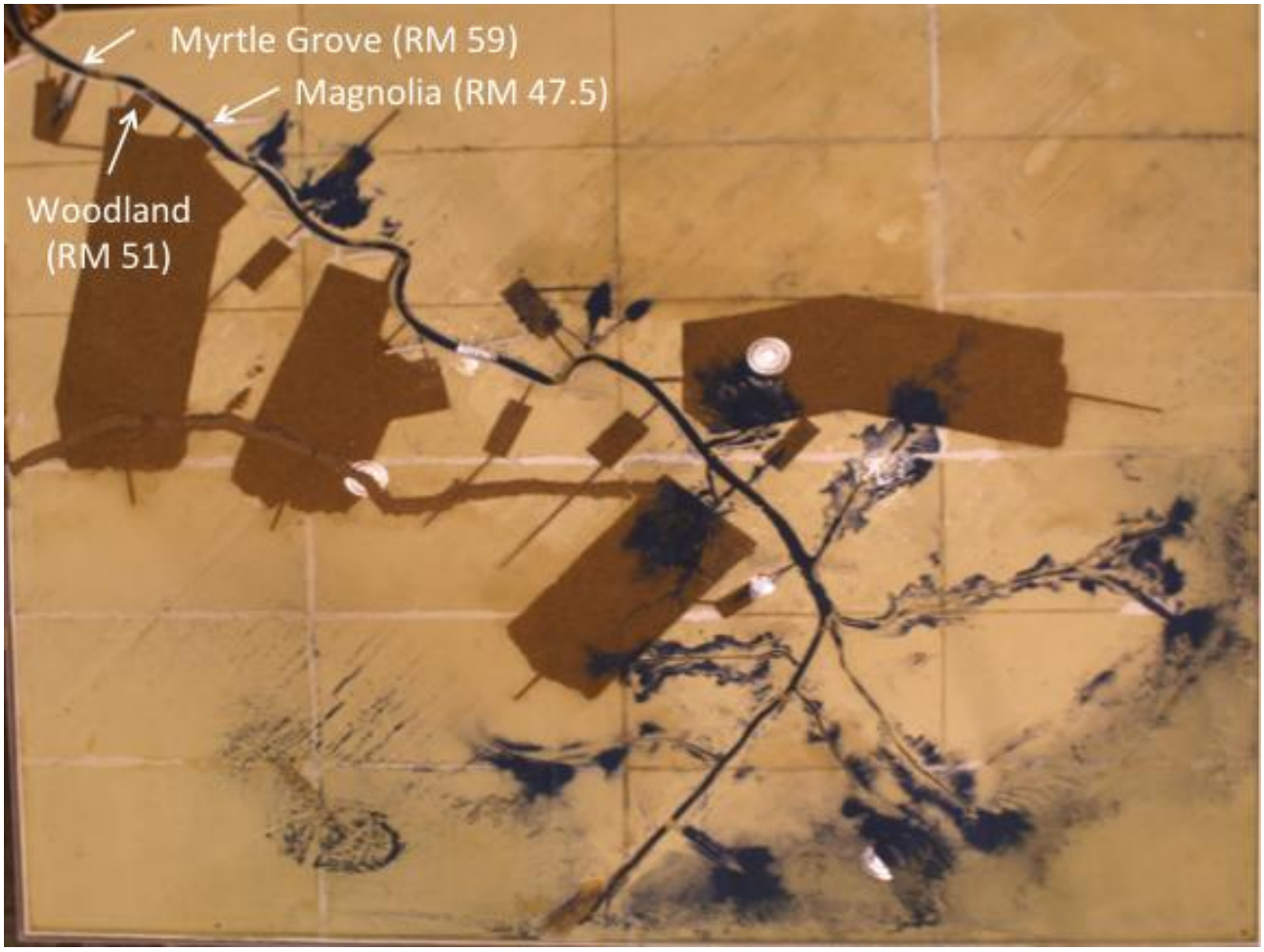

Figure. 5: Diversion Locations for the SSPM tests

As can be seen in Figure 6 the Myrtle Grove (RM 59) diversion site is located just downstream from a concave river bend and is similar in orientation to the placement of the original, USACE alignment. The channel bottom in this section is very deep and located close to the right descending bank. The Magnolia (RM 47.5) site is located downstream of a convex river bend near a lateral bar (Figure 7). As seen in Figure 8, the Woodland location (RM 51) has similar characteristics to Magnolia. These bathymetric data indicate that the water along the right bank at Myrtle Grove (RM 59) is sediment lean and at Magnolia (RM 47.5) and Woodland (RM 51) is sediment rich. Thus, it is anticipated that a diversion structure located near a convex bend or a lateral bar would divert much more sediment out of the river.

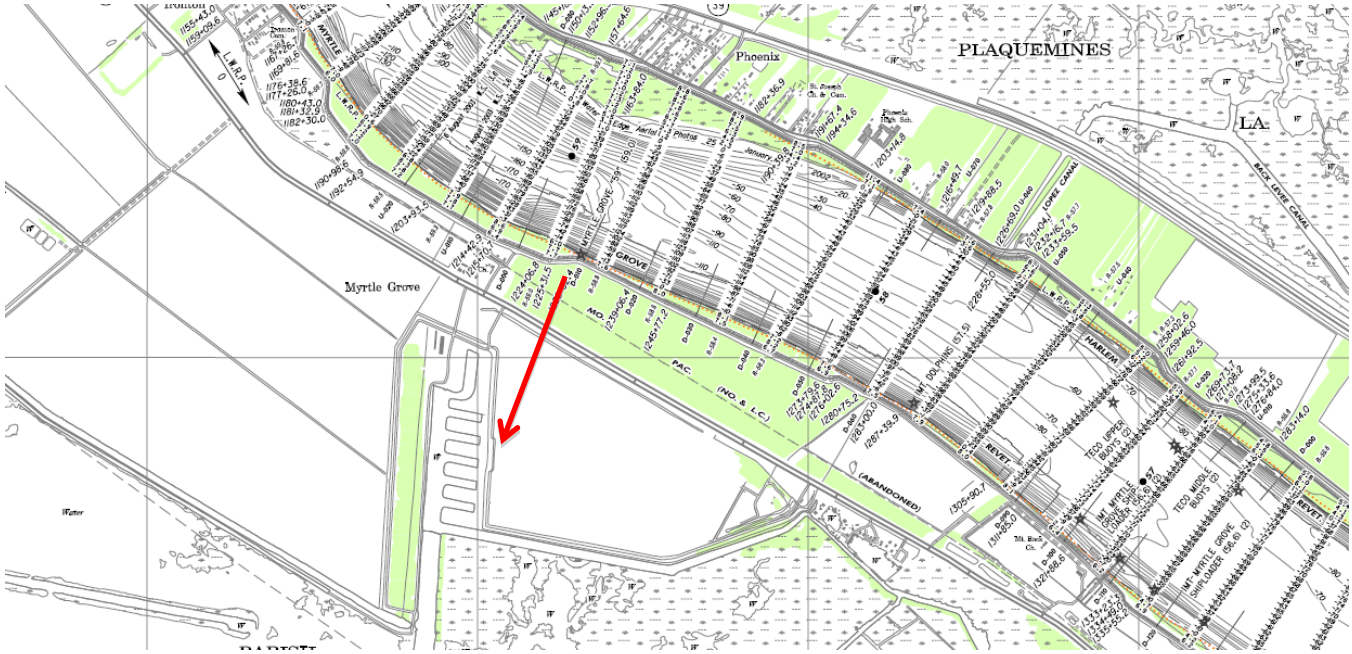

Figure 6- Myrtle Grove (RM 59) Diversion Location Bathymetric Survey (red arrow shows the approximate location of the diversion)

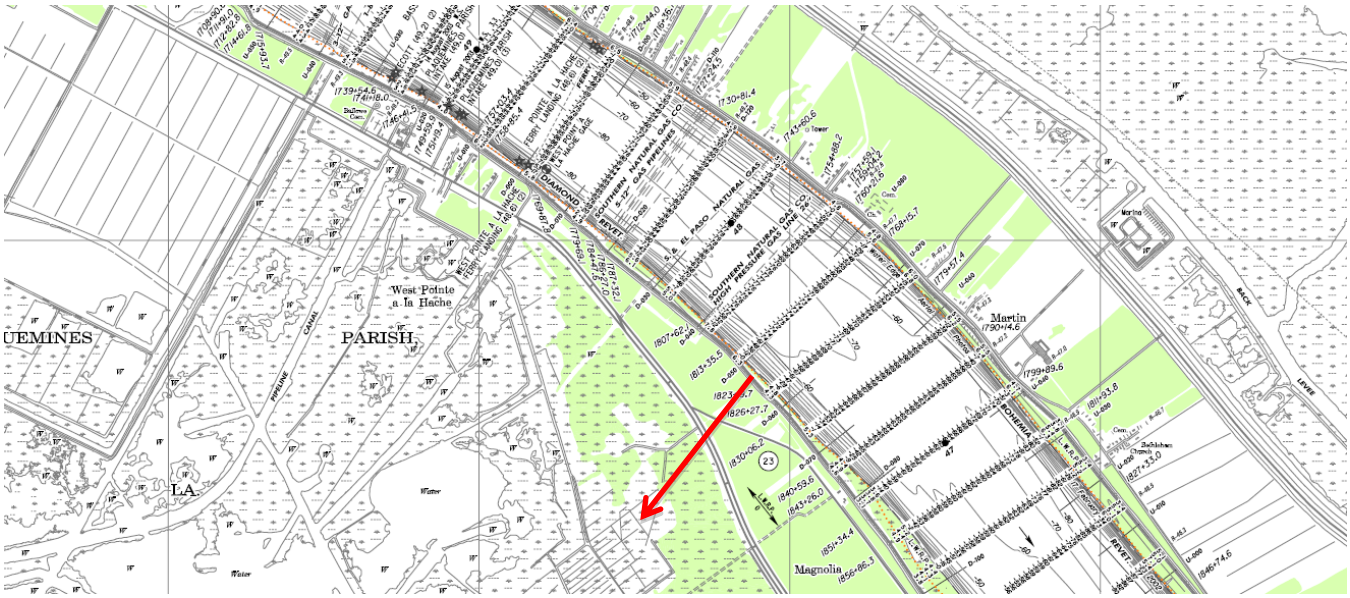

Figure 7 - Magnolia (RM 47.5) Diversion Location Bathymetric Survey (red arrow shows the approximate location of the diversion)

Figure 8 - Woodland (RM 51) Diversion Location Bathymetric Survey (red arrow shows the approximate location of the diversion)

Testing and Assessment of the Total Yearly Sediment Diversion

All tests were run using the SSPM standard operating procedures (BCG, 2004; Willson et al., 2007). Data collected during each test included river stages; mass of sediment injected, dredged, exiting model; and high-resolution overhead photos and video. This approach allows for more direct comparison between the different diversion locations and structure types.

Myrtle Grove and Magnolia Shallow 100 year

A 100-year simulation was made with two identical shallow diversion structures, one at Myrtle Grove (RM 59; Figures 5 and 6) and one at Magnolia (RM 47.5; Figures 5 and 7), operating simultaneously. As detailed in BCG (2005), thirty minutes of model time is equivalent to one year of prototype (river) time. Thus, a 100-year simulation takes 50 hours of model time, plus additional time for model setup, maintenance, periodic verification, etc. While this operational mode (i.e., both diversion operating simultaneously) is probably not likely to be followed, this configuration was used to optimize the experimental time. It should also be noted that the Myrtle Grove diversion would most likely have a negative impact on the Magnolia diversion, but not the other way. From the beginning, it was obvious that the Magnolia (RM 47.5) diversion structure captured a significantly larger amount of sediment than the Myrtle Grove (RM 59) diversion. High-resolution video was taken which clearly shows significant differences in the flow directions and sediment transport patterns at the two diversion locations. This video has been delivered to OCPR.

Figure 9 shows the surface area covered by the diverted sand deposits at Myrtle Grove (RM 59) and Magnolia (RM 47.5) after 100 years; Figure 10 is as an overhead photo of the entire SSPM at the same time.

It should be noted that because of the minimum entrainment velocities over the deposited sediments outside the main river channel, the areas covered by the sand deposits are relatively smaller than what would be in the prototype. Significantly higher flow turbulence in nature would transport the sands much further and would cover much larger areas.

Figure 9 – Comparative sediment diversion efficiencies at Myrtle Grove (RM 59) and Magnolia (RM 47.5). This photo shows Myrtle Grove (RM 59) on the left and Magnolia (RM 47.5) on the right after 100 years of operation with identical structures. As can be seen, the Magnolia (RM 47.5) location would produce much more sand to divert into the wetlands.

Figure. 10 – Bird's eye view of the model showing the total diverted sediments off the Mississippi River (other than the Myrtle Grove (RM 59) and the Magnolia (RM 47.5) sites) over the 100 year testing period. (Fig. 12 shows the corresponding sediment balance).

Sieve analysis of the sediments (Figure 11) show that the inflow sediment size remains consistent with the original SSPM design specifications (BCG, 2005) and that a higher % of finer sediment particles are being diverted at Myrtle Grove (RM 59), i.e., more from the surface layer of the river flow. On the other hand, the gradation curve for the Magnolia (RM 47.5) location is nearly identical to the inflow sediment gradation indicating that coarser sediment particles are being diverted from the bottom layers of the river flow, as bed loads are entrained by the secondary currents in this reach of the river.

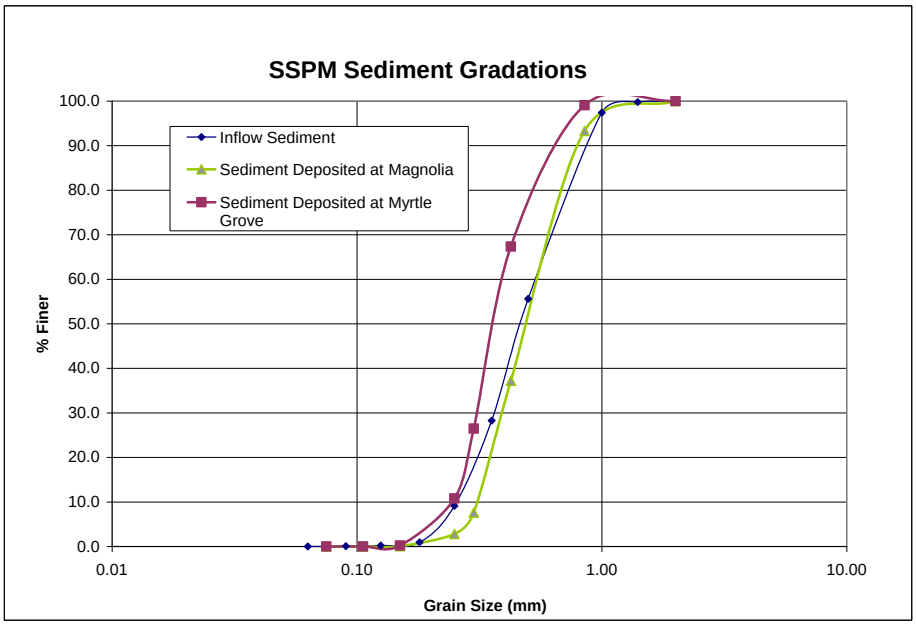

Figure. 11 – Sediment gradation curves for inflow sediment load, sediment deposited at Magnolia (RM 47.5) and sediment deposited at Myrtle Grove (RM 59)

Total sediment injected into the model was 2 billion tons; the amount dredged within the main channel was 842.5 million tons and amount deposited was 967.9 tons. Figure 12 shows that approximately 40% of the sediment injected into the river was dredged and that the total amount deposited within the system was just under 50%. The remaining 10% leaves the SSPM model surface via the overflow weirs, primarily off the continental shelf.

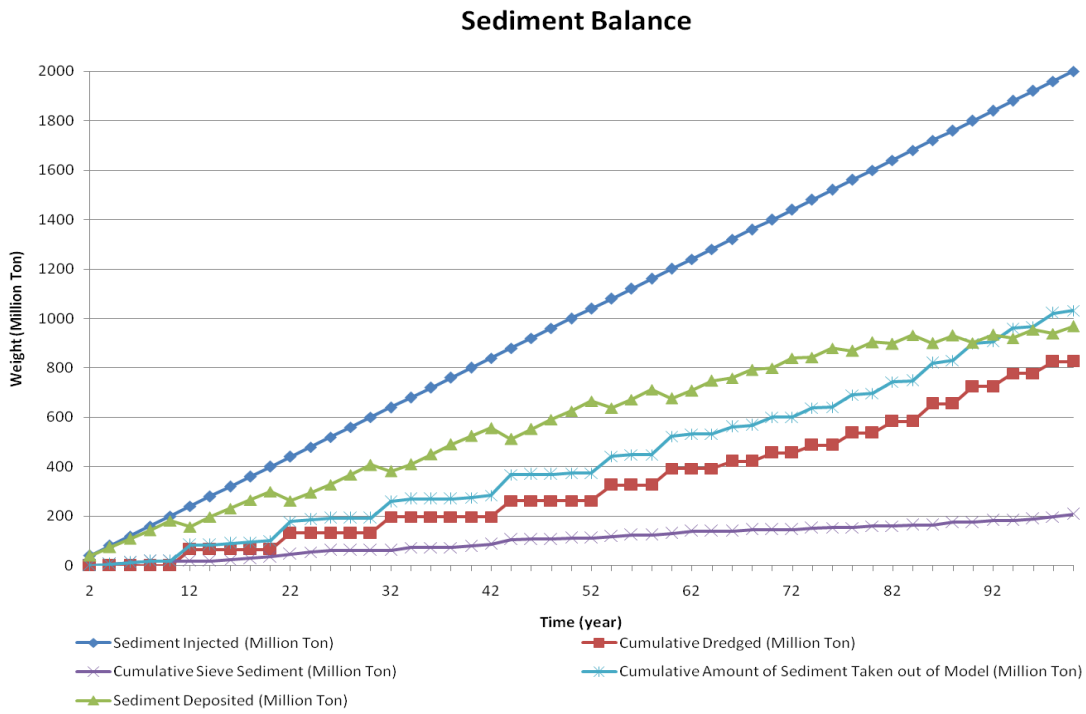

Figure. 12 – Example Sediment Balance After 100 Year Operations.

Woodland Shallow 50 & 100 Year

Two tests were run using the shallow diversion structure at Woodland (RM 51) (Figures 5 and 8). The first was conducted immediately following the Myrtle Grove and Magnolia shallow 100 year test and was a 100 year test. Due to time constraints, this experiment was run prior to removal of the model sediment from the Myrtle Grove and Magnolia diversion outfall areas. Thus, merging of the sediment deposits from the Magnolia and Woodland deposition areas make photos of just Woodland inconclusive and prevented independent measurements of the sediment deposited from each of the diversions. Thus, any analysis of the sediment deposited uses the combined weight of the Magnolia and Woodland deposited sediments. The second test was run for 50 years after a reset of the system. All of the standard data was collected in the 2nd test.

Magnolia and Myrtle Grove Deep 50 Year

50 year tests were run on the Magnolia (RM 47.5) and Myrtle Grove (RM 59) diversion locations. The deep diversion structures were based on the BCG design and the Myrtle Grove model deep diversion is shown in Figure 4. These tests were run using the same procedures as in the previous tests.

Figure 13 shows the sediment deposition areas after 50 years of testing. Qualitatively, it is obvious that the Magnolia (RM 47.5) and Woodland (RM 51) locations diverted more sediment than the Myrtle Grove (RM 47.5) location and that the shallow diversion structures appear to divert larger quantities of sediment than the deep structures.

Figure. 13 – Photographs of the Sediment Deposition Areas Following 50 years for the Different Test Conditions.

Sediment Transfer Efficiency Calculations

Annual sediment (sand) diversion efficiencies were calculated based on weights of sediments collected at Myrtle Grove (RM 59) for deep (50 year) and shallow (100 year) tests, the combined Magnolia (RM 47.5) and Woodland (RM 51) location for shallow tests, the Magnolia (RM 47.5) deep test, and the 50 year Woodland shallow test..

<u>Myrtle Grove (RM 59) Shallow Site Data</u>
Total weight of sediment diverted in 100 years: 352.2 g
Using 1g model = 87,815.58 t prototype, we calculate 30,928,645.83 t for 100 years
The annual amount of sediment diverted: 309,286.49t, i.e.,
309,286.49/20,000,000 = 1.55% for a maximum flow diversion of 150,000/1,250,000 = 12%
Effective sediment transfer efficiency = 1.55/12 = 12.8%

Magnolia (RM 47.5) + Woodland (RM 51) Shallow Site Data
Total weight of sediment diverted in 100 years: 3994.9 g.
Assuming 50% from each diversion, we have 1997.45 g at Magnolia (RM 47.5) and 1997.45 g at the Woodland (RM 51) Site.
Using 1g model = 87,851.58 t prototype, we calculate 175,407,244.78 t for 100 years
The annual amount of sediment diverted: 1,754,072.45 t, i.e.,
1,754,072.45/20,000,000 = 8.77% for a maximum flow diversion of 150,000/1,250,000 = 12%.
Effective sediment transfer efficiency: 8.77/12= 73%

Woodland (RM 51) Shallow Site Data
Total weight of sediment diverted in 50 years: 753.1 g
Using 1g model = 87,851.58 t prototype, we calculate 66,161,024.9 t for 50 years
The annual amount of sediment diverted: 1,323,220.5 t, i.e.
1,323,220.5 t/20,000,000 = 6.62% for a maximum flow diversion of 150,000/1,250,000 = 12%.
Effective sediment transfer efficiency: 55.13%

Magnolia (RM 47.5) Deep Site Data
Total weight of sediment diverted in 50 years: 372.0 g
Using 1g model = 87,851.58 t prototype, we calculate 32,680,790 t for 50 years
The annual amount of sediment diverted: 653,615 t, i.e.
653,615 t/20,000,000 = 3.26% for a maximum flow diversion of 150,000/1,250,000 = 12%.
Effective sediment transfer efficiency: 27.23%

Myrtle Grove (RM 59) Deep Site Data
Total weight of sediment diverted in 50 years: 33.4 g
Using 1g model = 87,851.58 t prototype, we calculate 2,934,242 t for 50 years
The annual amount of sediment diverted: 58,684.8 t, i.e.
653,615 t/20,000,000 = 0.29% for a maximum flow diversion of 150,000/1,250,000 = 12%.
Effective sediment transfer efficiency: 2.44%

The comparative efficiency of the sediment diversions at Magnolia (RM 47.5) and Woodland (RM 51) locations in relation to Myrtle Grove (RM 59) is 8.77/1.55 = 5.66 times greater. The comparative efficiency of the sediment diversion at the 50 year Woodland (RM 51) location in relation to Myrtle Grove (RM 59) is 6.62/1.55 = 4.27 times greater. In addition, comparison of the Magnolia (RM 47.5) deep test to the Myrtle Grove (RM 59) deep test showed that the Magnolia location was 3.26/0.29 = 11.2 times more efficient. These values confirm the original hypothesis that the Myrtle Grove location (at least the location tested in the SSPM) would not be conducive for efficient delivery of sediment into the diversion structure.

An additional comparison can be made between the deep and shallow diversions. The efficiency of the Magnolia (RM 47.5) shallow relative to the Magnolia (RM 47.5) deep is 8.77/3.26 = 2.7 times greater and the Myrtle Grove (RM 59) shallow relative to the Myrtle Grove (RM 59) deep is 1.55/0.29 = 5.3 times geater. Given the limitations of the SSPM to replicate complex 3D flow patterns, the results showing that the shallow diversions are more efficient than the deep ones should be viewed as semi-quantitative. Additional larger scale tests would be required to confirm and better understand the results.

Conclusions

For identical structural arrangements, the SSPM has allowed us to demonstrate that it is possible to divert significant amount of more sediment by adequate site selection of the diversion structure. Pre-study hypotheses concerning the impact of river channel geometry combined with analysis of the bathymetric survey charts and knowledge of riverine hydrodynamics and sediment transport processes were confirmed. The two sites located downstream of convex river bends near lateral bars, Magnolia (RM 47.5) and Woodland (RM 51), both performed much better than the Myrtle Grove (RM 59) site located downstream of a deep, concave river bend.

While not sufficient for site design work, the SSPM results showed that the shallow, wide diversion structures, proposed by CRC, outperformed the deep diversion structures. Note, however, that additional larger-scale physical modeling, analysis of field data and high-resolution 3D numerical modeling should be performed to guide detailed design.

References

- Brown, Cunningham, and Gannuch (BCG) Engineering Consultants, (2004) "Report of the Feasibility of Small Scale Physical Model of the Lower Mississippi River Delta for the Testing Water and Sediment Diversion Projects." State of Louisiana Department of Natural Resources DNR. Contract No. 2503-03-04. Dec. 2004.

- Sogreah, (2003). "Small-Scale Physical Model of the Mississippi Delta", Report No. 345044 submitted to Louisiana Department of Natural Resources, 47 pp.

- Willson, C.S., N. Dill, S. Danchuk, W. Barlett, and R. Waldron, 2007, Physical and Numerical Modeling of River and Sediment Diversions, Proceedings of the Sixth International Symposium on Coastal Engineering and Science of Coastal Sediment Processes, May 13-17, 2007, New Orleans, LA. Ed. N.C. Kraus and J.D. Rosati, 749-761.

APPENDIX B

CONCEPTUAL DESIGNS

MYRTLE GROVE SEDIMENT DIVERSION STRUCTURE

DRAFT

PREPARED
FOR

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BY:

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3012 26TH STREET
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June, 2009

CONCEPTUAL DESIGNS

MYRTLE GROVE SEDIMENT DIVERSION STRUCTURE

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CONCEPTUAL DESIGNS

MYRTLE GROVE SEDIMENT DIVERSION STRUCTURE

1. GENERAL. This project study was authorized by the Office of Coastal Protection and Restoration, of the State of Louisiana, by letter and notice to proceed dated January 2, 2009, under Task Order 2, Contract No. 2503-08-04. The purpose of the study is to determine feasibility of a large sediment and freshwater diversion project in the vicinity of Myrtle Grove, Louisiana, for the purpose of land building and marsh creation using sediments available from the Mississippi River, and to quantify the benefits and impacts related to the project. The purpose of this report is to provide essential features of the structural means needed to cause a successful project outcome irrespective of the ultimate peak capacity chosen by applicable authority.

2. BASIC ASSUMPTIONS. This effort is being carried out based upon the basic assumption that this Diversion Project can be located in the vicinity of Myrtle Grove at a point on the river where the highest potential for successful sediment diversion and land building can be found. Under consideration are sites at Mile 60, Above Head of Passes, (AHP), Mile 58, and Mile 48. Important considerations in site selection include extent of existing infrastructure, likely cost of relocations, and social and transportation impacts of any choice. Preliminary studies have already been carried out by the Corps of Engineers at Miles 58 and 60 to determine costs of both freshwater only and sediment diversions at 5,000 and 15,000 cfs levels of discharge. These conceptual designs are being applied to the vicinity of Mile 48 AHP because preliminary results of the Small Scale Physical Model (SSPM), have show that the likely hood of success is better here than at Miles 58 or 60 AHP.

3. BASIC HYDROLOGY OF MISSISSIPPI RIVER.

a. References.

1) "*Dredging Alternatives Study, Cubits Gap, Lower Mississippi River*", dated January 1991, by Ronald R. Copeland, PhD., the Waterways Experiment Station, Vicksburg, Mississippi.

2) "*Report on Feasibility of Small Scale Physical Model of Lower Mississippi River Delta for Testing Water and Sediment Diversion Projects*", dated December 2004, by BCG Engineering & Consulting, Inc., Coastal Restoration Consultants, and Louisiana State University, Baton Rouge, Louisiana.

3) "Shoaling *Implications of West Bay Sediment Diversion Project on Pilottown An*chorage", dated June 2009, by BCG Engineering & Consulting, Inc., for the Coastal Protection and Restoration Authority, State of Louisiana.

4) "Information Paper, MCR/LMVD Task Group Study of Navigation and Freshwater Diversion Projects, Baton Rouge to the Gulf", dated October 1985, by the U. S. Army Corps of Engineers, Vicksburg, Mississippi.

5) "*Mississippi River Sediment, Nutrient, and Freshwater Redistribution Study*", dated July 2000, by the Louisiana Coastal Wetlands Conservation & Restoration Task Force, for the U.S. Army Corps of Engineers, New Orleans District.

6) "*The Transport of Sediment by Gravity for Coastal Restoration and Wetland Rehabilitation in Louisiana*", dated September 15, 1997 by the University of New Orleans, Sogreah Engineering Institute, and Coastal Restoration Consultants, LLC, Baton Rouge, Louisiana.

b. Historical Peak Flows. Table 1 is a table of measured annual peak discharges made on specified dates at Tarbert's Landing, Louisiana on the Mississippi River. Peak flows for 1983, 1997, and 2008 were modified at the latitude of Myrtle Grove to no more than 1,250,000 cfs by a diversion to Lake Pontchartrain of excess flow above 1,250,000 cfs. The average peak flow for the period shown is 1,092,000 cfs at Tarbert's Landing and 1,067,000 cfs at Myrtle Grove. However, unmeasured leakages through Bonnet Carré Floodway diminish the peak.

c. Average Flow Hydrograph. The shifted annual histograph taken from Plate 4 in Reference 1) is a stair step plot of flows that was used to drive a 1-D numerical model of the Mississippi River in the 1991 study. See Plate 4, Attachment 1, in Appendix A. This histogram, plus adjustment to the average peak flow at Myrtle Grove to account for leakage at Bonnet Carré, and design diversions at Davis Pond and Carnarvon, was used to generate the primary design flow hydrograph at the latitude of the Myrtle Grove Sediment Diversion Structure. That flow hydrograph is shown at Attachment 2.

d. Average Rating Curve. The River level within the reach, Mile 48 AHP to 60 AHP, must be known for any given design discharge, in order to size the opening of a diversion structure to be able to convey water and sediments out of the river into the waters of Barataria Bay. However, the water surface of the lower reach of the Mississippi River below the latitude of the Bonnet Carré Floodway is greatly influenced by tides in the Gulf of Mexico. Consequently, a general plot of stage at any given time versus flow will produce a wide range of scattered points within a range of two (2) feet vertically over time. Such a scatter of points is useless and an average of the points must be plotted with associated flows in order to produce a Rating Curve at Myrtle Grove that may also be applied at Magnolia. Such a rating curve is given at Attachment 3.

4. BASIC HYDRAULIC ANALYSES. Any analysis of a long-term riverine project should include a Potamology Study to establish how the river has been behaving in the recent past in order to avoid unwanted changes or remediation in the future, particularly if the project is of notable importance and is likely to have a long life cycle, say 100 years, for example. In the case of the Mississippi River, this is of extreme importance due to the expense associated with remediation and the requirement for flood control and navigation safety. Fortunately, the Revetment Program provided by the U.S. Army Corps of Engineers is providing long-term stability in this study reach. Attachments 4 and 5 demonstrate the long-term stability of two typical cross-sections of the Mississippi River at Miles 50 and 64 AHP as provided by Mobile Boundary Hydraulics, LLPC, which were compared to historical hydrographic surveys in the 1980's at those same locations. Just as important are the considerations that must be given to the design of a long-term diversion structure and conveyance channel. Adequate, mechanical control, scour protection, and sediment transport efficiency are paramount to a safe, productive land building project. Likewise, changes in water surface elevations in the river, and in the receiving water bodies, as well as the local communities must be quantified to avoid negative benefits, such as changes in local drainage or water supply, i.e., salinity levels, etc. In order to quantify these impacts, hydraulic studies must be made in conjunction with known Hydrology to assess the changes that will occur over the life of the diversion. For example, if the project is completely successful, depositional effects will change the tail water levels in the receiving water

body, and reduce the efficiency of the diversion structure over time. To offset such impacts, the structure must be sized to meet future expectations.

TABLE 1

ANNUAL PEAK DISCHARGES AT TARBERT'S LANDING, LOUISIANA

YEAR OF FLOW EVENT	Q IN 1,000 CFS	MONTH PEAK OCCURRED
1982	873	APR.
1983	1470	JUN.
1984	1199	MAY
1985	1128	MAR.
1986	822	OCT.
1987	970	MAR.
1988	1000	JAN.
1989	1138	MAR.
1990	1230	JUN.
1991	1303	JAN.
1992	770	APR.
1993	1190	APR.
1994	1164	MAY
1995	1167	JUN.
1996	1026	JUN.
1997	1480	MAR.
1998	1080	MAY
1999	1179	FEB.
2000	684	APR.
2001	1120	MAR.
2002	1116	APR.
2003	1015	JUN.
2004	1007	DEC.
2005	1229	FEB.
2006	735	MAR.
2007	930	JAN.
2008	1456	APR.

a. Natural River Cross-sections. The siting of a diversion requires particular attention to the geomorphology in the vicinity of the project. Consequently studies were made to determine the most logical locations strictly on the basis of the shape and depth of the Mississippi River in the study reach. Attachments 4 and 5 are examples of cross-sections that occur at point bars in the River at bends where the river may change directions. This occurs at Mile 64 AHP on Attachment 5 where the cross-section is plotted looking downstream. The point bar is on the right side of the River and is a natural feature of meandering rivers. Large quantities of sand are naturally stored here and are generally best suited for the location of sediment diversion projects.

b. Diversion Channel Cross-sections. Given the express purpose of conveying high concentrations of sediments in full suspension above the bed of the diversion channel into the receiving water body, that channel must be super efficient, must have as small a wetted perimeter as practical to provide a large hydraulic radius, must be smoother to conserve available energy, yet must have a high flow velocity and must cause turbulent flow for good transport capability. However, hydraulic efficiency and turbulence are generally at cross purposes and therefore counter balance each other, and sediment diversion places addition requirements on the design, because the depth of the point bar, -35 feet at Mile 50 AHP, will govern the shape of the diversion channel, and the wetted perimeter. Based on the average annual peak flow of approximately 1.05 million cfs, a river stage of seven (7) feet is expected, neglecting the tidal effects, so a total depth of 42 feet ((-35') + (7')) over the gate sill would dictate the most efficient cross-section. It is common knowledge that greater hydraulic efficiency can be achieved if the width of a channel is twice the depth, in which the width would be 2 x 42 feet or 84 feet. However, these dimensions, 42 feet x 84 feet, will provide more than twice the diversion flow being used, 15,000 cfs, as the target flow for modular design. However, this size channel could be used as a modular target to increase the design flow by 2. Therefore, the basic design for the diversion channel will be a 42 feet x 40 feet concrete flume that will produce adequate turbulence at an average velocity of 9 feet per second.

c. Water Surface Profiles. For this preliminary design, a site at Mile 48 near Magnolia was chosen as a basis to construct a 1-D HEC-RAS model of the Mississippi River and the diversion channel, for the purpose of computing a water surface profile in the river and through the diversion channel and control gate for flows of 15,000 and 70,000 cfs. These two flows were selected to bracket the practical limits of expected diversions for the Myrtle Grove project. The flow rate in the Mississippi River was chosen as 750,000 cfs for these computations to be representative of an intermediate rate between incipient sediment motion (600,000 cfs) and the average annual peak of 1,050,000 cfs. Attachment 6 shows a water surface profile between Mile 47 AHP and Mile 49 AHP for a flow of 680,000 cfs below the diversion channel and a flow of 750,000 cfs upstream of the diversion channel. Attachment 7 is a plot of the water surface profile along the diversion channel for a flow rate of 70,000 cfs. Attachment 8 is a tabulation of the hydraulic computations made in the HEC-RAS model for the flow conditions shown in Attachments 6 and 7. The important flow parameters to note here are the energy gradelines at the junction of the diversion channel with the river and the flow velocities in the diversion channel that would sustain sediment entrainment to the receiving water body. Note that the differences in the water surface elevations at the junction is the head required to accelerate the flow coming from the river at approximately 3 feet per second to 9 feet per second into the diversion channel. This is demonstrated by the sharp drop in water elevation near the upstream end on the diversion channel water surface profile in Attachment 7.

5. BASIC SEDIMENT TRANSPORT. It is of primary importance to this project that it is able to transport high sediment concentrations of sands, silts, and clays effectively in to the Barataria Basin. This can be accomplished at any time there are suspended materials in the Mississippi River and sufficient water surface differences between the river and the basin. But what are the limitations that ultimately render the diversion ineffective with respect to sands, which are the building block to land creation. Every stable sea coast either is composed of rock, or is fronted with a sandy beach that lines the weaker soils beneath it. This section discusses tasks that have been carried out to determine the threshold velocity that will just begin to move bed load that has settled to the bed of the channel at the end of the last seasonal operation.

a. Basic Assumptions. Parameters governing the problems include water temperature and density, salinity, grain size and distribution, water depth and average velocity through the channel. We will assume that river water is fresh and the temperature is 15 degrees centigrade, that the water depth is minimum of 38 feet, and that the grain size distribution is similar to that used at Belle Chasse for operation of the Small Scale Physical Model. Attachment 9 presents similar date for bed load gradations in the Mississippi River below Venice, Louisiana. These are characterized as: D_{10} = 0.1 mm, D_{50} size = 0.19 mm, and D_{84} size = 0.3 mm. The calculations that follow will determine by trial and error what the average velocity must be in the diversion channel for incipient motion at the bed for the median sizes.

b. Basic Transport Calculations. The procedure, and knowledge, used here is that outlined by Richard Soulsby in a book entitled "*Dynamics of Marine Sands*" and published in 1997 by Telford Publications, 1 Heron Parkway, London, UK, particularly Chapter 3, Page 45 on "Currents". So having the site particulars previously given in a. above, by trial velocities of 2 to 8 feet per second were used to forecast the incipient motion of Mississippi River bed material through the diversion structure and channel, and the transport per unit width in meters squared per second, m^2/s, which may then be converted to volume by multiplying by 1 meter width, then converting to cubic yards. Attachments 10 to 17 provide a print out of the calculations.

c. Results. The results of the sediment calculations indicate that some sands will begin moving off the bar into the diversion structure when the discharge is as low as 4,000 cfs for any river condition above 500,000 cfs, which is needed to induce incipient bed material motion for the median grain sizes considered typical in the reach. However, Attachment 9 shows an envelope of the sampled data, and some sizes smaller than the median will begin to move earlier on a rising flow hydrographic. Consequently, although pulsing is the preferred method of operation among many proponents of the project, it will be feasible to transport sediment through the project for longer periods. This feature will allow for more effective project management and land building opportunities. Table 2 demonstrates that on the basis of an average 81 day diversion period, as shown on the Diversion Schedule, Attachment 18, 1.66 million yards of sand and 6.6 million yards of sand, for 15,000 and 70,000 cfs, respectively, could be diverted annually. Since sands make up about 25% of the total suspended load and silts and clays make up the remainder, it is possible than to divert $^{1.66}/_{.25}$ = 6.64 million up to $^{6.6}/_{.25}$ = 26.4 million yards of river sediments over an 81 day diversion period for the current range of diversions under consideration. However, for a much longer diversion period, say 120 days annually, the deposition volume could be increased by 50% of these limits. Attachment 18 is the outflow rate for a 15,000 cfs diversion structure and the Mississippi River average annual flow hydrograph shown in Attachment 2. A similar diversion schedule for 70,000 cfs would produce flow rates about 4.6 times higher than those shown on Attachment 18.

6. CONCEPTUAL DIVERSION STRUCTURE DESIGNS. Since at this point in time, June 25, 2009, it is not yet known with any certainty what size and flow capacity will be funded at the Myrtle Grove project site, the Office of Coastal Protection and Restoration, and the team of Consultants, concluded that a modular approach to the design process would be appropriate. Therefore, a basic size that would provide a peak 15,000 cfs flow rate through a diversion structure for a mean annual peak flow rate of 1,050,000 cfs in the Mississippi River, was chosen as the building block for the modular approach. Details of this design, and the design of the ultimate size of 70,000 cfs are given in the following discussion.

TABLE 2

TOTAL ANNUAL LOAD TRANSPORT POTENTIAL
(in cubic yards)

Feet / Second	2	2.5	3	4	5	6	7	8
Meters / Second	0.61	0.76	0.914	1.22	1.52	1.83	2.13	2.44
M^2/S/M	1.53 E-05	7.44 E-05	21.0 E-05	80.0 E-05	215.0 E-05	469.9 E-05	868.0 E-05	1,483.0 E-05
CU^3/Yards/M	2.0 E-05	9.75 E-05	27.5 E-05	104.8 E-05	282.0 E-05	615.6 E-05	1,137.0 E-05	1,943.0 E-05
Yards/Day/M	1.73	8.42	23.76	90.5	243.6	532.0	982.0	1,679.0
Times 40'/3.28'/M	21 Yds. / Day	103 Yds. / Day	289 Yds. / Day	1,104 Yds. / Day	2,971 Yds. / Day	6,490 Yds. / Day	11,980 Yds. / Day	20,484 Yds. / Day
81 Days / Year	1,710 Yds. / Year	8,345 Yds. / Year	$2.3x10^4$ Yds. / Year	$8.9x10^4$ Yds. / Year	$2.4x10^5$ Yds. / Year	$5.26x10^5$ Yds. / Year	$9.7x10^5$ Yds. / Year	$166x10^6$ Yds. / Year
Times 160'/3.20'/M	84.4 Yds. / Day	410.9 Yds. / Day	1,159 Yds. / Day	4,416 Yds. / Day	11,888 Yds. / Day	25,962 Yds. / Day	47,922 Yds. / Day	81,935 Yds. / Day
81 Days / Year	$6.8x10^3$ Yds. / Year	$3.3x10^4$ Yds. / Year	$69.4x10^4$ Yds. / Year	$3.6x10^5$ Yds. / Year	$9.6x10^5$ Yds. / Year	$2.1x10^6$ Yds. / Year	$3.9x10^6$ Yds. / Year	$6.6x10^6$ Yds. / Year

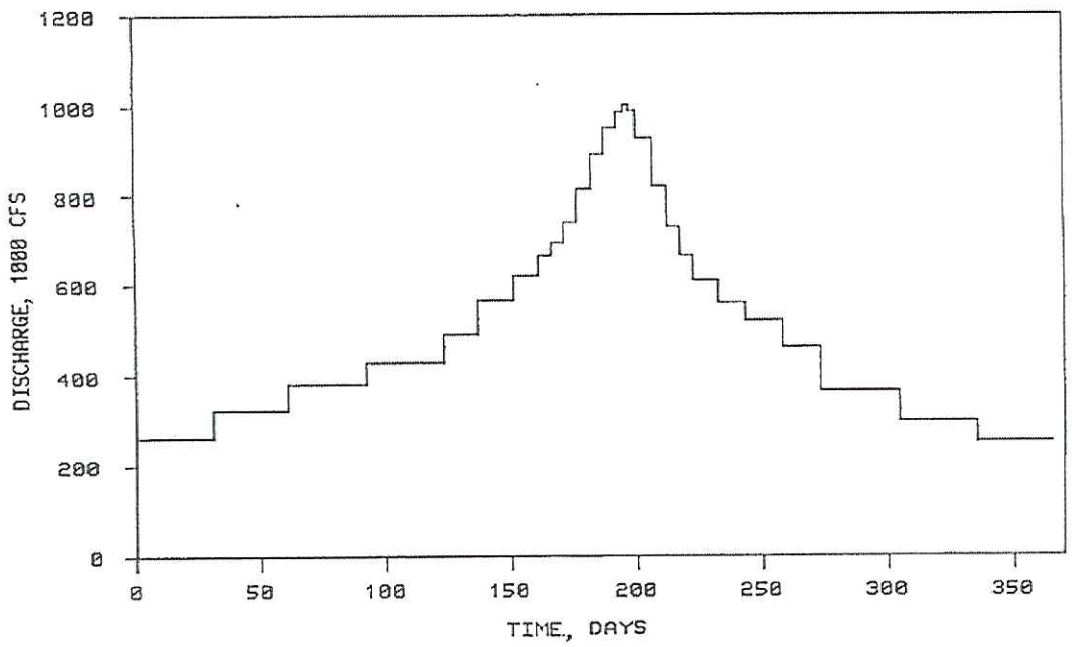
a. Basic 15,000 cfs Diversion Structure. Basic features of the design are shown on the drawings, Sheets 1 to 3 in Appendix B. The general layout of the plan is shown on Sheet 1 and extends from within the Mississippi River, some 830 feet at the upstream end and to an existing oil well access canal some 5,300 feet downstream from the control structure. Typical diversion channel cross-sections are shown on Sheet 2, and a typical elevation drawing of the gated control structure is shown on Sheet 3. The gates will be four (4) individual 35 feet wide by 10'6" high segments that can be dogged at any desired gate opening to facilitate diversion choices to enhance sediment diversion or freshwater mitigation. River works in the shape of an 830 foot deflection dike and free standing sheet pile wall, are included to enhance water and sediment entry into the diversion channel. The structure will have a single gate lifting crane to operate the gates.

b. Modular 70,000 cfs Diversion Structure. The ultimate size project structure is essentially an enlarged version of the basic design, in that many features/shapes/etc. are similar. Economics of scale would reduce the cost per cfs diverted, and the method of flume construction would change from a strut supported side wall configuration of the single gated plan to a flume not unlike the walls of a navigation lock chamber, but less expensive. See Sheets 4 and 5 in Appendix B for typical channel cross-sections. The deflection dike and sheet pile wall would be identical to the basic design as well as the features of the gated structure. There will be four (4) gate bays that will be identical to the single gated structure except that one (1) gate lifting crane will travel by rail from gate bay to gate bay to open or close any specified number of gates on a daily basis. See Sheet 3.

APPENDIX A

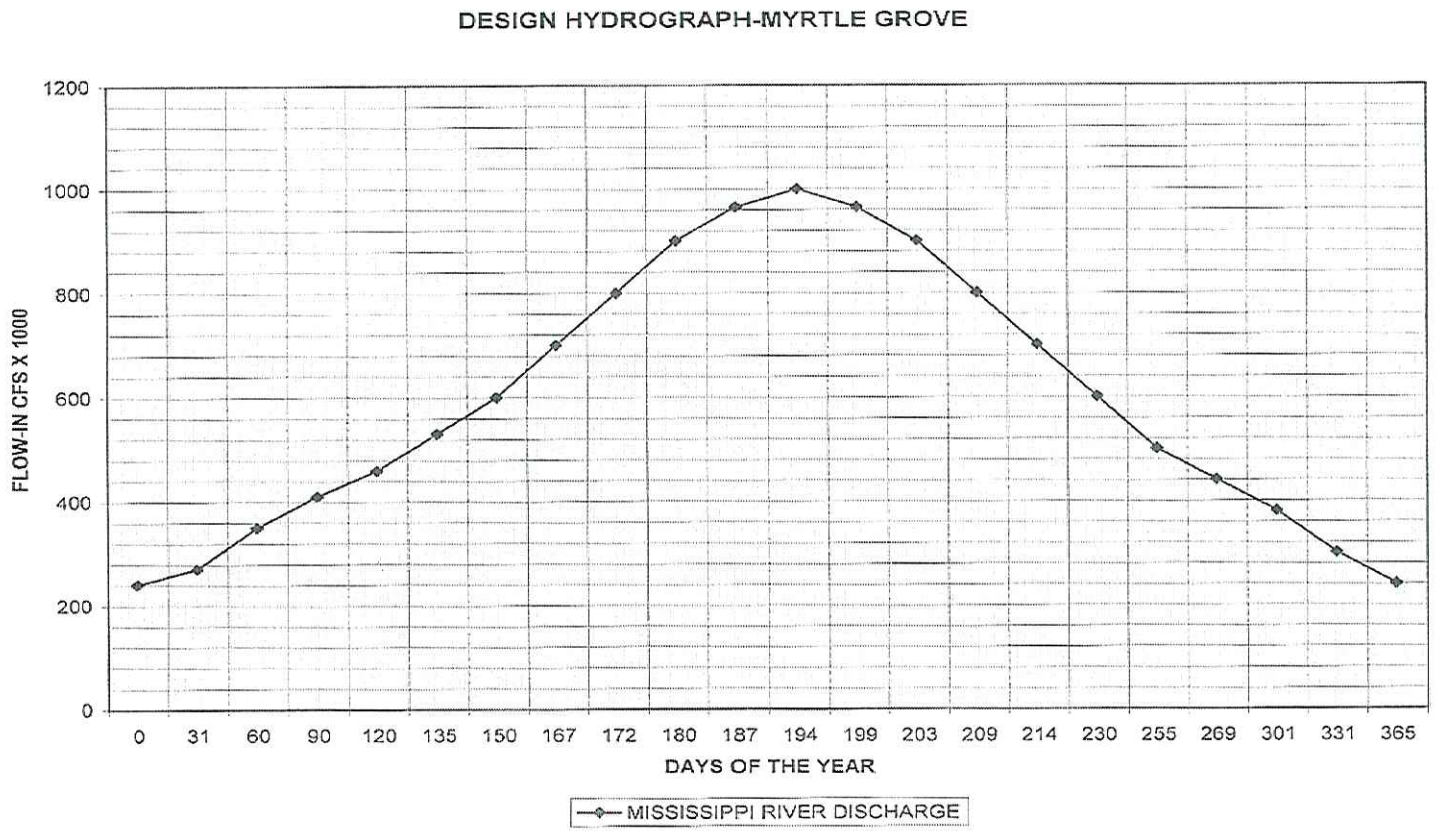
Attachments 1 to 18

PLATE 4

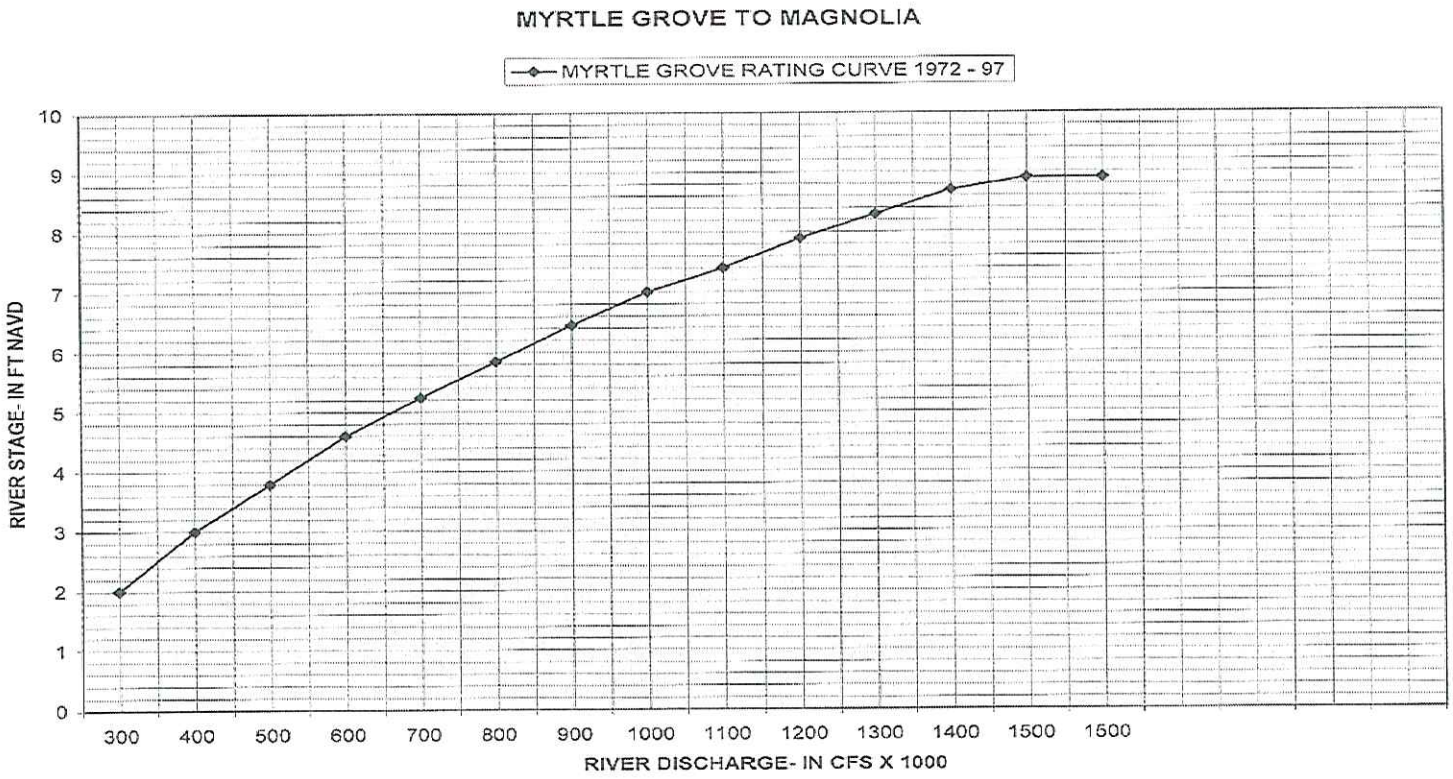


MISSISSIPPI RIVER
SHIFTED ANNUAL HISTOGRAPH

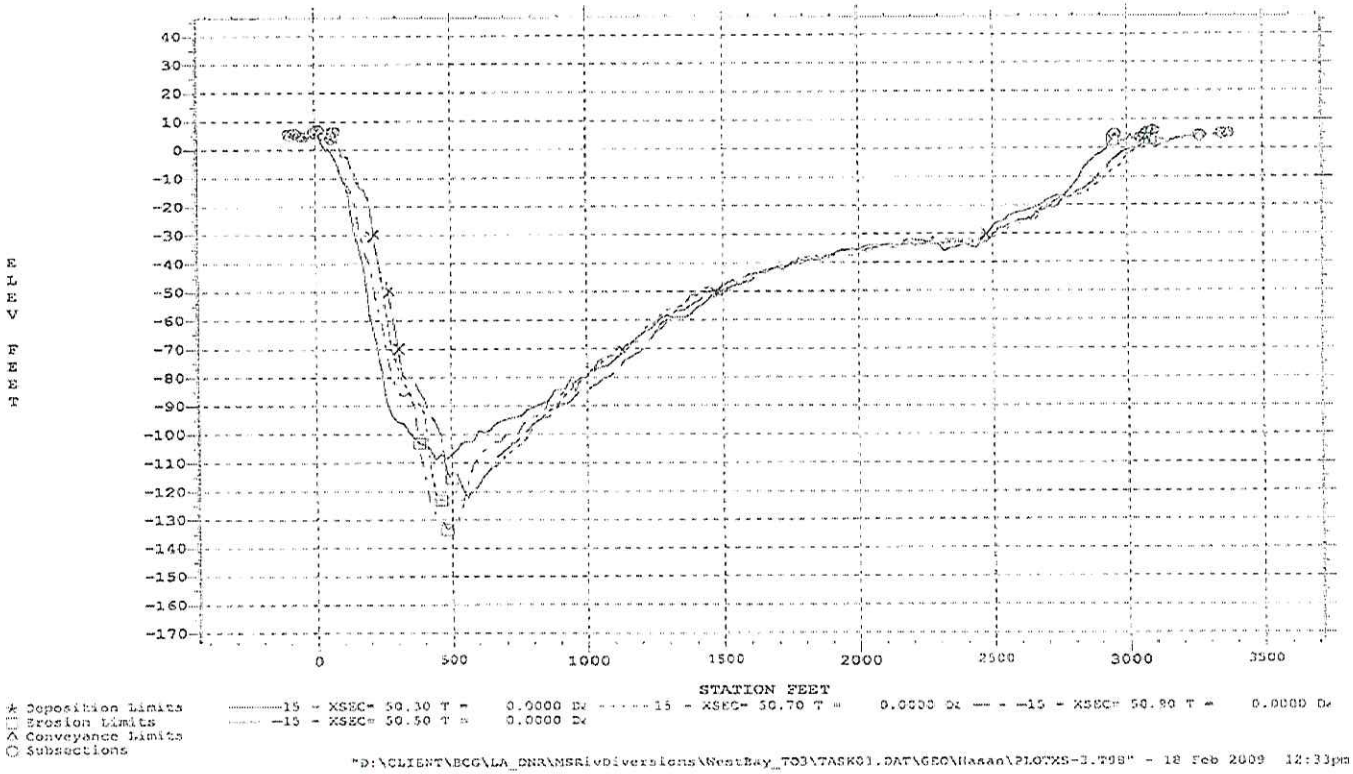
ATTACHMENT 1 – **MISSISSIPPI RIVER ANNUAL HISTOGRAPH OF DISCHARGE IN THE PROJECT REACH**



ATTACHMENT 2 – **DESIGN FLOW HYDROGRAPH FOR PRELIMINARY DESIGN ANALYSIS**

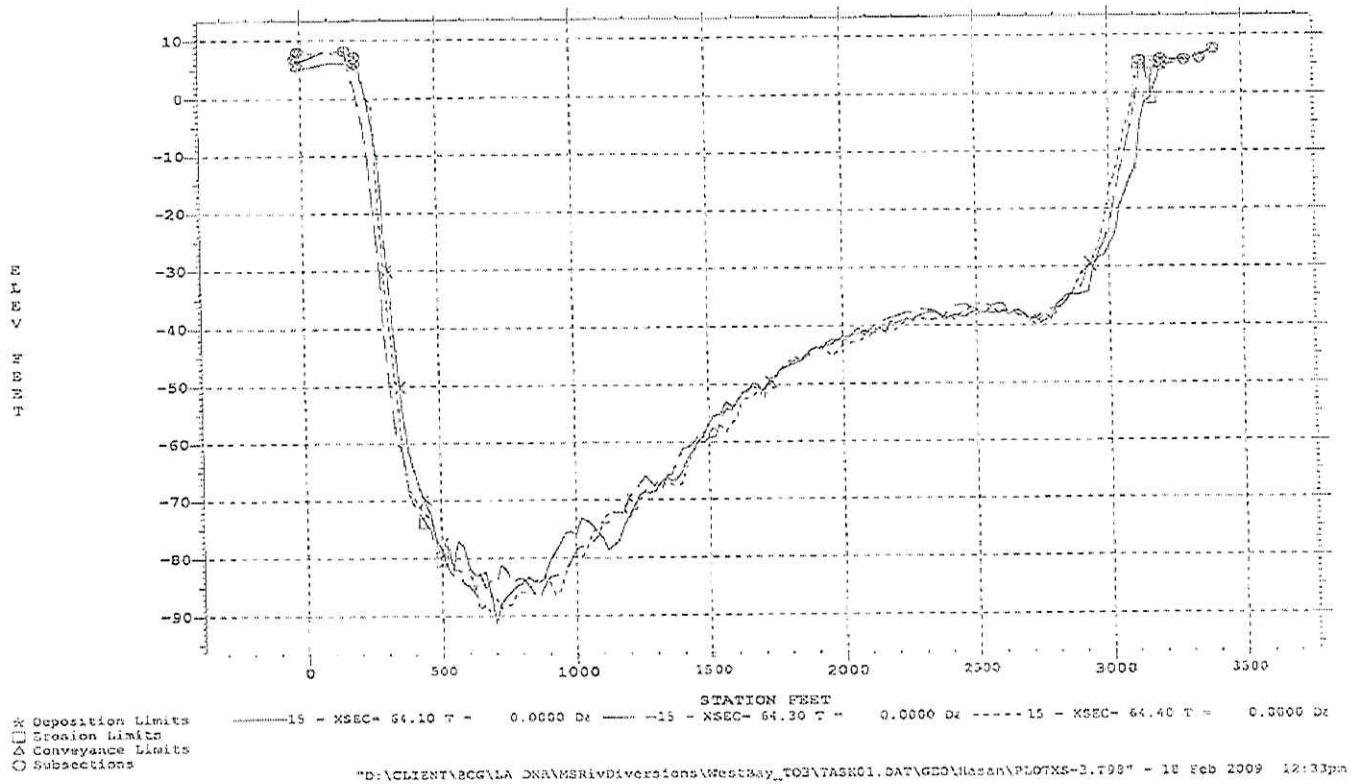


ATTACHMENT 3 – **AVERAGE MYRTLE GROVE RATING CURVE.**



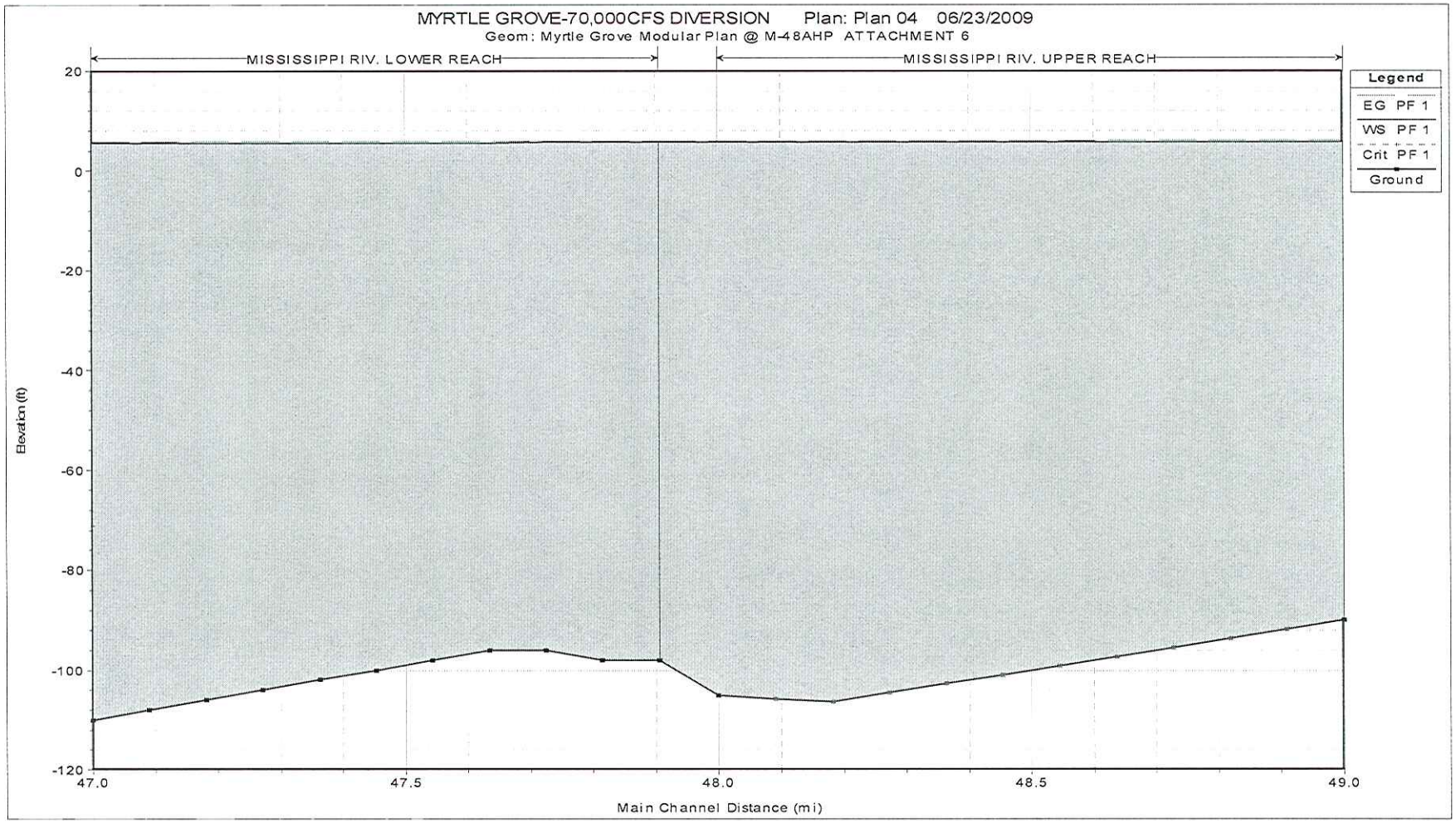
Comparative X-sections for determining relative stability of river at selected locations in study reach..

ATTACHMENT 4

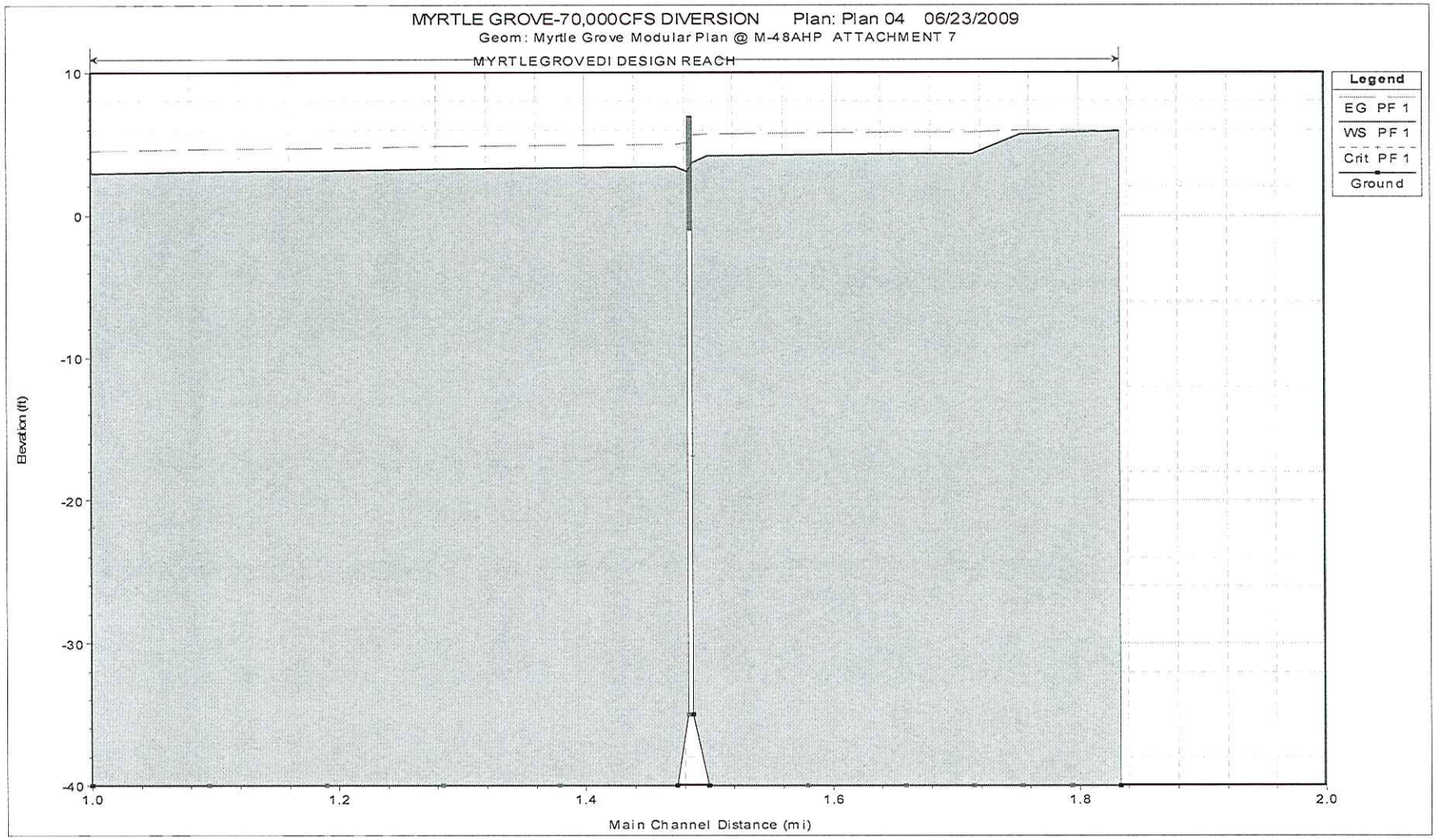


Comparative X-section No.2 for determining relative stability of river at selected locations.

ATTACHMENT 5



ATTACHMENT 6 – WATER SURFACE PROFILE IN MISSISSIPPI RIVER IN VICINITY OF MAGNOLIA, LA. FOR FLOW OF 750,000 CFS UPSTREAM AND 680,000 CFS DOWNSTREAM OF DIVERSION



ATTACHMENT 7 - WATER SURFACE PROFILE FOR 70,000CFS DIVERSION CHANNEL AT MILE 48 AHP

HEC-RAS Plan: Plan 06 Profile: PF 1

River	Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
MISSISSIPPI RIV.	LOWER REACH	47	PF 1	735000.00	-110.00	5.65	-69.66	5.91	0.000021	4.11	180036.70	3376.90	0.09
MISSISSIPPI RIV.	LOWER REACH	47.09	PF 1	735000.00	-108.00	5.66		5.92	0.000021	4.12	179225.80	3359.94	0.09
MISSISSIPPI RIV.	LOWER REACH	47.18	PF 1	735000.00	-106.00	5.67		5.93	0.000021	4.14	178431.30	3331.17	0.09
MISSISSIPPI RIV.	LOWER REACH	47.27	PF 1	735000.00	-104.00	5.68		5.94	0.000021	4.15	177665.60	3278.80	0.09
MISSISSIPPI RIV.	LOWER REACH	47.36	PF 1	735000.00	-102.00	5.69		5.95	0.000021	4.16	176927.60	3223.95	0.10
MISSISSIPPI RIV.	LOWER REACH	47.45	PF 1	735000.00	-100.00	5.69		5.96	0.000022	4.18	176224.20	3165.87	0.10
MISSISSIPPI RIV.	LOWER REACH	47.54	PF 1	735000.00	-98.00	5.70		5.98	0.000022	4.19	175555.10	3103.87	0.10
MISSISSIPPI RIV.	LOWER REACH	47.63	PF 1	735000.00	-96.00	5.71		5.99	0.000022	4.20	174924.40	3036.85	0.10
MISSISSIPPI RIV.	LOWER REACH	47.72	PF 1	735000.00	-96.00	5.72		6.00	0.000022	4.21	174730.60	2963.24	0.10
MISSISSIPPI RIV.	LOWER REACH	47.81	PF 1	735000.00	-98.00	5.74		6.01	0.000021	4.18	175985.90	2944.01	0.10
MISSISSIPPI RIV.	LOWER REACH	47.9	PF 1	735000.00	-98.00	5.75		6.02	0.000021	4.16	175995.70	2937.76	0.10
MISSISSIPPI RIV.	UPPER REACH	48	PF 1	750000.00	-105.00	5.76		6.03	0.000026	4.22	177722.80	3390.78	0.10
MISSISSIPPI RIV.	UPPER REACH	48.0909*	PF 1	750000.00	-105.79	5.77		6.04	0.000026	4.21	178323.00	3387.48	0.10
MISSISSIPPI RIV.	UPPER REACH	48.1818*	PF 1	750000.00	-106.36	5.78		6.06	0.000026	4.20	178478.90	3383.39	0.10
MISSISSIPPI RIV.	UPPER REACH	48.2727*	PF 1	750000.00	-104.55	5.79		6.07	0.000026	4.21	178057.30	3360.63	0.10
MISSISSIPPI RIV.	UPPER REACH	48.3636*	PF 1	750000.00	-102.73	5.80		6.08	0.000025	4.22	177584.20	3330.60	0.10
MISSISSIPPI RIV.	UPPER REACH	48.4545*	PF 1	750000.00	-100.91	5.82		6.09	0.000025	4.24	177061.40	3294.22	0.10
MISSISSIPPI RIV.	UPPER REACH	48.5454*	PF 1	750000.00	-99.09	5.83		6.11	0.000025	4.25	176498.00	3250.68	0.10
MISSISSIPPI RIV.	UPPER REACH	48.6363*	PF 1	750000.00	-97.27	5.84		6.12	0.000025	4.26	175905.30	3195.79	0.10
MISSISSIPPI RIV.	UPPER REACH	48.7272*	PF 1	750000.00	-95.45	5.85		6.13	0.000024	4.28	175295.40	3107.52	0.10
MISSISSIPPI RIV.	UPPER REACH	48.8181*	PF 1	750000.00	-93.64	5.86		6.14	0.000023	4.29	174708.40	2999.28	0.10
MISSISSIPPI RIV.	UPPER REACH	48.9090*	PF 1	750000.00	-91.82	5.87		6.16	0.000023	4.31	174179.40	2947.00	0.10
MISSISSIPPI RIV.	UPPER REACH	49.0	PF 1	750000.00	-90.00	5.88		6.17	0.000023	4.32	173629.00	2936.48	0.10
MYRTLEGROVEDI	DESIGN REACH	1.0	PF 1	15000.00	-40.00	3.40	-23.65	4.56	0.000373	8.64	1735.50	40.08	0.23
MYRTLEGROVEDI	DESIGN REACH	1.09*	PF 1	15000.00	-40.00	3.60		4.75	0.000360	8.60	1743.39	40.08	0.23
MYRTLEGROVEDI	DESIGN REACH	1.18*	PF 1	15000.00	-40.00	3.78		4.92	0.000327	8.57	1750.78	40.08	0.23
MYRTLEGROVEDI	DESIGN REACH	1.27*	PF 1	15000.00	-40.00	3.95		5.08	0.000324	8.53	1757.70	40.08	0.23
MYRTLEGROVEDI	DESIGN REACH	1.36*	PF 1	15000.00	-40.00	4.12		5.24	0.000287	8.50	1764.18	40.08	0.23
MYRTLEGROVEDI	DESIGN REACH	1.45	PF 1	15000.00	-40.00	4.27		5.38	0.000284	8.47	1770.24	40.08	0.22
MYRTLEGROVEDI	DESIGN REACH	1.5*	PF 1	15000.00	-35.00	4.06		5.49	0.000342	9.60	1561.97	40.08	0.27
MYRTLEGROVEDI	DESIGN REACH	1.515		Inl Struct									
MYRTLEGROVEDI	DESIGN REACH	1.53	PF 1	15000.00	-35.00	4.20	-18.65	5.62	0.000330	9.57	1567.71	40.08	0.27
MYRTLEGROVEDI	DESIGN REACH	1.55	PF 1	15000.00	-40.00	4.58		5.68	0.000248	8.41	1782.72	40.08	0.22
MYRTLEGROVEDI	DESIGN REACH	1.625*	PF 1	15000.00	-40.00	4.69		5.78	0.000246	8.39	1787.12	40.08	0.22
MYRTLEGROVEDI	DESIGN REACH	1.7*	PF 1	15000.00	-40.00	4.79		5.88	0.000245	8.37	1791.21	40.08	0.22
MYRTLEGROVEDI	DESIGN REACH	1.775*	PF 1	15000.00	-40.00	4.86		5.94	0.000214	8.36	1793.05	40.08	0.22
MYRTLEGROVEDI	DESIGN REACH	1.85*	PF 1	15000.00	-40.00	6.04		6.05	0.000001	0.85	17624.10	551.50	0.03
MYRTLEGROVEDI	DESIGN REACH	1.925*	PF 1	15000.00	-40.00	6.04		6.05	0.000000	0.70	21387.41	650.37	0.02
MYRTLEGROVEDI	DESIGN REACH	2.0	PF 1	15000.00	-40.00	6.05		6.05	0.000001	0.58	25993.78	750.39	0.02

ATTACHMENT 8 – **HEC-RAS TABULATION FOR A 15,000CFS DIVERSION PLAN- WATER SURFACE PROFILES**

HEC-RAS Plan: Plan 04 Profile: PF 1

River	Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
MISSISSIPPI RIV.	LOWER REACH	47	PF 1	680000.00	-110.00	5.65	-71.08	5.87	0.000019	3.80	180038.70	3376.90	0.09
MISSISSIPPI RIV.	LOWER REACH	47.09	PF 1	680000.00	-108.00	5.66		5.88	0.000018	3.81	179221.60	3359.94	0.09
MISSISSIPPI RIV.	LOWER REACH	47.18	PF 1	680000.00	-106.00	5.66		5.89	0.000018	3.83	178423.00	3330.94	0.09
MISSISSIPPI RIV.	LOWER REACH	47.27	PF 1	680000.00	-104.00	5.67		5.90	0.000018	3.84	177653.20	3278.53	0.09
MISSISSIPPI RIV.	LOWER REACH	47.36	PF 1	680000.00	-102.00	5.68		5.91	0.000018	3.85	176911.20	3223.47	0.09
MISSISSIPPI RIV.	LOWER REACH	47.45	PF 1	680000.00	-100.00	5.69		5.92	0.000018	3.86	176204.00	3165.25	0.09
MISSISSIPPI RIV.	LOWER REACH	47.54	PF 1	680000.00	-98.00	5.70		5.93	0.000019	3.88	175531.20	3103.10	0.09
MISSISSIPPI RIV.	LOWER REACH	47.63	PF 1	680000.00	-96.00	5.70		5.94	0.000019	3.89	174897.00	3035.90	0.09
MISSISSIPPI RIV.	LOWER REACH	47.72	PF 1	680000.00	-98.00	5.71		5.95	0.000019	3.89	174699.00	2962.08	0.09
MISSISSIPPI RIV.	LOWER REACH	47.81	PF 1	680000.00	-98.00	5.72		5.96	0.000018	3.86	175948.80	2943.93	0.09
MISSISSIPPI RIV.	LOWER REACH	47.9	PF 1	680000.00	-98.00	5.73		5.97	0.000018	3.80	175954.40	2937.00	0.09
MISSISSIPPI RIV.	UPPER REACH	48	PF 1	750000.00	-105.00	5.71		5.99	0.000026	4.22	177579.20	3390.46	0.10
MISSISSIPPI RIV.	UPPER REACH	48.0909*	PF 1	750000.00	-105.70	5.73		6.00	0.000026	4.21	178179.70	3387.13	0.10
MISSISSIPPI RIV.	UPPER REACH	48.1818*	PF 1	750000.00	-106.38	5.74		6.01	0.000026	4.21	178335.90	3383.00	0.10
MISSISSIPPI RIV.	UPPER REACH	48.2727*	PF 1	750000.00	-104.55	5.75		6.03	0.000026	4.22	177915.40	3357.66	0.10
MISSISSIPPI RIV.	UPPER REACH	48.3636*	PF 1	750000.00	-102.73	5.76		6.04	0.000026	4.23	177443.60	3327.40	0.10
MISSISSIPPI RIV.	UPPER REACH	48.4545*	PF 1	750000.00	-100.91	5.77		6.05	0.000025	4.24	176922.40	3290.73	0.10
MISSISSIPPI RIV.	UPPER REACH	48.5454*	PF 1	750000.00	-99.09	5.78		6.07	0.000025	4.25	176360.90	3246.63	0.10
MISSISSIPPI RIV.	UPPER REACH	48.6363*	PF 1	750000.00	-97.27	5.80		6.08	0.000025	4.27	175770.60	3191.50	0.10
MISSISSIPPI RIV.	UPPER REACH	48.7272*	PF 1	750000.00	-95.45	5.81		6.09	0.000024	4.28	175164.40	3100.99	0.10
MISSISSIPPI RIV.	UPPER REACH	48.8181*	PF 1	750000.00	-93.64	5.82		6.10	0.000023	4.30	174581.90	2997.40	0.10
MISSISSIPPI RIV.	UPPER REACH	48.9090*	PF 1	750000.00	-91.82	5.83		6.11	0.000023	4.31	174055.20	2947.64	0.10
MISSISSIPPI RIV.	UPPER REACH	49.0	PF 1	750000.00	-90.00	5.84		6.13	0.000023	4.32	173505.20	2938.25	0.10
MYRTLEGROVEDI	DESIGN REACH	1.0	PF 1	70000.00	-40.00	2.95	-21.90	4.56	0.000200	10.19	6871.47	160.08	0.27
MYRTLEGROVEDI	DESIGN REACH	1.09*	PF 1	70000.00	-40.00	3.06		4.66	0.000199	10.16	6888.85	160.08	0.27
MYRTLEGROVEDI	DESIGN REACH	1.18*	PF 1	70000.00	-40.00	3.16		4.76	0.000177	10.14	6905.14	160.08	0.27
MYRTLEGROVEDI	DESIGN REACH	1.27*	PF 1	70000.00	-40.00	3.26		4.84	0.000176	10.11	6920.46	160.08	0.27
MYRTLEGROVEDI	DESIGN REACH	1.36*	PF 1	70000.00	-40.00	3.35		4.93	0.000156	10.09	6934.86	160.08	0.27
MYRTLEGROVEDI	DESIGN REACH	1.45	PF 1	70000.00	-40.00	3.43		5.01	0.000155	10.07	6948.37	160.08	0.27
MYRTLEGROVEDI	DESIGN REACH	1.5*	PF 1	70000.00	-35.00	3.11		5.16	0.000200	11.46	6097.03	160.07	0.33
MYRTLEGROVEDI	DESIGN REACH	1.515		Inl Struct									
MYRTLEGROVEDI	DESIGN REACH	1.53	PF 1	70000.00	-35.00	3.67	-16.90	5.65	0.000192	11.32	6185.93	160.08	0.32
MYRTLEGROVEDI	DESIGN REACH	1.55	PF 1	70000.00	-40.00	4.19		5.71	0.000131	9.99	7069.70	160.08	0.26
MYRTLEGROVEDI	DESIGN REACH	1.625*	PF 1	70000.00	-40.00	4.25		5.77	0.000130	9.89	7079.33	160.08	0.26
MYRTLEGROVEDI	DESIGN REACH	1.7*	PF 1	70000.00	-40.00	4.30		5.82	0.000114	9.88	7088.17	160.08	0.26
MYRTLEGROVEDI	DESIGN REACH	1.775*	PF 1	70000.00	-40.00	4.34		5.85	0.000114	9.87	7093.89	160.08	0.26
MYRTLEGROVEDI	DESIGN REACH	1.85*	PF 1	70000.00	-40.00	5.74		5.99	0.000017	4.01	17457.32	549.69	0.13
MYRTLEGROVEDI	DESIGN REACH	1.925*	PF 1	70000.00	-40.00	5.83		6.00	0.000010	3.29	21247.66	648.94	0.10
MYRTLEGROVEDI	DESIGN REACH	2.0	PF 1	70000.00	-40.00	5.89		6.00	0.000010	2.71	25877.70	749.36	0.08

ATTACHMENT 8A – **HEC-RAS TABULATION FOR A 70,000CFS DIVERSION PLAN WATER SURFACE PROFILES**

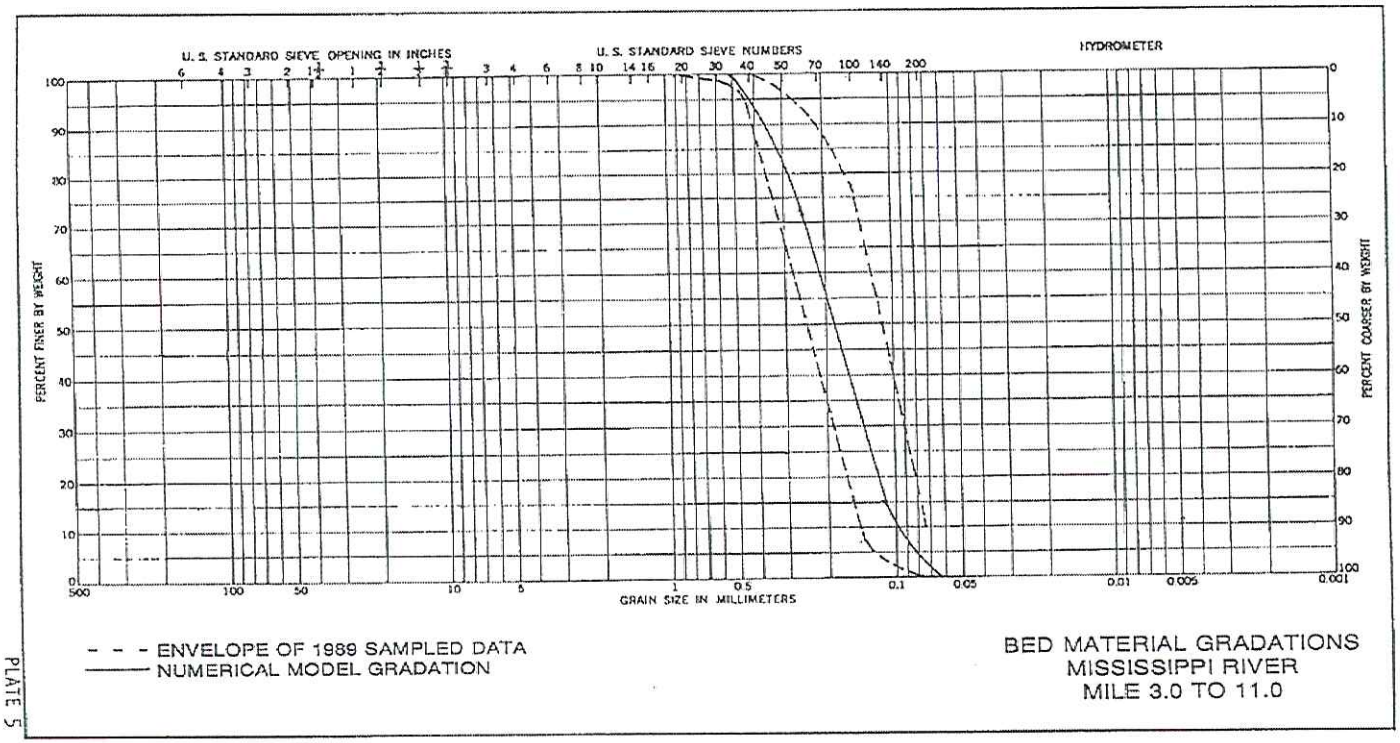


PLATE 5

ATTACHMENT 9 – **TYPICAL BED MATERIAL GRAIN SIZES IN THE LOWER MISSISSIPPI RIVER**

SandCalc - [Total Load - Currents - van Rijn (formula)]

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

22/06/09 15:00:29

OUTPUT PARAMETERS	
total volumetric sediment transport rate in current direction (m^2/s)	1.526E-05
INPUT PARAMETERS	
water depth (m)	11.6
kinematic viscosity of water (m^2/s)	1.191E-06
ratio of densities of grain and water (-)	2.583
median grain diameter d50 (mm)	.18
d90 grain diameter (mm)	.3497
depth-averaged current speed (m/s)	.61

ATTACHMENT 10

UNIT TRANSPORT RATE AT A VELOCITY OF 2 FPS

SandCalc - [Total Load - Currents - van Rijn (formula)]

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

22/06/09 14:51:58

OUTPUT PARAMETERS

total volumetric sediment transport rate in current direction (m^2/s)	.0000744

INPUT PARAMETERS

water depth (m)	11.6
kinematic viscosity of water (m^2/s)	1.191E-06
ratio of densities of grain and water (-)	2.583
median grain diameter d50 (mm)	.18
d90 grain diameter (mm)	.3497
depth-averaged current speed (m/s)	.762

ATTACHMENT 11

UNIT TRANSPORT RATE AT A VELOCITY OF 2.5 FPS

SandCalc

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Total Load - Currents - van Rijn (formula)

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

22/06/09 14:27:46

OUTPUT PARAMETERS

total volumetric sediment transport rate in current direction (m^2/s)	.0002117

INPUT PARAMETERS

water depth (m)	11.6
kinematic viscosity of water (m^2/s)	1.191E-06
ratio of densities of grain and water (-)	2.583
median grain diameter d50 (mm)	.18
d90 grain diameter (mm)	.3497
depth-averaged current speed (m/s)	.914

ATTACHMENT 12

UNIT TRANSPORT RATE AT A VELOCITY OF 3 FPS

SandCalc - [Total Load - Currents - van Rijn (formula)]

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

23/06/09 08:00:57

OUTPUT PARAMETERS	
total volumetric sediment transport rate in current direction (m^2/s)	.000803

INPUT PARAMETERS	
water depth (m)	12.8
kinematic viscosity of water (m^2/s)	1.157E-06
ratio of densities of grain and water (-)	2.65
median grain diameter d50 (mm)	.18
d90 grain diameter (mm)	.3
depth-averaged current speed (m/s)	1.22

ATTACHMENT 13

UNIT TRANSPORT RATE AT A VELOCITY OF 4 FPS.

SandCalc - [Total Load - Currents - van Rijn (formula)]

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

23/06/09 08:08:24

OUTPUT PARAMETERS

total volumetric sediment transport rate in current direction (m^2/s)	.00868

INPUT PARAMETERS

water depth (m)	12.8
kinematic viscosity of water (m^2/s)	1.157E-06
ratio of densities of grain and water (-)	2.65
median grain diameter d50 (mm)	.18
d90 grain diameter (mm)	.3
depth-averaged current speed (m/s)	2.13

ATTACHMENT 16

UNIT TRANSPORT RATE AT A VELOCITY OF 7 FPS

SandCalc - [Total Load - Currents - van Rijn (formula)]

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

23/06/09 08:09:44

OUTPUT PARAMETERS

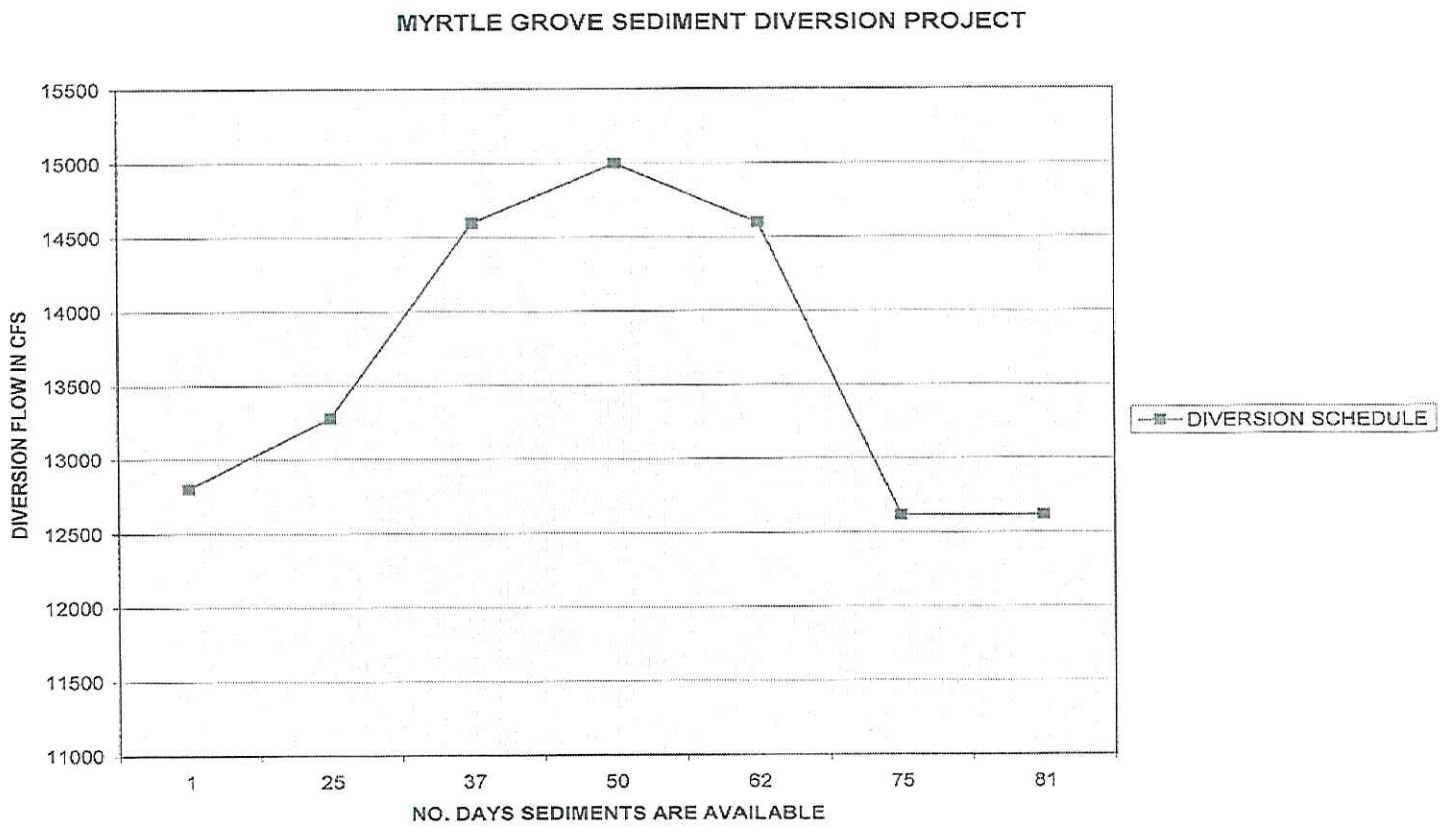
total volumetric sediment transport rate in current direction (m^2/s)	.01483

INPUT PARAMETERS

water depth (m)	12.8
kinematic viscosity of water (m^2/s)	1.157E-06
ratio of densities of grain and water (-)	2.65
median grain diameter d50 (mm)	.18
d90 grain diameter (mm)	.3
depth-averaged current speed (m/s)	2.44

ATTACHMENT 17

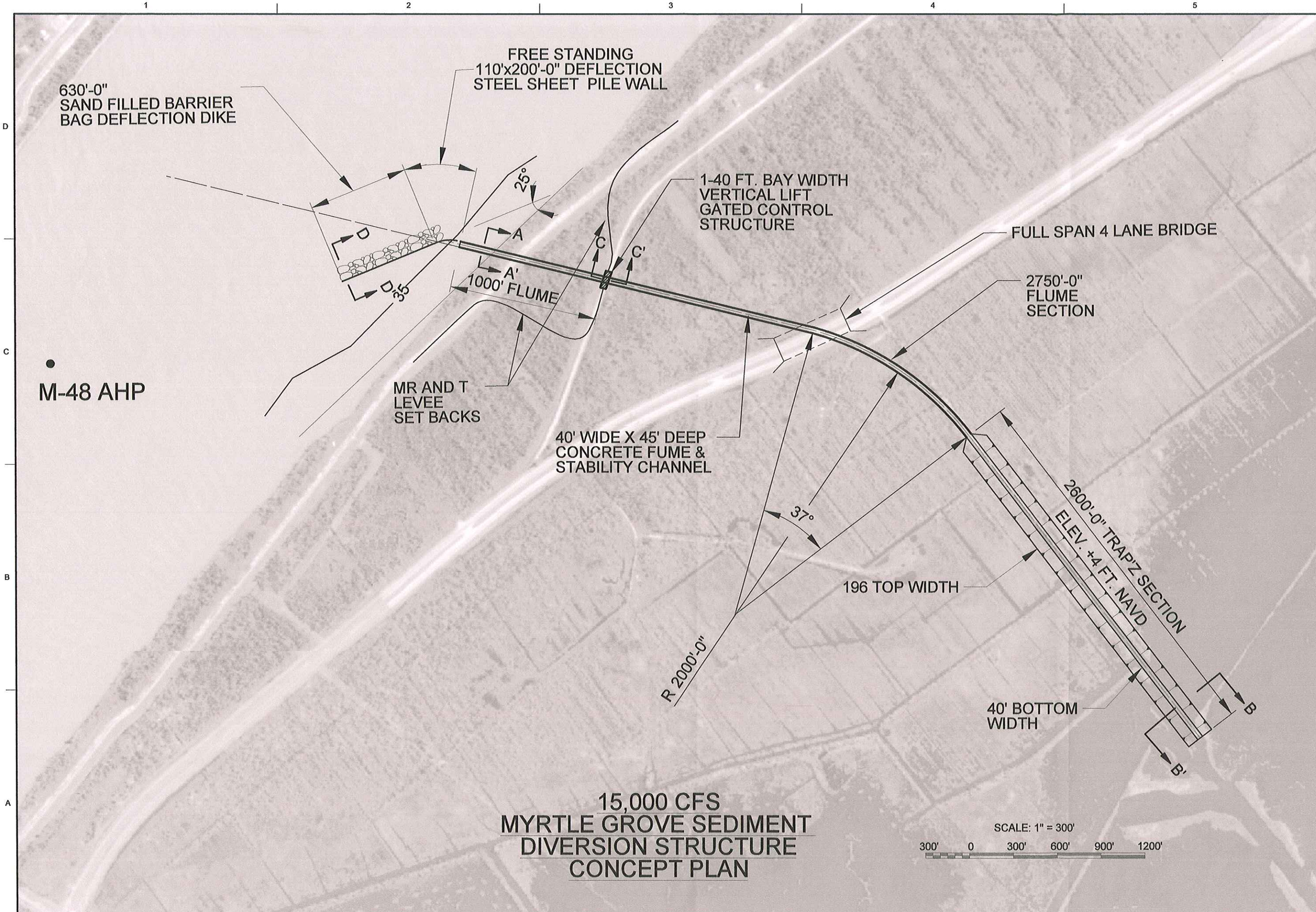
UNIT TRANSPORT RATE AT A VELOCITY OF 8 FPS



ATTACHMENT 18 – **PLOT OF DIVERSION FLOW VERSUS DAYS FLOW EXCEEDS 600,000CFS IN MISSISSIPPI.**

APPENDIX B

Drawings 1 to 5



MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: Stephen Gibson
DATE: JULY, 2009
DWN BY: BCG
CKD BY: SM
SUBMITTED BY: Rodney J. Germuch
PLOT SCALE: 1" = 300'
PLOT DATE: JULY, 2009
SIZE: ANSI D
FILE NAME: FIG 1.DWG

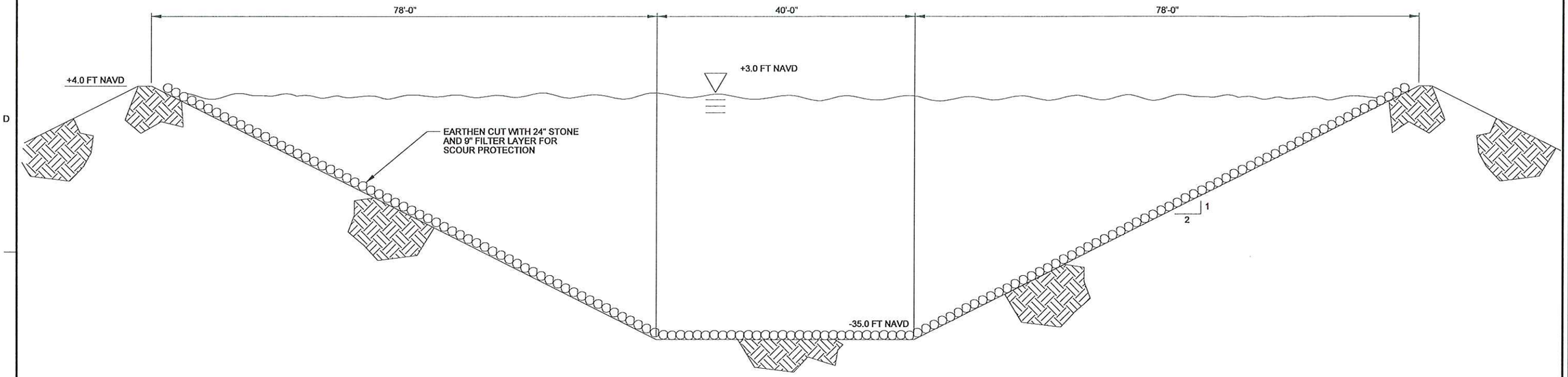
COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

CONCEPTUAL PLAN

SHEET IDENTIFICATION
1



TYPICAL SECTION B-B'

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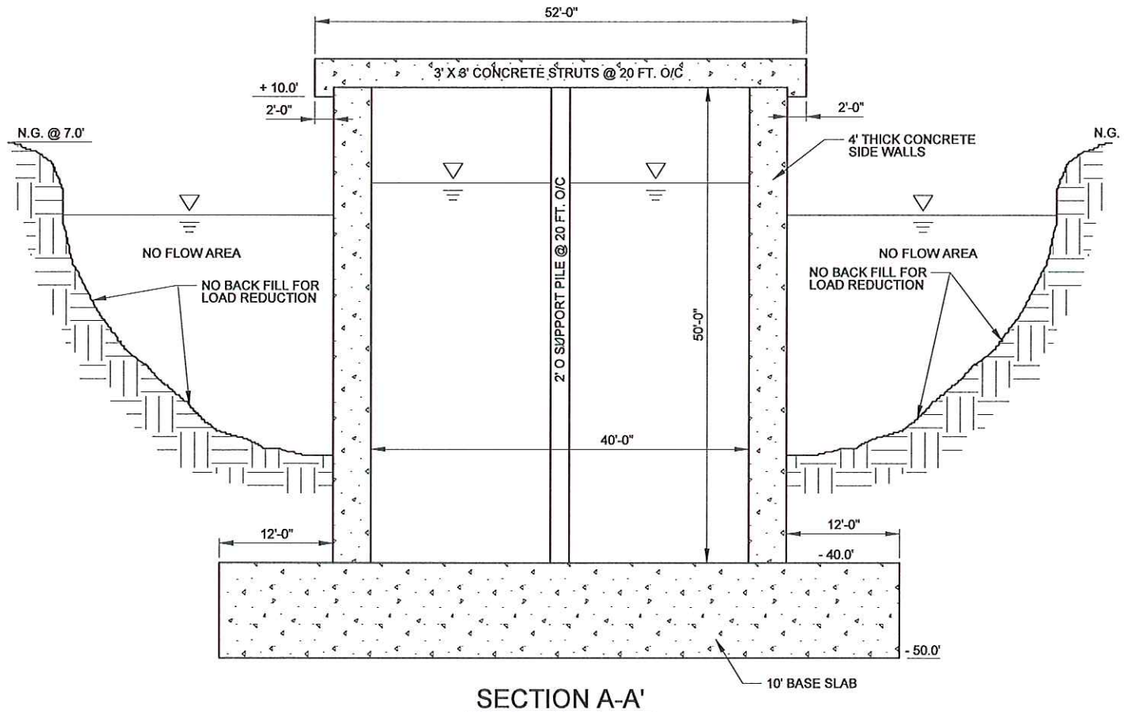

SECTION A-A'

40' X 45' DIVERSION CANAL BETWEEN MISSISSIPPI RIVER AND CONTROL STRUCTURE

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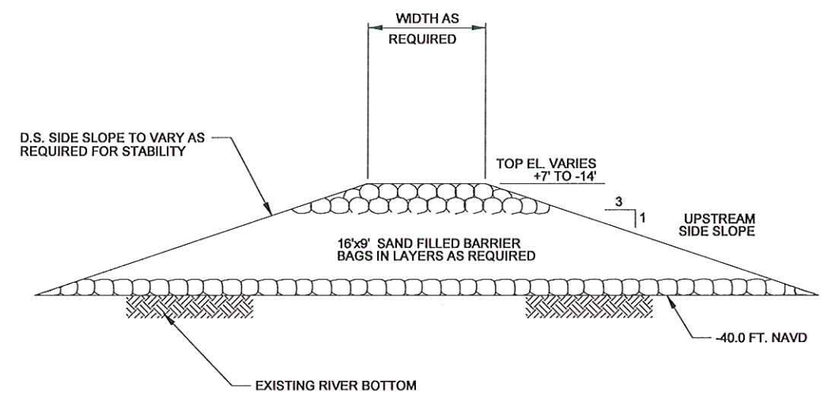

TYPICAL SECTION D-D'

N.T.S.

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Sudhir Mehta
DATE: JULY, 2009
DWN BY: SCS
CKD BY: SM
SUBMITTED BY: Rodney J. Germuch
PLOT SCALE: NTS
PLOT DATE: JULY, 2009
SIZE: ANSI D
FILE NAME: FIG 2 40X45 DIVERSION CANAL

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MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

TYPICAL SECTION

SHEET IDENTIFICATION
2

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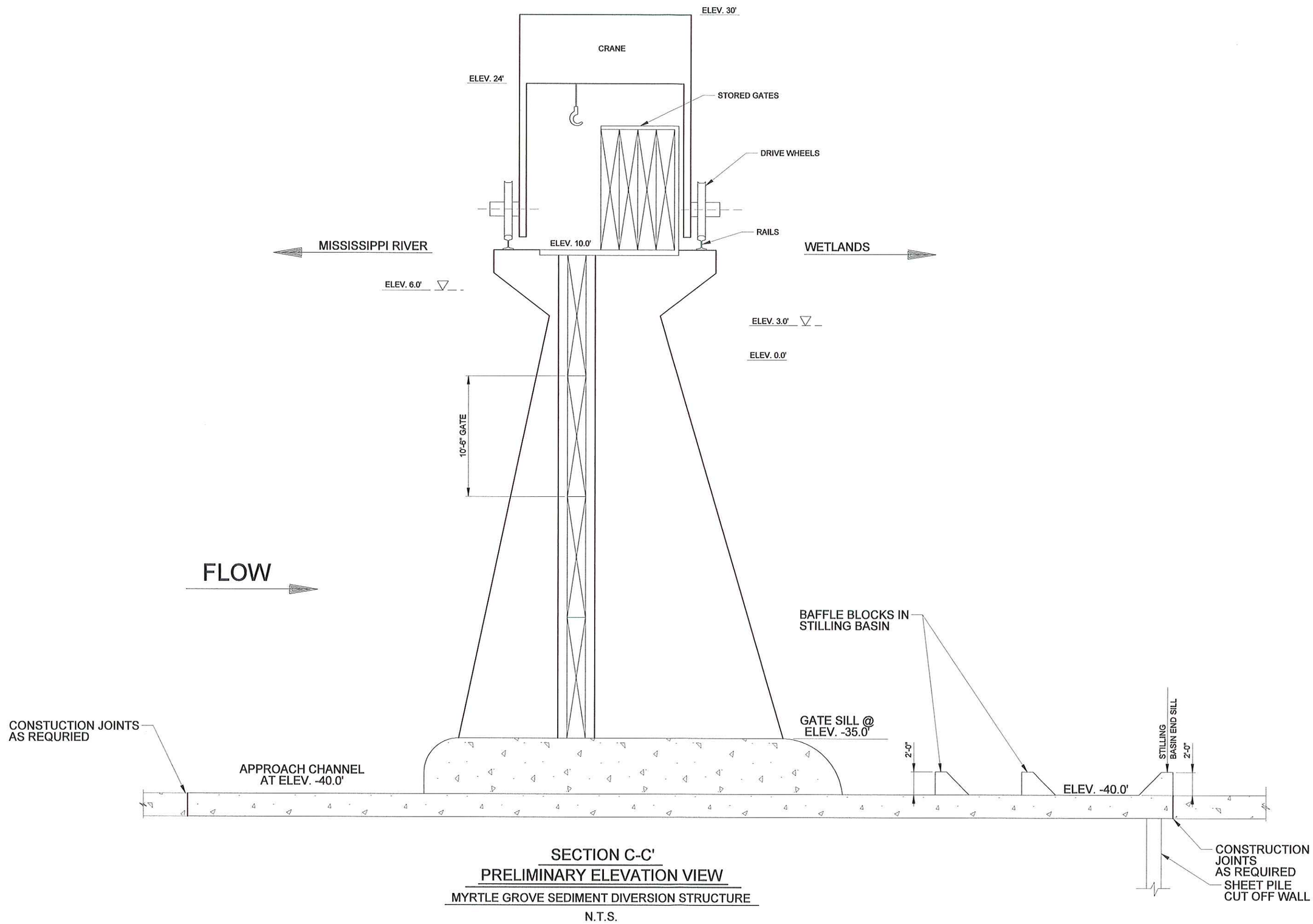

SECTION C-C'
PRELIMINARY ELEVATION VIEW
MYRTLE GROVE SEDIMENT DIVERSION STRUCTURE

N.T.S.

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Sergio Maria
DATE: JULY, 2009
DWN BY: BCG
CKD BY: SM
SUBMITTED BY: Rodney J. Gernuch
PLOT SCALE: 1/4"= 1'-0"
PLOT DATE: JULY, 2009
SIZE: ANSI D
FILE NAME: FIG 3

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MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

TYPICAL SECTION

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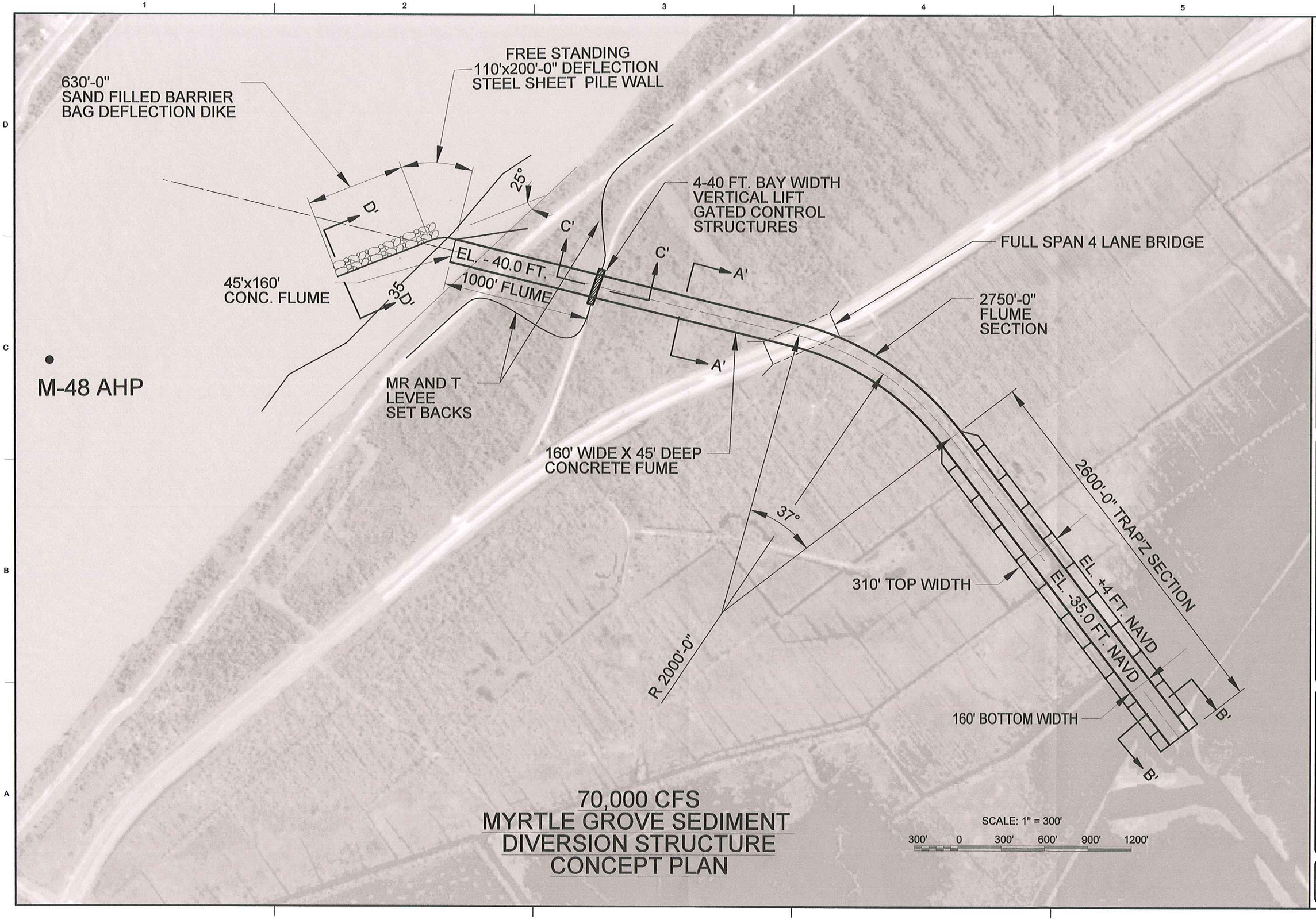

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Sudhir Malla
DWN BY: BCG
CKD BY: SM
SUBMITTED BY: Rodney J. Gernuch
PLOT SCALE: 1" = 300'
PLOT DATE: JULY 2009
SIZE: ANSI D
FILE NAME: FIG-4
DATE: JULY, 2009

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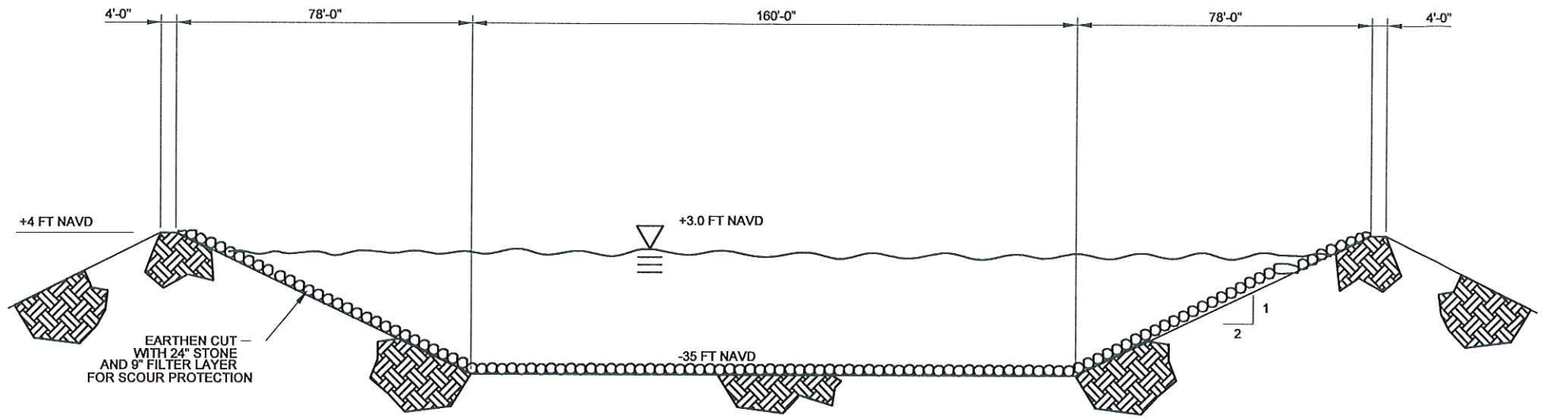
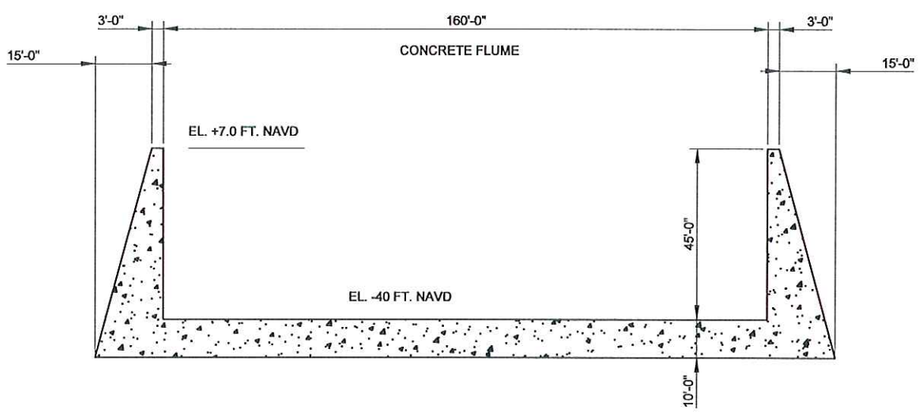
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

CONCEPTUAL PLAN

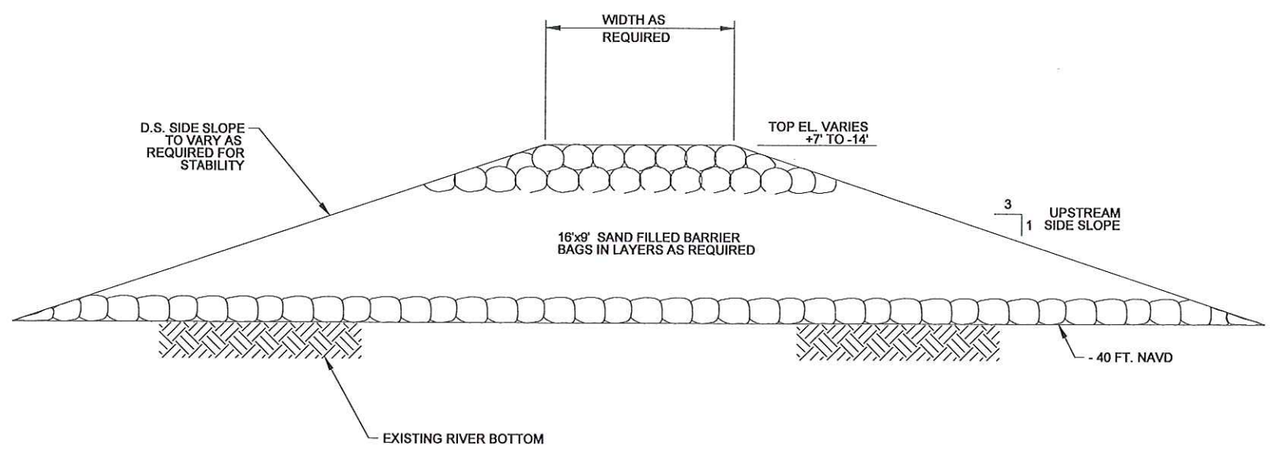
SHEET IDENTIFICATION
4



TYPICAL SECTION B-B'



TYPICAL SECTION A-A'



TYPICAL SECTION D-D'

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Randall Mills
DATE: JULY, 2009
DWN BY: BDD
CKD BY: SM
SUBMITTED BY: Rodney J. Germach
PLOT SCALE: 1/2"=1'-0"
PLOT DATE: JULY, 2009
SIZE: ANSI D
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MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

TYPICAL SECTIONS

SHEET IDENTIFICATION
5

...\FIG 5_FLUME SECTIONS.dgn 6/25/2009 8:48:03 AM

APPENDIX C

CONCEPTUAL DESIGNS

MYRTLE GROVE SEDIMENT DIVERSION STRUCTURE

RIVER MILE 60.2 AHP

VOLUME II

REVISED

PREPARED
FOR

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BY:

BCG ENGINEERING & CONSULTING, INC.
3012 26TH STREET
METAIRIE, LA 70002

MARCH 2011

CONCEPTUAL DESIGNS

MYRTLE GROVE SEDIMENT DIVERSION STRUCTURE RIVER MILE 60.2 AHP

VOLUME II

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VOLUME II

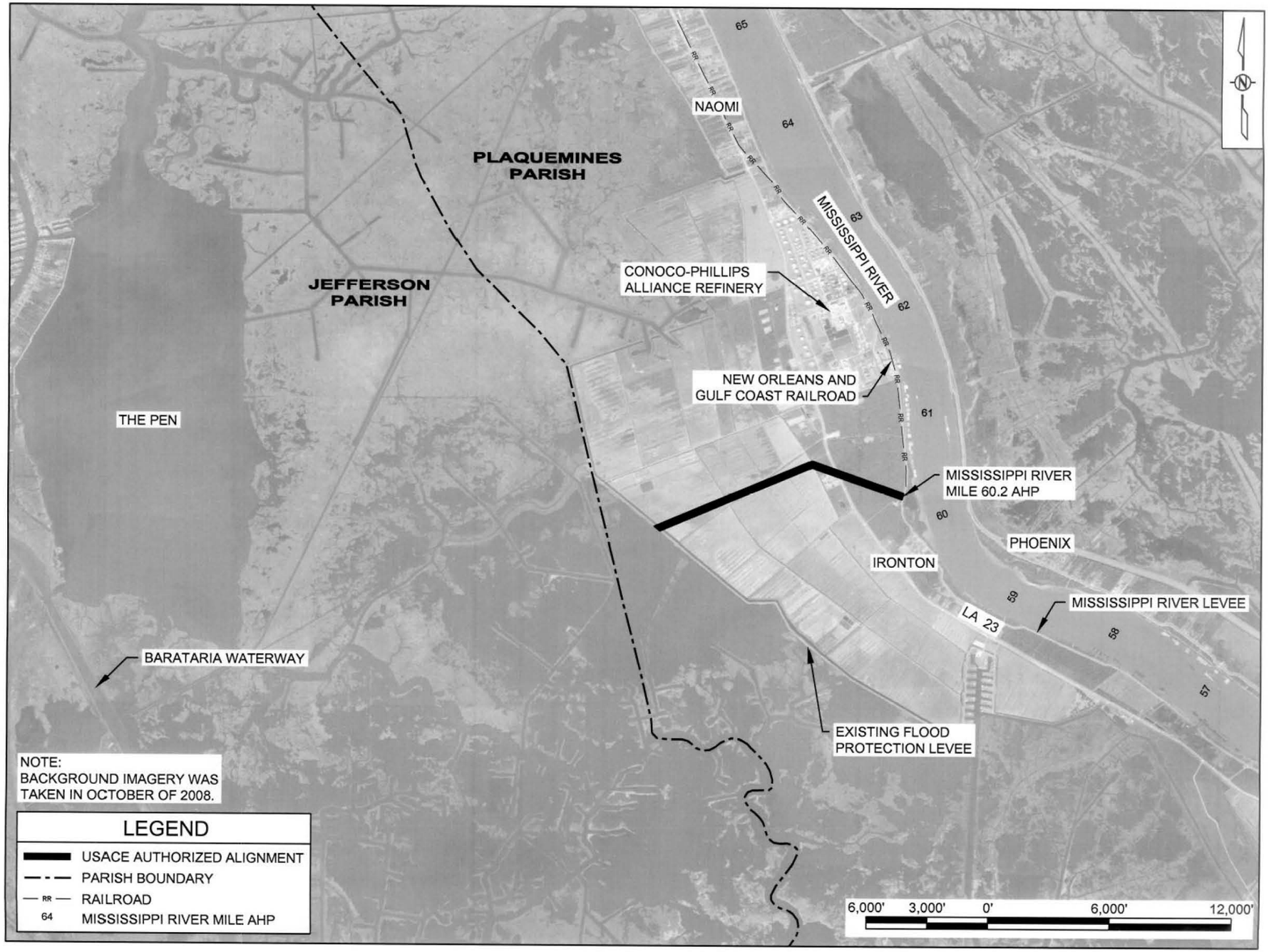
CONCEPTUAL DESIGNS

MYRTLE GROVE SEDIMENT DIVERSION STRUCTURE

RIVER MILE 60.2 AHP

1. GENERAL. This project study was authorized by the Office of Coastal Protection and Restoration, of the State of Louisiana, by letter and notice to proceed dated December 21, 2009 under Task Order 2, Contract No. 2503-10-35. The purpose of the study is to determine feasibility of a sediment and freshwater diversion project in the vicinity of Myrtle Grove, Louisiana, for the purpose of land building and marsh creation using sediments available from the Mississippi River, and to quantify the benefits and impacts related to the project. The purpose of this report is to provide essential features of the structural means needed to cause a successful project outcome irrespective of the ultimate peak capacity chosen by applicable authority.

2. PROJECT SETTING AND BASIC ASSUMPTIONS. This project would be cited in Plaquemines Parish, at a point between Ironton on the south, the Alliance Refinery on the north, the Mississippi River on the east, and Barataria Bay and Bayou DuPont on the west at the discharge end of the diversion project. See the Vicinity Map, Page 2. The diversion channel would cross primarily pasture lands and timber land near the Mississippi River. These properties range in elevations from +7.0 feet near the Mississippi River to -3 feet NAVD88 near the back protection levee at the west end. Important considerations in site selection include extent of existing infrastructure, likely cost of relocations, and social and transportation impacts of any choice. The authorized alignment, shown on Page 2, has significant handicaps in that it is not the preferred location for several reasons. First is the acute angle the conveyance channel makes with the River at the river bank, and the greater depth at this location, both of which do not allow for a hydraulically efficient entrance channel, and the acute angle of the transition channel into the existing barrow pits, and secondly, because the diversion channel is so long, i.e., more than 12,000 feet in length the hydraulic gradient is greatly reduced and will affect any sediment carrying potential this site might provide. These characteristics completely negate any advantages of this alignment, as an alternative to the River Mile (RM) 48 Above Head of Passes (AHP) site presented in Volume I of this series of Conceptual Designs, for an efficient water and sediment diversion and land building project. The high sill alternative, presented in Volume I, was not considered for this location due to the vast requirement for real estate. This effort, presented in Volume II, was carried out based upon the basic assumption that this Diversion Project could be located in the vicinity of Myrtle Grove at a point on the river where the highest potential for successful sediment diversion and land building could be found. Under consideration is a site at RM 60.7. These conceptual designs, for a 15,000 cfs, 45,000 cfs, and 75,000 cfs diversion are being applied to the vicinity of RM 60.7 AHP because that is the currently authorized location.



Vicinity Map

3. BASIC HYDROLOGY OF MISSISSIPPI RIVER.

a. References.

1) "*Dredging Alternatives Study, Cubits Gap, Lower Mississippi River*", dated January 1991, by Ronald R. Copeland, PhD., the Waterways Experiment Station, Vicksburg, Mississippi.

2) "*Report on Feasibility of Small Scale Physical Model of Lower Mississippi River Delta for Testing Water and Sediment Diversion Projects*", dated December 2004, by BCG Engineering & Consulting, Inc., Coastal Restoration Consultants, and Louisiana State University, Baton Rouge, Louisiana.

3) "Shoaling *Implications of West Bay Sediment Diversion Project on Pilottown An*chorage", dated June 2009, by BCG Engineering & Consulting, Inc., for the Coastal Protection and Restoration Authority, State of Louisiana.

4) "Information Paper, MCR/LMVD Task Group Study of Navigation and Freshwater Diversion Projects, Baton Rouge to the Gulf", dated October 1985, by the U. S. Army Corps of Engineers, Vicksburg, Mississippi.

5) "*Mississippi River Sediment, Nutrient, and Freshwater Redistribution Study*", dated July 2000, by the Louisiana Coastal Wetlands Conservation & Restoration Task Force, for the U.S. Army Corps of Engineers, New Orleans District.

6) "*The Transport of Sediment by Gravity for Coastal Restoration and Wetland Rehabilitation in Louisiana*", dated September 15, 1997 by the University of New Orleans, Sogreah Engineering Institute, and Coastal Restoration Consultants, LLC, Baton Rouge, Louisiana.

b. Historical Peak Flows. Table 1 is a table of measured annual peak discharges made on specified dates at Tarbert's Landing, Louisiana on the Mississippi River for the period 1982 to 2009. Peak flows for 1983, 1997, and 2008 were modified at the latitude of Myrtle Grove to no more than 1,250,000 cfs by a diversion to Lake Pontchartrain of excess flow above 1,250,000 cfs. The average peak flow for the period shown is 1,092,000 cfs at Tarbert's Landing and 1,067,000 cfs at Myrtle Grove. However, unmeasured leakages through Bonnet Carré Floodway diminish the peak. The long term flow hydrograph, averages 1939 to 2009, at Tarbert Landing on the Mississippi River are shown on Attachment 1 in Appendix A.

c. Average Flow Hydrograph. The shifted annual histograph taken from Plate 4 in Reference 1) is a stair step plot of flows that was used to drive a 1-D numerical model of the Mississippi River in the 1991 study. See a copy of Plate 4, Attachment 2, in Appendix A. This histogram, plus adjustment to the average peak flow at Myrtle Grove to account for leakage at Bonnet Carré, and design diversions at Davis Pond and Carnarvon, was used to generate the primary design flow hydrograph at the latitude of the Myrtle Grove Sediment Diversion Structure. That flow hydrograph is shown at Attachment 3 in Appendix A.

d. Average Rating Curve. The River level within the reach, RM 64 AHP to RM 44 AHP, must be known for any given design discharge, in order to size the opening of a diversion structure to be able to convey water and sediments out of the river into the waters of Barataria Bay. See recorded river stage hydrographs in the reach for 2008 and 2009, Attachment 1, Appendix A.

However, the water surface of the lower reach of the Mississippi River below the latitude of the Bonnet Carré Floodway is greatly influenced by tides in the Gulf of Mexico. Consequently, a general plot of stage at any given time versus flow will produce a wide range of scattered points within a range of two (2) feet vertically over time. Such a scatter of points is useless and an average of the points must be plotted with associated flows in order to produce a Rating Curve at Myrtle Grove. Such a rating curve is given at Attachment 4.

4. BASIC HYDRAULIC ANALYSES. Any analysis of a long-term riverine project should include a Potamology Study to establish how the river has been behaving in the recent past in order to avoid unwanted changes or remediation in the future, particularly if the project is of notable importance and is likely to have a long life cycle, say 100 years, for example. In the case of the Mississippi River, this is of extreme importance due to the expense associated with remediation and the requirement for flood control and navigation safety. Fortunately, the Revetment Program provided by the U.S. Army Corps of Engineers is providing long-term stability in this study reach. Attachments 5 and 6 demonstrate the long-term stability of two typical cross-sections of the Mississippi River at RMs 50 and 64 AHP as provided by Mobile Boundary Hydraulics, PLLC, which were compared to historical hydrographic surveys in the 1980's at those same locations. Another example of long term channel stability is shown in Appendix B in bed profile plot for 1935 and 1992, between RM 62.5 and RM 55.0. See Sheet 1. Just as important are the considerations that must be given to the design of a long-term diversion structure and conveyance channel. Adequate, mechanical control, scour protection, and sediment transport efficiency are paramount to a safe, productive land building project. Likewise, changes in water surface elevations in the river, and in the receiving water bodies, as well as the local communities, must be quantified to avoid negative benefits, such as changes in local drainage or water supply, i.e., salinity levels, etc. In order to quantify these impacts, hydraulic studies must be made in conjunction with known hydrology to assess the changes that will occur over the life of the diversion. For example, if the project is completely successful, depositional effects will change the tail water levels in the receiving water body, and reduce the efficiency of the diversion structure over time. To offset such impacts, the structure must be sized to meet future expectations.

TABLE 1

ANNUAL PEAK DISCHARGES AT TARBERT'S LANDING, LOUISIANA

YEAR OF FLOW EVENT	Q IN 1,000 CFS	MONTH PEAK OCCURRED
1982	873	APR.
1983	1470	JUN.
1984	1,199	MAY
1985	1,128	MAR.
1986	822	OCT.
1987	970	MAR.
1988	1,000	JAN.
1989	1,138	MAR.
1990	1,230	JUN.
1991	1,303	JAN.
1992	770	APR.
1993	1,190	APR.
1994	1,164	MAY

YEAR OF FLOW EVENT	Q IN 1,000 CFS	MONTH PEAK OCCURRED
1995	1,167	JUN.
1996	1,026	JUN.
1997	1,480	MAR.
1998	1,080	MAY
1999	1,179	FEB.
2000	684	APR.
2001	1,120	MAR.
2002	1,116	APR.
2003	1,015	JUN.
2004	1,007	DEC.
2005	1,229	FEB.
2006	735	MAR.
2007	930	JAN.
2008	1,456	APR.
2009	1,277	MAY

a. Natural River Cross-sections. The sitting of a diversion requires particular attention to the geomorphology in the vicinity of the project. Consequently studies were made to determine the most logical locations strictly on the basis of the shape and depth of the Mississippi River in the study reach. Attachments 5 and 6 are examples of cross-sections that occur at point bars in the River at bends where the river may change directions. This occurs less prominently at RM 60.2 AHP on Attachment 5A where the cross-section is plotted looking downstream. The point bar is on the right side of the River and is a natural feature of meandering rivers. Large quantities of sand are naturally stored here and are generally best suited for the location of sediment diversion projects. However, the River is deeper offshore at RM 60.2 AHP, - 54 feet, and is a handicap to this location.

b. Diversion Channel Cross-sections. See Sheet 2 in Appendix B for plan view of the project. Given the express purpose of conveying high concentrations of sediments in full suspension above the bed of the diversion channel into the receiving water body, that channel must have as small a wetted perimeter as practical to provide a large hydraulic radius, must be hydraulically smooth to conserve available energy, yet must have a high flow velocity and must cause turbulent flow for good transport capability. However, hydraulic efficiency and turbulence are generally at cross purposes and therefore counter balance each other, and sediment diversion places addition requirements on the design, because the depth of the point bar, -40 feet at RM 60.2 AHP, will govern the shape of the diversion channel, and the wetted perimeter. Based on the average annual peak flow of approximately 1.05 million cfs, a river stage of nine (9) feet is expected, neglecting the tidal effects, so a total depth of 44 feet (-35' + 9') over the gate sill would dictate the most efficient cross-section. It is common knowledge that greater hydraulic efficiency can be achieved if the width of a channel is twice the depth, in which the width would be 2 x 44 feet or 88 feet. However, these dimensions, 44 feet x 88 feet, will provide more than twice the diversion flow being used, 15,000 cfs, as the target flow for modular design. However, this size channel could be used as a modular target to increase the design flow to 70,000 cfs. Therefore, the basic design for the diversion channel will be a 44 feet x 30 feet concrete flume that will produce adequate turbulence at an average velocity of eleven (11) feet per second.

c. Water Surface Profiles. For this preliminary design, a site at RM 60.2 near Myrtle Grove was chosen as a basis to construct a 1-D HEC-RAS model of the diversion channel, for the purpose of computing a water surface profile from the river through the diversion channel and control gate for a flow of 15,000 cfs. The flow rate in the Mississippi River was chosen as 750,000 cfs for these computations to be representative of an intermediate rate between incipient sediment motion (600,000 cfs) and the average annual peak of 1,050,000 cfs. Attachment 7 is a plot of the water surface profile along the diversion channel for a flow rate of 15,000 cfs. Attachment 8 is a tabulation of the unsteady flow hydraulic computations made in the HEC-RAS model for the flow conditions shown in Attachment 7. The important flow parameters to note here are the energy grade lines at the junction of the diversion channel with the river and the flow velocities in the diversion channel that would sustain sediment entrainment to the receiving water body. Note that the differences in the water surface elevations at the junction is the head required to accelerate the flow coming from the river at approximately three feet per second to eleven feet per second into the diversion channel and control structure. This is demonstrated by the sharp drop in water elevation near the upstream end on the diversion channel water surface profile in Attachment 7. A water surface profile along the diversion channel for a flow rate of 70,000 cfs is shown at Attachment 9 and the unsteady flow hydraulic computation for the flow is tabulated in Attachment 10.

5. BASIC SEDIMENT TRANSPORT. It is of primary importance to this project that it is able to transport high sediment concentrations of sands, silts, and clays effectively in to the Barataria Basin. This can be accomplished at any time there are suspended materials in the Mississippi River and sufficient water surface differences between the river and the basin. But what are the limitations that ultimately render the diversion ineffective with respect to sands, which are the building block to land creation. Every stable sea coast either is composed of rock, or is fronted with a sandy beach that lines the weaker soils beneath it. This section discusses tasks that have been carried out to determine the threshold velocity that will just begin to move bed load that has settled to the bed of the conveyance channel at the end of the last seasonal operation.

a. Basic Assumptions. Parameters governing the problems include water temperature and density, salinity, grain size and distribution, water depth and average velocity through the channel. We will assume that river water is fresh and the temperature is 15 degrees centigrade, that the water depth is minimum of 38 feet, and that the grain size distribution is similar to that used at Belle Chasse for operation of the Small Scale Physical Model. Attachment 11 presents similar date for bed load gradations in the Mississippi River below Venice, Louisiana. These are characterized as: D_{10} = 0.1 mm, D_{50} size = 0.19 mm and D_{84} size = 0.3 mm. The calculations that follow will determine by trial and error what the average velocity must be in the diversion channel for incipient motion at the bed for the median sizes.

b. Basic Transport Calculations. The procedure, and knowledge, used here is that outlined by Richard Soulsby in a book entitled "*Dynamics of Marine Sands*" and published in 1997 by Telford Publications, 1 Heron Parkway, London, UK, particularly Chapter 3, Page 45 on "Currents". So having the site particulars previously given in a. above, by trial velocities of two to eight feet per second were used to forecast the incipient motion of Mississippi River bed material through the diversion structure and channel, and the transport per unit width in meters squared per second, $m^2/_S$, which may then be converted to volume by multiplying by one meter width, then converting to cubic yards. Attachments 12 to 18 provide a print out of the calculations.

c. Results. The results of the sediment calculations indicate that some sands will begin moving off the bar into the diversion structure when the discharge is as low as 4,000 cfs for any river condition above 500,000 cfs, which is needed to induce incipient bed material motion for the median grain sizes considered typical in the reach. However, Attachment 11 shows an envelope of the sampled data, and some sizes smaller than the median will begin to move earlier on a rising flow hydrograph. Consequently, although pulsing is the preferred method of operation among many proponents of the project, it will be feasible to transport sediment through the project for longer periods. This feature could allow for more effective project management and land building opportunities. Table 2 demonstrates that on the basis of an average 81 day diversion period, as shown on the Diversion Schedule, Attachment 19, 1.19 million yards of sand for 15,000, could be diverted annually. Since sands make up about 25% of the total suspended load and silts and clays make up the remainder, it is possible then to divert $^{1.19}/_{.25}$ = 4.76 million cubic yards of river sediments over an 81 day diversion period for the current diversions under consideration. However, for a much longer diversion period, say 120 days annually, the deposition volume could be increased by 50% of these limits. Attachment 19 is the outflow rate for a 15,000 cfs diversion structure and the Mississippi River average annual flow hydrograph shown in Attachment 3. A similar diversion schedule for 75,000 cfs would produce flow, and sediments rates, approximately four and a half (4.5) times higher than those shown on Attachment 19 and Table 2 shows this.

6. CONCEPTUAL DIVERSION STRUCTURE DESIGNS. Since at this point in time, February 2010, it is not yet known with any certainty what size and flow capacity will be funded at the Myrtle Grove project site, the Office of Coastal Protection and Restoration, and the team of Consultants, concluded that a modular approach to the design process would be appropriate. Therefore, a basic size that would provide a peak 15,000 cfs flow rate through a diversion structure for a mean annual peak flow rate of 1,050,000 cfs in the Mississippi River was chosen as the building block for the modular approach. Details of this design, and the design of the ultimate size of 75,000 cfs, are given in the following discussion. More details for a 45,000 cfs, as well as the 75,000 cfs, diversion are given in Appendices C thru F.

TABLE 2

TOTAL ANNUAL LOAD TRANSPORT POTENTIAL
(in cubic yards)

Feet / Second	2	2.5	3	4	5	6	7	8
Meters / Second	0.61	0.76	0.914		1.52	1.83	2.13	2.44
$M^2/S/M$	1.22 E-05	0.614 E-04	1.82 E-04		2.06 E-03	4.50 E-03	8.32 E-03	14.22 E-03
CU^3/Yards/M	1.59 E-05	0.802 E-04	2.38 E-04		2.69 E-03	5.88 E-03	10.87 E-03	18.59 E-03
Yards/Day/M	1.37	6.9	20.6		232.0	508.0	939.2	1,606.2
Times 30'/3.28'/M	12.5 Yds. / Day	63.1 Yds. / Day	188.5 Yds. / Day		2,123 Yds. / Day	4,650 Yds. / Day	8,594 Yds. / Day	14,697 Yds. / Day
81 Days / Year	1,015 Yds. / Year	5,114 Yds. / Year	1.53x4 Yds. / Year		$1.72x10^5$ Yds. / Year	$3.77x10^5$ Yds. / Year	$6.96x10^5$ Yds. / Year	$1.19x10^6$ Yds. / Year
Times 140'/3.28'/M	58.5 Yds. / Day	294.5 Yds. / Day	879.3 Yds. / Day		9,903 Yds. / Day	21,683Yds. / Day	40,088 Yds. / Day	68,557 Yds. / Day
81 Days / Year	4,737 Yds. / Year	$2.39x10^4$ Yds. / Year	$7.12x10^4$ Yds. / Year		$8.02x10^5$ Yds. / Year	$1.76x10^6$ Yds. / Year	$3.25x10^6$ Yds. / Year	$5.6x10^6$ Yds. / Year

a. Basic 15,000 cfs Diversion Structure. Basic features of the design are shown on the drawings, Sheets 2 to 4 in Appendix B. The general layout of the plan is shown on Sheet 2 and extends from within the Mississippi River, some 1,100 feet at the upstream end and some 11,900 feet downstream from the control structure. The concrete flume section is 770 feet in length. Typical diversion channel cross-sections are shown on Sheet 3, and a typical elevation drawing of the gated control structure is shown on Sheet 4. The gates will be five individual 30 feet wide by ten feet (30' X 10') high segments that can be dogged at any desired gate opening to facilitate diversion choices to enhance sediment diversion or freshwater mitigation. River works in the shape of a 1,200 foot deflection dike and free standing sheet pile wall, are included to enhance water and sediment entry into the diversion channel. The structure will have a single gate lifting crane to operate the gates.

b. Modular 75,000 cfs Diversion Structure. The ultimate size project structure is essentially an enlarged version of the basic design, in that many features/shapes/etc. are similar. Economics of scale would reduce the cost per cfs diverted, and the method of flume construction would change from a strut supported side wall configuration of the single gated plan to a flume not unlike the walls of a navigation lock chamber, but less expensive. See Sheets 5 and 6 in Appendix B for typical channel cross-sections. The deflection dike and sheet pile wall would be identical to the basic design as well as the features of the gated structure. There will be four 32-foot wide gate bays that will be identical to the single gated structure except that one gate lifting crane will travel by rail from gate bay to gate bay to open or close any specified number of gates on a daily basis. See Sheet 4.

7. LOCATION SPECIFIC REQUIREMENTS.

a. Existing Drainage. This particular location and alignment will sever a portion of the drainage system currently in existence on Conoco Phillips Company lands, as shown on Sheet 2 in Appendix B, and measures will need to be taken to render the drainage system whole again. Several alternative measures were investigated that could do this. One of the most practical alternatives investigated was an inverted jack and bore installed siphon that would restore a connection of the drainage canal to either side of the diversion channel. Another alternative would be a steel pipe connection from one side to the other that would be placed directly in the diversion flow at an elevation near that of the drainage system invert. Typical alternatives are shown on Sheets 5 and 7 in Appendix B.

b. Utilities Relocation. There are public utilities, water forced main, a natural gas line, and several commercial power lines, that will require major relocations and modifications to span across the project channel. In addition an elevated four-lane bridge and access ramps will be required to accommodate area traffic on State Hwy. 23. Typical sections and profiles are shown on Sheets 8 and 9 in Appendix B.

c. Levee Protection. Federal levees and local back levees exist at the Mississippi River bank and at the National Food and Beverage Company western boundary. This will require that adjacent levees, equal in design criteria to Federal Mississippi River and Tributaries Project Levees, must be provided on each bank of the diversion channel, since these levees will contain Mississippi River water.

8. BUDGET LEVEL COST ESTIMATE. See Table 3 for a probable construction cost estimate for the 15,000 cfs Myrtle Grove Freshwater and Sediment Diversion Project as located by the U.S. Army Corps of Engineers at Mississippi RM 60.2 AHP. Note that the estimate is a construction cost estimate that does not include any land acquition costs or engineering and design fees, or any costs for supervision and administration during construction. Also note that excavation in Table 3 is excavation needed for construction of the flume section, the control structure, and the outflow conveyance channel. Dredging is that required to form the marsh transition channel, and levees are the project levees parallel to the convenience channel needed to protect adjacent pasture land from the diverted Mississippi River flows. We believe the estimate is on the conservative side of $180 million dollars, because the plan includes a transition section into the receiving marsh land that is optional and is intended to promote better sediment transport through hydraulic efficiency. To end the discharge channel abruptly at the limits of the existing back levee would cause an undesirable backwater effect that would reduce sediment transport through the conveyance channel.

However for cost management purposes, the transition feature could be scaled back or eliminated as desired. The transition into the marsh land is shown schematically on the model plan at Attachment 20 in Appendix A. The channel invert profile is shown at Attachment 7 in Appendix A. Elimination of the Marshland transition would realize an initial cost savings of $18.8 million dollars and a 10% loss in discharge capacity at the point in time of completion of the project. The long-term consequence of this action, however, would be higher maintenance costs to the project over the useful life of the project due to clogging of the conveyance channel at an earlier point in time and perhaps annual or almost annual maintenance due to sediment deposition. A scaled back marshland transition of 50% would cost about $7 million and there would be only a 4% loss in capacity, but would provide some degree of advanced maintenance to project advantage over the non-transition option.

The cost estimate includes a cost for replacing Louisiana Highway 23 with a four-lane, Class 4 highway bridge system essentially as shown on Sheet 8 in Appendix B. However the estimate does not include any allowances for the relocation of other utilities, such as electrical service or water supply, but is covered in the contingencies in Table 3.

On the basis of this cost estimate, shown in Table 3, it is possible to expand the project to a size that could pass 45,000 cfs at a peak river stage near 9.0 feet or above, and remain within the projected Federal funding limit.

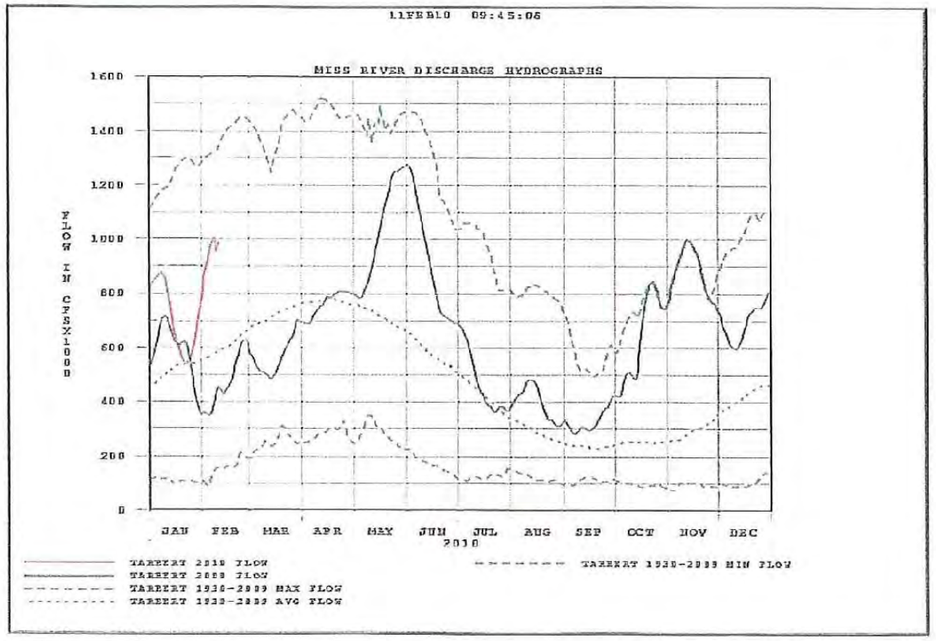
TABLE 3

BUDGETARY COST ESTIMATE
MYRTLE GROVE DIVERSION CHANNEL
RIVER MILE 60.2

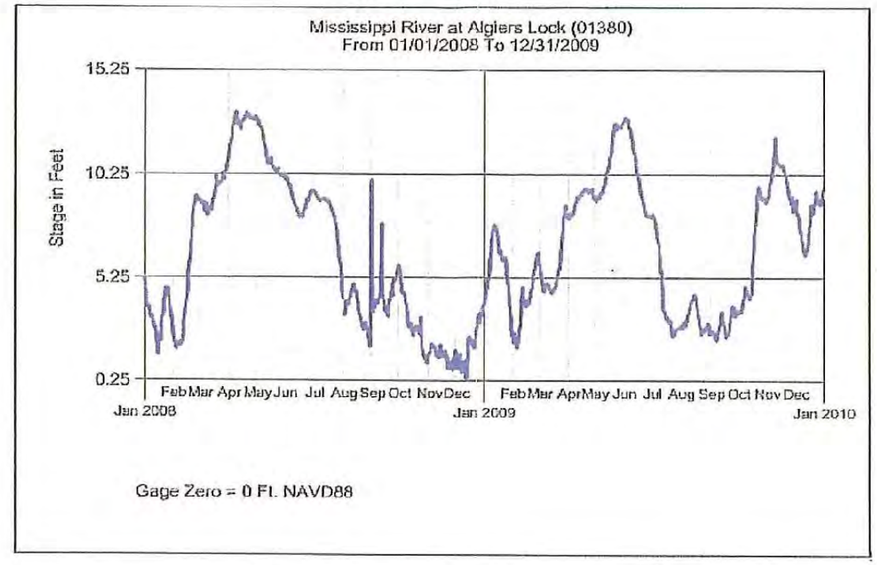
ITEM NO.	DESCRIPTION	UNIT	QUANITY	UNIT PRICE	AMOUNT
1.	Mobilization	Lump			$1,700,000.00
2.	Excavation	CY	937,240	$10.00	$9,372,400.00
3.	Dredging	CY	752,580	$25.00	$18,814,500.00
4.	Levees	CY	1,125,932	$14.00	$15,763,048.00
5.	Rip-rap	Tons	538,253	$70.00	$37,677,710.00
6.	Crushed Stones	CY	113,269	$55.00	$6,229,795.00
7.	Geo-textile Fabric	SY	484,622	$10.00	$4,846,220.00
8.	Dike (Sand-filled Geo-tubes)	CY	477,637	$20.00	$9,552,740.00
9.	Reinforced Concrete				
	A. Flume:				
	1. Base Slab	CY	17,111	$500.00	$8,555,500.00
	2. Walls	CY	9,411	$700.00	$6,587,700.00
	B. T-walls:				
	1. Base Slab	CY	700	$500.00	$350,000.00
	2. Walls	CY	570	$700.00	$399,000.00
	C. Gate Structure:				
	1. Base Slab	CY	500	$500.00	$250,000.00
	2. Piers	CY	372	$700.00	$260,400.00
	D. Pile Caps for 60 inch Pipe	CY	21	$700.00	$14,700.00
10.	Structural Steel Gates	Tons	27	$30,000.00	$810,000.00
11.	Piling				
	HP 14 x 73	L.F.	163,000	$75.00	$12,225,000.00
	12 inch dia. Steel Pipe Piles	L.F.	800	$75.00	$60,000.00
12.	PZ-22 Cut-off	SF	5,880	$45.00	$264,600.00
13.	60 inch dia. x 3/8 inch w.t.				
	AWWA C200 Pipe	L.F.	620	$1,200.00	$744,000.00
14.	20 inch Dia. Sched 80 Shell Oil Overhead Pipe Crossing and Pipe Supports	L.F.	450	$450.00	$202,500.00
15.	40 T Gantry Crane	Each	1	$500,000.00	$500,000.00
16.	LA 23 Bridge	L.F.	1,000	$14,625.00	$14,625,000.00
17.	Demobilization	Lump			$850,000.00
					$143,809,048.00
	SUBTOTAL				**$150,654,813.00**
18.	Contingencies (20%)	Lump			$30,130,962.60
	TOTAL				**$180,785,775.60**
NOTES:					
1. All prices are in current dollars and do not include future inflation.					
2. Quantities shown are based on preliminary design only and are not all inclusive.					
3. Land acquisition costs not included.					

APPENDIX A

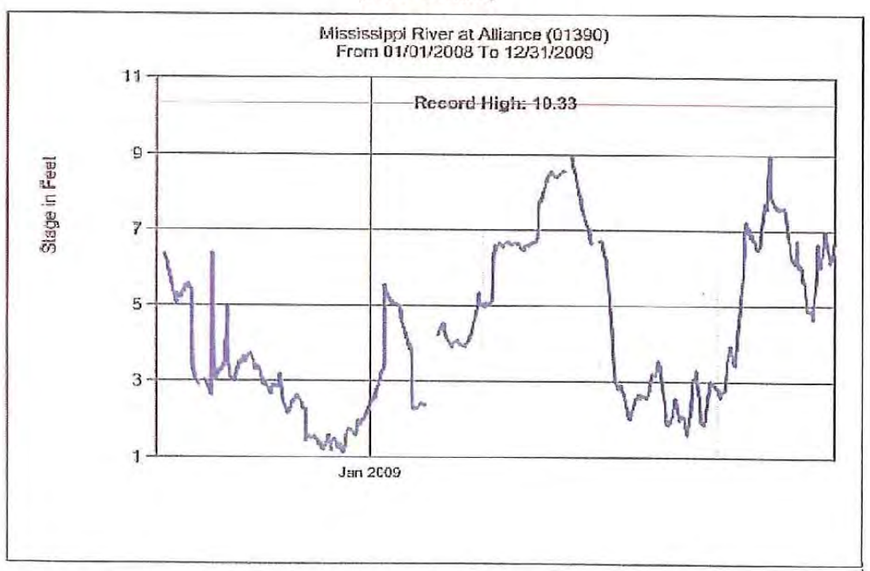
Attachments 1 to 20



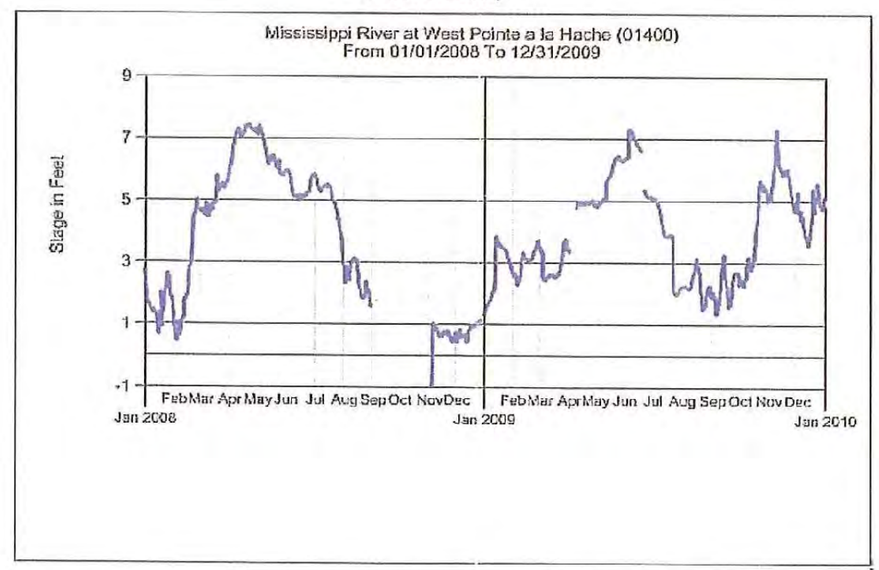
Mississippi River at Algiers Lock (01380)
(08:00 Central)



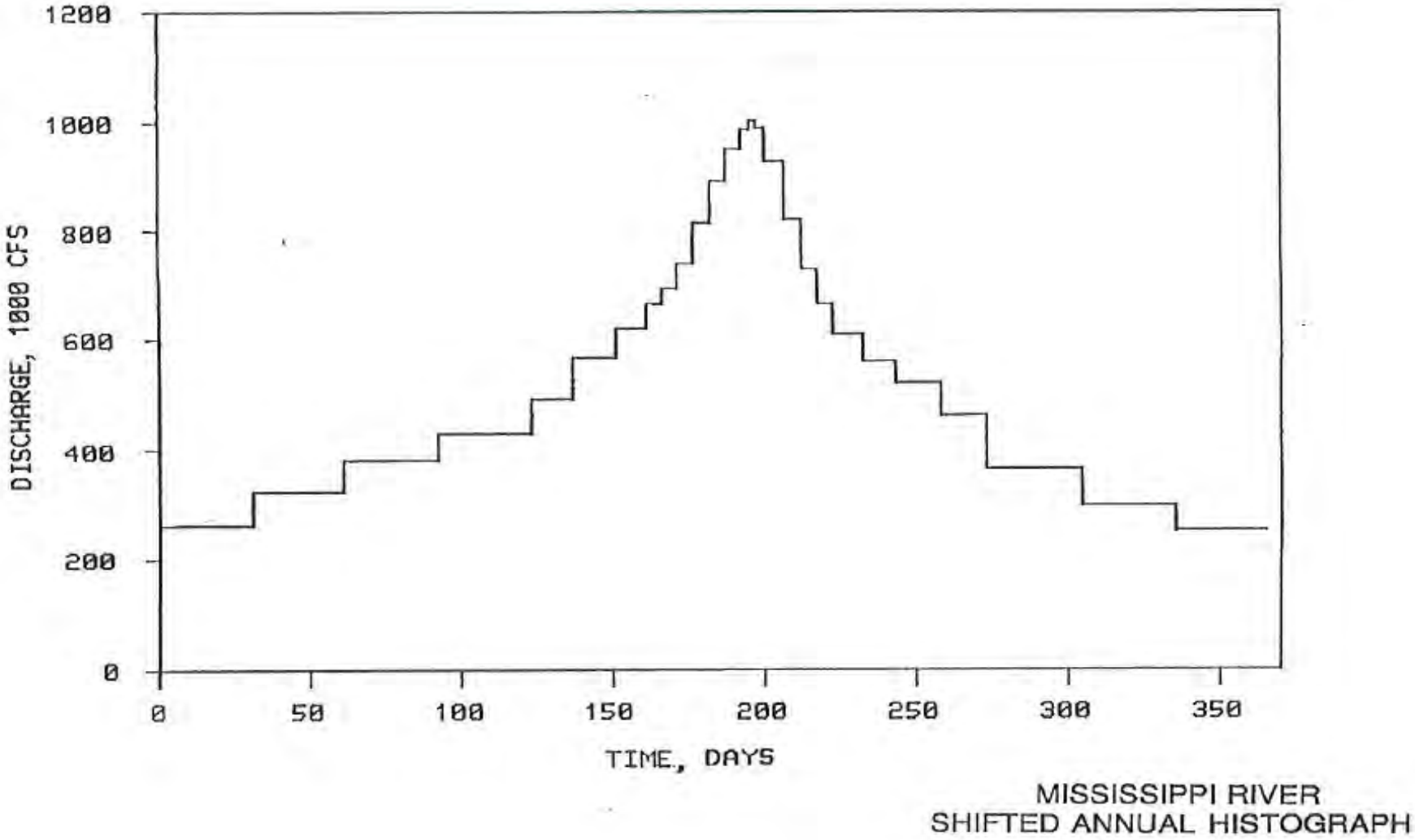
Mississippi River at Alliance (01390)
(08:00 Central)



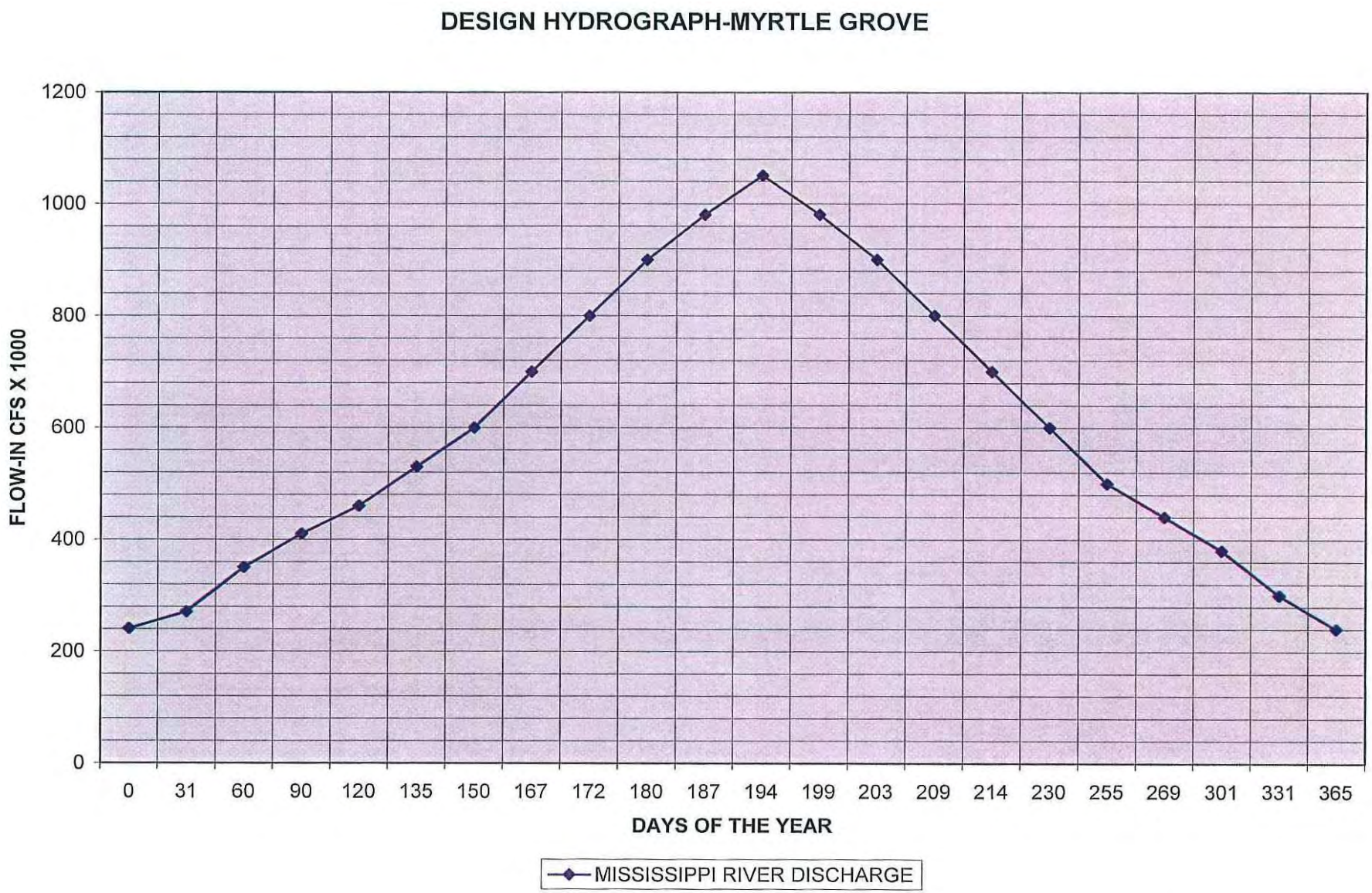
Mississippi River at West Pointe a la Hache (01400)
(08:00 Central)



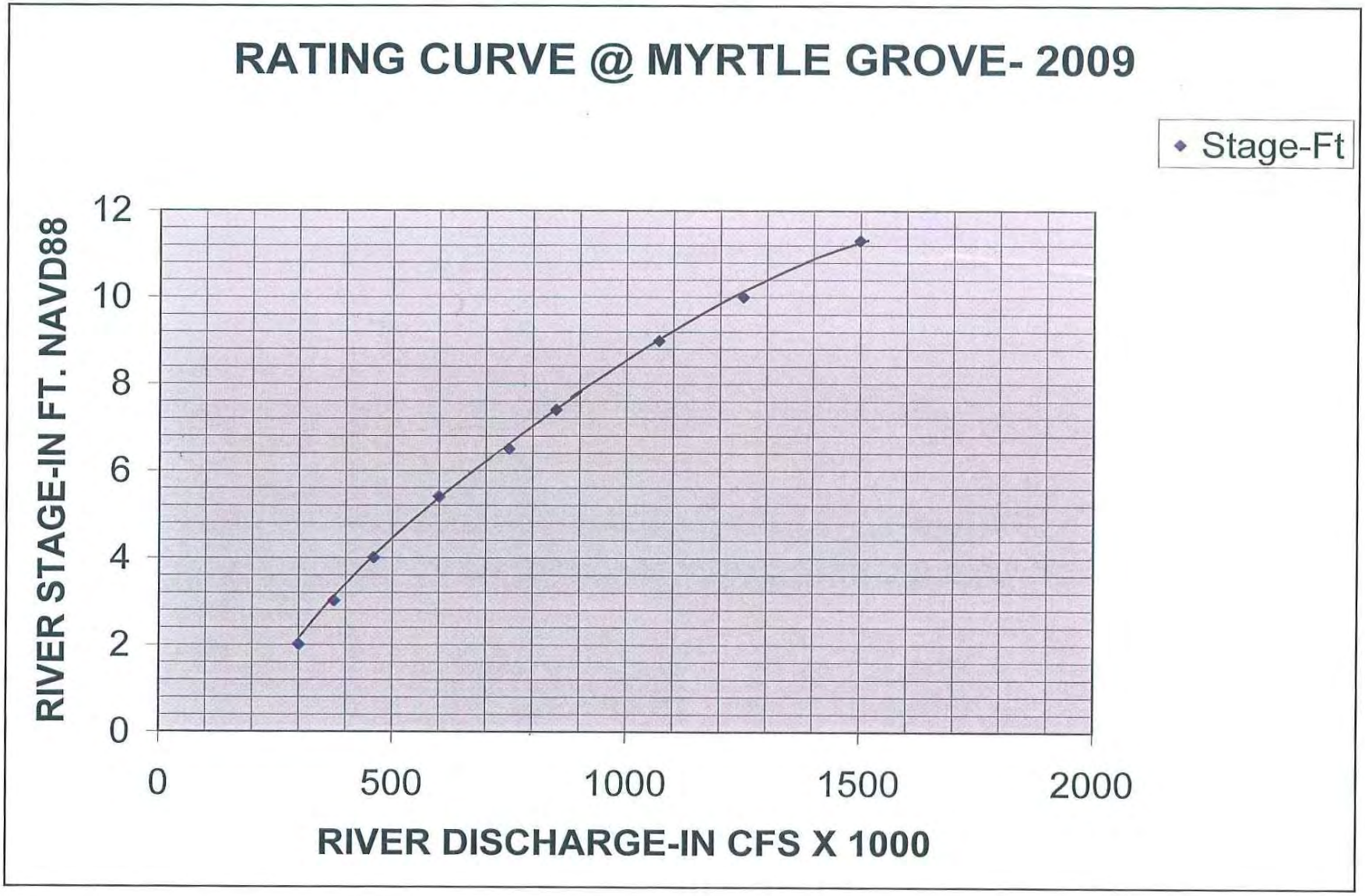
ATTACHMENT 1 - HYDROLOGIC DATA FOR THE MISSISSIPPI RIVIER IN VICINITY OF MYRTLE GROVE, AT MILE 60.7 ABOVE HEAD OF PASSES



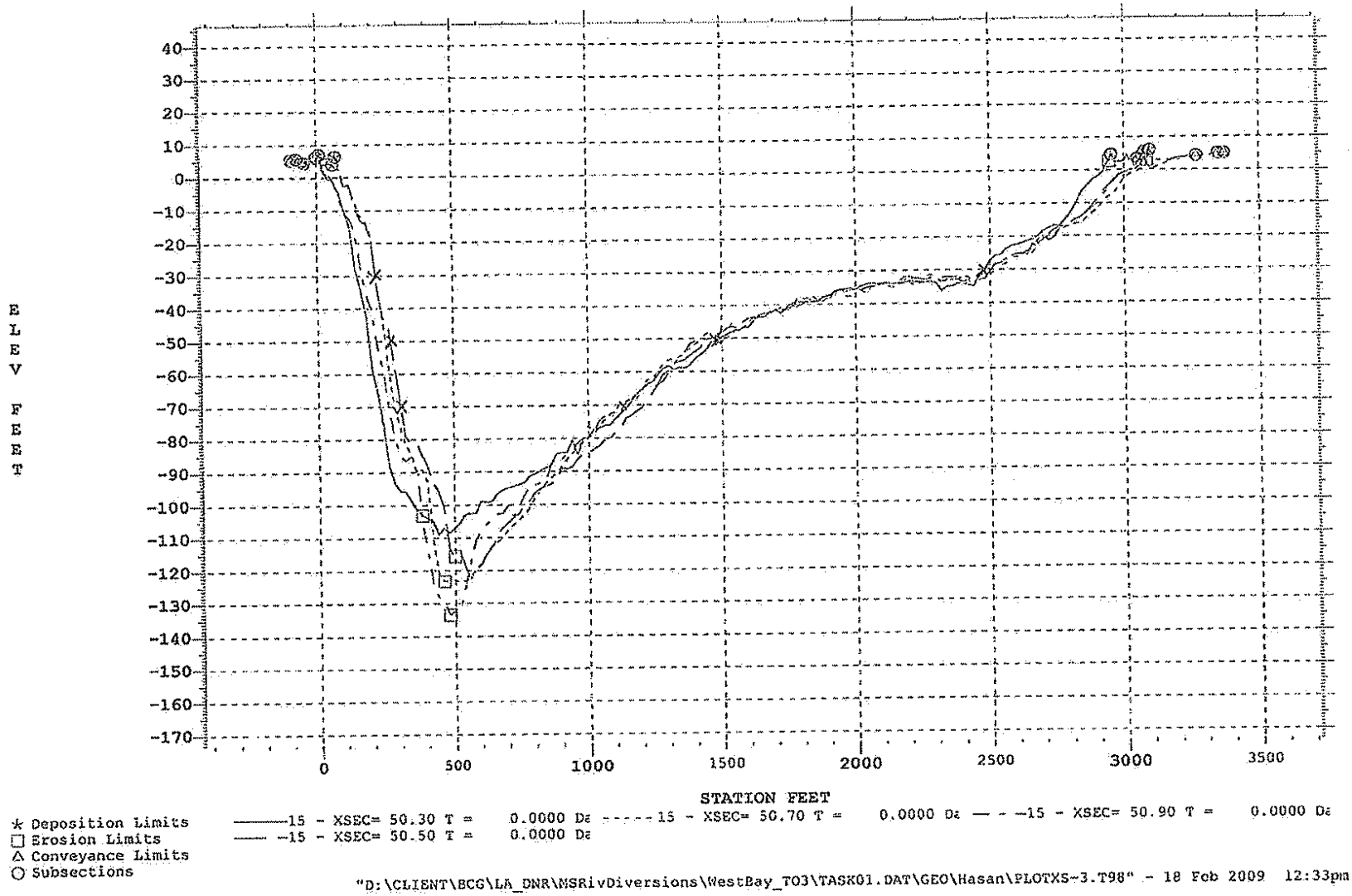
ATTACHMENT 2 – MISSISSIPPI RIVER ANNUAL HISTOGRAPH OF DISCHARGE IN THE PROJECT REACH



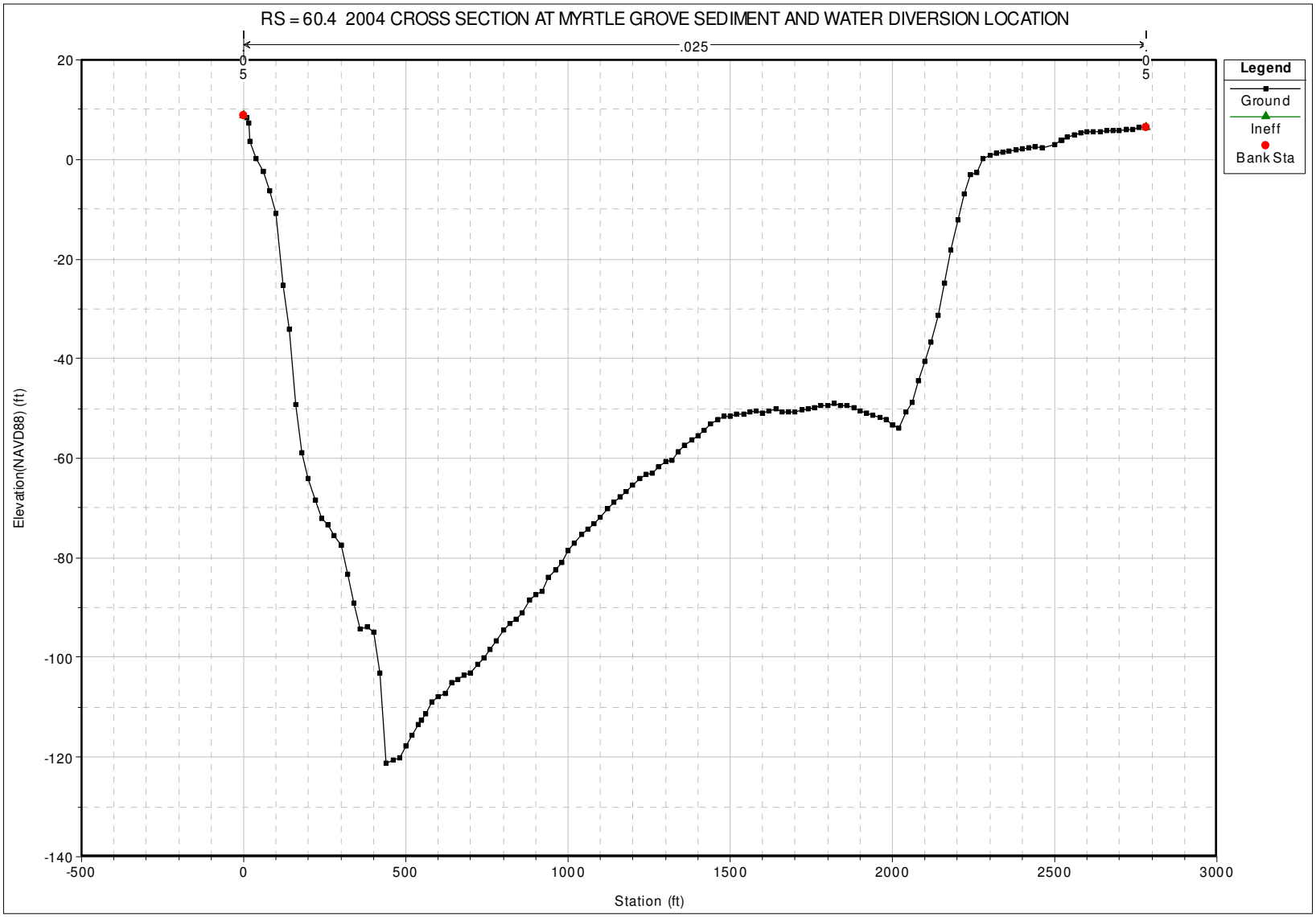
ATTACHMENT 3 – CONCEPTUAL DESIGN FLOW HYDROGRAPH IN MISSISSIPPI RIVER



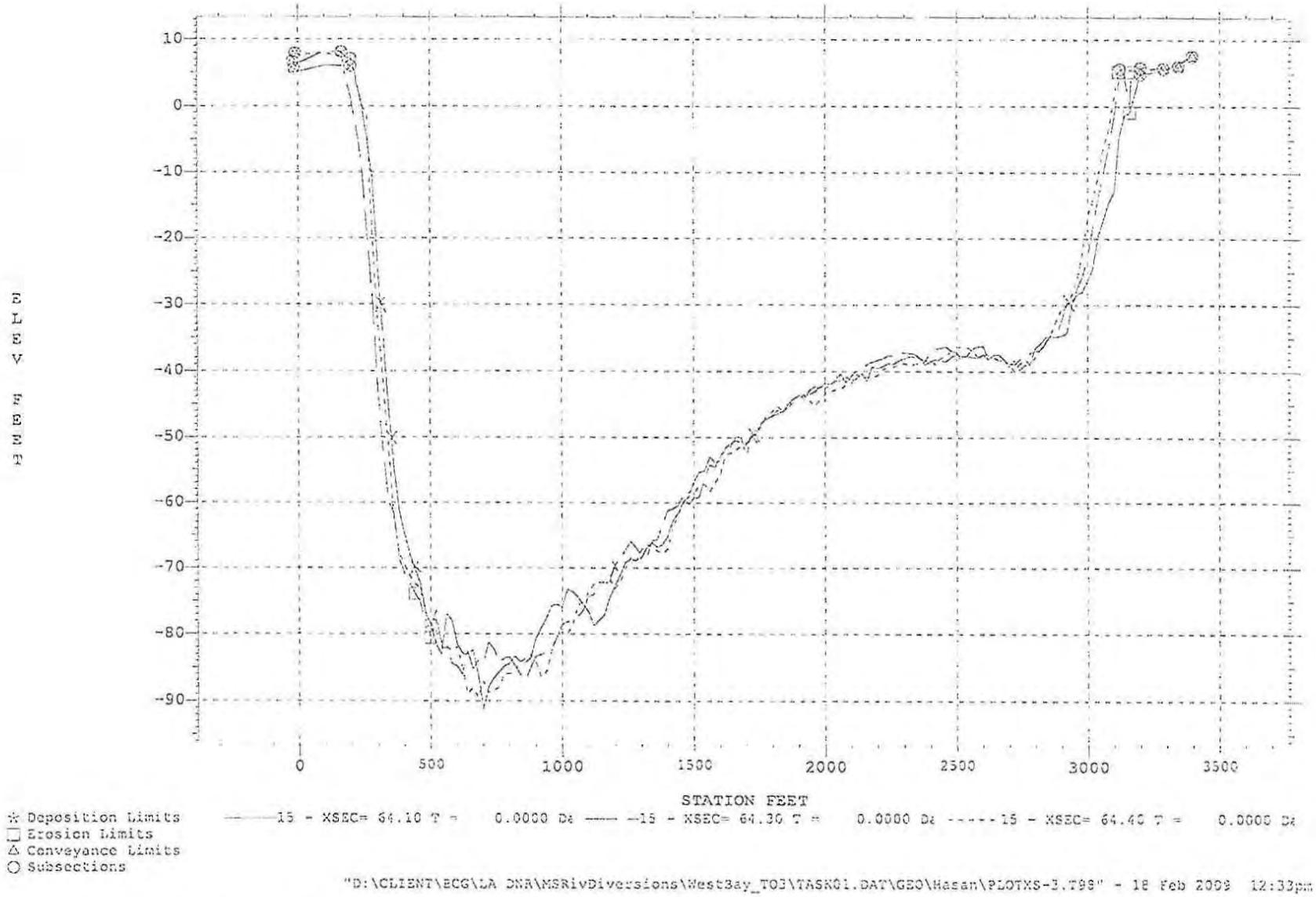
ATTACHMENT 4 – CONCEPTUAL DESIGN RATING CURVE FOR MILE 60.2 AHP



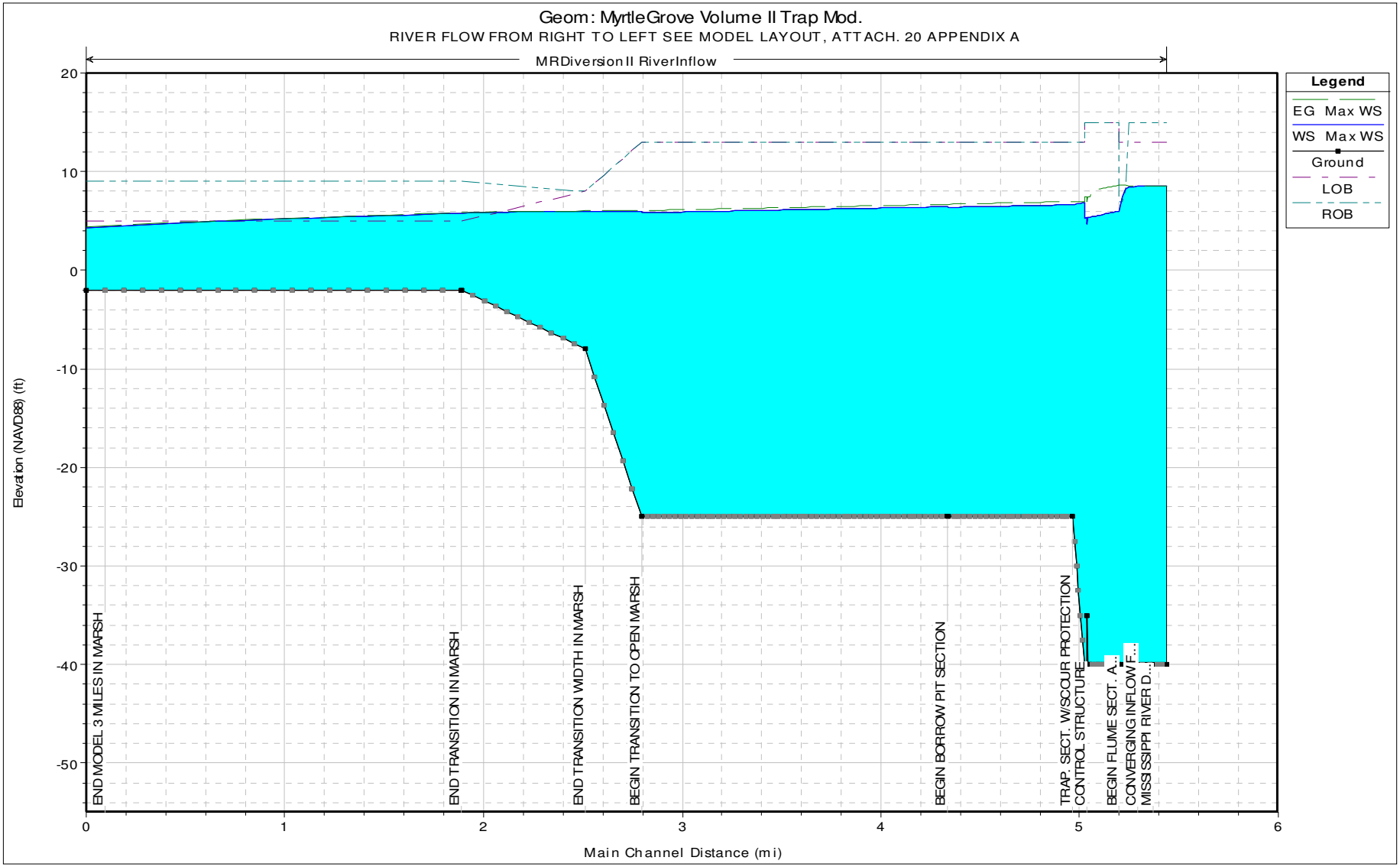
ATTACHMENT 5 - COMPARATIVE CROSS-SECTION NO. 2 FOR DETERMINING RELATIVE STABILITY OF RIVER AT SELECTED LOCATIONS IN STUDY REACH



ATTACHMENT 5A - SECTION SHOWN LOOKING DOWNSTREAM WITH DIVERSION ON THE RIGHT SIDE – NOTE STEEP GRADIENT BETWEEN TOP OF BANK AND – 54 FT. CONTOUR



ATTACHMENT 6 - COMPARATIVE CROSS-SECTIONS FOR DETERMINING RELATIVE STABILITY OF RIVER AT SELECTED LOCATIONS IN STUDY REACH



ATTACHMENT 7 – HEC-RAS MODEL PROFILES FOR 15,000 CFS FLOW/SEDIMENT DIVERSION PLAN

HEC-RAS Plan: Plan 04 River: MRDiversionII Reach: RiverInflow Profile: 24FEB2010 1100

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
RiverInflow	0.00	24FEB2010 1100	13764.73	-2.00	3.77	-0.43	3.83	0.000200	1.91	7190.31	1262.96	0.14
RiverInflow	4.98000*	24FEB2010 1100	13804.92	-2.00	3.87		3.92	0.000190	1.89	7315.34	1263.53	0.14
RiverInflow	9.96000*	24FEB2010 1100	13845.18	-2.00	3.96		4.02	0.000182	1.86	7434.42	1264.07	0.14
RiverInflow	14.94*	24FEB2010 1100	13885.59	-2.00	4.05		4.10	0.000174	1.84	7548.22	1264.58	0.13
RiverInflow	19.92*	24FEB2010 1100	13926.19	-2.00	4.14		4.19	0.000167	1.82	7657.24	1265.07	0.13
RiverInflow	24.9*	24FEB2010 1100	13966.94	-2.00	4.22		4.27	0.000160	1.80	7761.98	1265.55	0.13
RiverInflow	29.88*	24FEB2010 1100	14007.87	-2.00	4.30		4.35	0.000155	1.78	7862.74	1266.00	0.13
RiverInflow	34.86*	24FEB2010 1100	14048.96	-2.00	4.38		4.43	0.000149	1.76	7959.88	1266.44	0.12
RiverInflow	39.84*	24FEB2010 1100	14090.21	-2.00	4.45		4.50	0.000145	1.75	8053.84	1266.86	0.12
RiverInflow	44.82*	24FEB2010 1100	14131.67	-2.00	4.52		4.57	0.000140	1.74	8144.90	1267.27	0.12
RiverInflow	49.8*	24FEB2010 1100	14173.37	-2.00	4.59		4.64	0.000136	1.72	8233.26	1267.67	0.12
RiverInflow	54.78*	24FEB2010 1100	14215.28	-2.00	4.66		4.71	0.000132	1.71	8319.10	1268.06	0.12
RiverInflow	59.76*	24FEB2010 1100	14257.40	-2.00	4.73		4.77	0.000129	1.70	8402.62	1268.44	0.12
RiverInflow	64.74*	24FEB2010 1100	14299.74	-2.00	4.79		4.83	0.000125	1.69	8483.91	1268.80	0.11
RiverInflow	69.72*	24FEB2010 1100	14342.31	-2.00	4.85		4.90	0.000122	1.67	8563.11	1269.16	0.11
RiverInflow	74.7*	24FEB2010 1100	14385.08	-2.00	4.91		4.96	0.000120	1.66	8640.45	1269.51	0.11
RiverInflow	79.68*	24FEB2010 1100	14428.06	-2.00	4.97		5.02	0.000117	1.66	8716.07	1269.85	0.11
RiverInflow	84.66*	24FEB2010 1100	14471.31	-2.00	5.03		5.07	0.000114	1.65	8790.05	1270.02	0.11
RiverInflow	89.64*	24FEB2010 1100	14514.81	-2.00	5.09		5.13	0.000112	1.64	8862.46	1270.04	0.11
RiverInflow	94.62*	24FEB2010 1100	14558.56	-2.00	5.14		5.19	0.000110	1.63	8933.36	1270.07	0.11
RiverInflow	99.60	24FEB2010 1100	14602.56	-2.00	5.20		5.24	0.000107	1.62	9002.86	1270.10	0.11
RiverInflow	99.80	24FEB2010 1100	14604.33	-2.00	5.20		5.24	0.000107	1.62	9005.62	1270.10	0.11
RiverInflow	102.963*	24FEB2010 1100	14630.66	-2.55	5.24		5.27	0.000086	1.52	9603.42	1256.15	0.10
RiverInflow	106.127*	24FEB2010 1100	14656.80	-3.09	5.26		5.29	0.000070	1.44	10160.42	1241.18	0.09
RiverInflow	109.290*	24FEB2010 1100	14682.71	-3.64	5.28		5.31	0.000058	1.37	10704.32	1226.13	0.08
RiverInflow	112.454*	24FEB2010 1100	14708.41	-4.18	5.30		5.33	0.000049	1.31	11212.25	1211.02	0.08
RiverInflow	115.618*	24FEB2010 1100	14733.89	-4.73	5.32		5.34	0.000042	1.26	11709.25	1195.86	0.07
RiverInflow	118.781*	24FEB2010 1100	14759.14	-5.27	5.33		5.36	0.000036	1.21	12172.58	1180.66	0.07
RiverInflow	121.945*	24FEB2010 1100	14784.18	-5.82	5.34		5.37	0.000032	1.17	12626.32	1165.48	0.06
RiverInflow	125.109*	24FEB2010 1100	14808.94	-6.36	5.35		5.37	0.000028	1.13	13047.61	1150.23	0.06
RiverInflow	128.272*	24FEB2010 1100	14833.45	-6.91	5.36		5.38	0.000025	1.10	13459.75	1135.00	0.06
RiverInflow	131.436*	24FEB2010 1100	14857.70	-7.45	5.37		5.39	0.000022	1.07	13840.61	1119.74	0.05
RiverInflow	134.6	24FEB2010 1100	14881.68	-8.00	5.38		5.40	0.000020	1.05	14212.35	1104.46	0.05
RiverInflow	137.133*	24FEB2010 1100	14900.57	-10.83	5.39		5.40	0.000015	1.00	14876.85	978.96	0.05
RiverInflow	139.666*	24FEB2010 1100	14917.21	-13.67	5.39		5.40	0.000013	1.01	14725.08	853.22	0.04
RiverInflow	142.2*	24FEB2010 1100	14931.54	-16.50	5.39		5.41	0.000013	1.09	13742.09	727.42	0.04
RiverInflow	144.733*	24FEB2010 1100	14943.56	-19.33	5.39		5.41	0.000016	1.25	11937.46	601.54	0.05
RiverInflow	147.266*	24FEB2010 1100	14953.30	-22.17	5.38		5.42	0.000027	1.61	9314.53	475.57	0.06
RiverInflow	149.80	24FEB2010 1100	14960.72	-25.00	5.34		5.44	0.000086	2.55	5857.02	349.25	0.11
RiverInflow	150.00	24FEB2010 1100	14961.16	-25.00	5.26		5.44	0.000057	3.38	4428.95	270.37	0.15
RiverInflow	151.450*	24FEB2010 1100	14963.89	-25.00	5.27		5.45	0.000057	3.38	4425.66	270.26	0.15
RiverInflow	152.901*	24FEB2010 1100	14966.63	-25.00	5.28		5.46	0.000057	3.38	4423.79	270.19	0.15
RiverInflow	154.352*	24FEB2010 1100	14969.38	-25.00	5.29		5.47	0.000057	3.39	4420.52	270.08	0.15

ATTACHMENT 8 – UNSTEADY FLOW COMPUTATIONS – 15,000 CFS DIVERSION

HEC-RAS Plan: Plan 04 River: MRDiversionII Reach: RiverInflow Profile: 24FEB2010 1100 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
RiverInflow	155.802*	24FEB2010 1100	14972.15	-25.00	5.30		5.47	0.000057	3.39	4418.65	270.01	0.15
RiverInflow	157.253*	24FEB2010 1100	14974.92	-25.00	5.30		5.48	0.000057	3.39	4415.39	269.90	0.15
RiverInflow	158.704*	24FEB2010 1100	14977.70	-25.00	5.31		5.49	0.000057	3.39	4413.53	269.83	0.15
RiverInflow	160.155*	24FEB2010 1100	14980.48	-25.00	5.32		5.50	0.000058	3.40	4410.28	269.72	0.15
RiverInflow	161.605*	24FEB2010 1100	14983.28	-25.00	5.33		5.51	0.000058	3.40	4407.04	269.60	0.15
RiverInflow	163.056*	24FEB2010 1100	14986.09	-25.00	5.34		5.52	0.000058	3.40	4405.20	269.54	0.15
RiverInflow	164.507*	24FEB2010 1100	14988.90	-25.00	5.35		5.53	0.000058	3.41	4401.96	269.42	0.15
RiverInflow	165.957*	24FEB2010 1100	14991.73	-25.00	5.35		5.53	0.000058	3.41	4400.13	269.35	0.15
RiverInflow	167.408*	24FEB2010 1100	14994.56	-25.00	5.36		5.54	0.000058	3.41	4396.91	269.24	0.15
RiverInflow	168.859*	24FEB2010 1100	14997.40	-25.00	5.37		5.55	0.000058	3.41	4395.09	269.17	0.15
RiverInflow	170.31*	24FEB2010 1100	15000.26	-25.00	5.38		5.56	0.000058	3.42	4391.88	269.05	0.15
RiverInflow	171.760*	24FEB2010 1100	15003.12	-25.00	5.39		5.57	0.000059	3.42	4388.67	268.93	0.15
RiverInflow	173.211*	24FEB2010 1100	15005.99	-25.00	5.40		5.58	0.000059	3.42	4386.86	268.86	0.15
RiverInflow	174.662*	24FEB2010 1100	15008.87	-25.00	5.40		5.59	0.000059	3.42	4383.67	268.75	0.15
RiverInflow	176.112*	24FEB2010 1100	15011.75	-25.00	5.41		5.60	0.000059	3.43	4381.87	268.68	0.15
RiverInflow	177.563*	24FEB2010 1100	15014.65	-25.00	5.42		5.60	0.000059	3.43	4378.69	268.56	0.15
RiverInflow	179.014*	24FEB2010 1100	15017.56	-25.00	5.43		5.61	0.000059	3.43	4376.90	268.49	0.15
RiverInflow	180.465*	24FEB2010 1100	15020.47	-25.00	5.44		5.62	0.000059	3.43	4373.73	268.37	0.15
RiverInflow	181.915*	24FEB2010 1100	15023.39	-25.00	5.45		5.63	0.000059	3.44	4370.56	268.25	0.15
RiverInflow	183.366*	24FEB2010 1100	15026.33	-25.00	5.46		5.64	0.000059	3.44	4368.79	268.18	0.15
RiverInflow	184.817*	24FEB2010 1100	15029.27	-25.00	5.46		5.65	0.000060	3.44	4365.64	268.05	0.15
RiverInflow	186.267*	24FEB2010 1100	15032.22	-25.00	5.47		5.66	0.000060	3.44	4363.88	267.98	0.15
RiverInflow	187.718*	24FEB2010 1100	15035.18	-25.00	5.48		5.67	0.000060	3.45	4360.73	267.86	0.15
RiverInflow	189.169*	24FEB2010 1100	15038.15	-25.00	5.49		5.67	0.000060	3.45	4358.98	267.79	0.15
RiverInflow	190.62*	24FEB2010 1100	15041.13	-25.00	5.50		5.68	0.000060	3.45	4355.85	267.67	0.15
RiverInflow	192.070*	24FEB2010 1100	15044.11	-25.00	5.51		5.69	0.000060	3.46	4352.72	267.54	0.15
RiverInflow	193.521*	24FEB2010 1100	15047.11	-25.00	5.52		5.70	0.000060	3.46	4350.99	267.47	0.15
RiverInflow	194.972*	24FEB2010 1100	15050.11	-25.00	5.52		5.71	0.000060	3.46	4347.87	267.34	0.15
RiverInflow	196.422*	24FEB2010 1100	15053.13	-25.00	5.53		5.72	0.000060	3.46	4346.15	267.27	0.15
RiverInflow	197.873*	24FEB2010 1100	15056.15	-25.00	5.54		5.73	0.000061	3.47	4343.04	267.15	0.15
RiverInflow	199.324*	24FEB2010 1100	15059.18	-25.00	5.55		5.74	0.000061	3.47	4341.33	267.07	0.15
RiverInflow	200.775*	24FEB2010 1100	15062.22	-25.00	5.56		5.75	0.000061	3.47	4338.23	266.95	0.15
RiverInflow	202.225*	24FEB2010 1100	15065.27	-25.00	5.57		5.76	0.000061	3.48	4335.14	266.82	0.15
RiverInflow	203.676*	24FEB2010 1100	15068.33	-25.00	5.58		5.76	0.000061	3.48	4333.45	266.75	0.15
RiverInflow	205.127*	24FEB2010 1100	15071.40	-25.00	5.59		5.77	0.000061	3.48	4330.37	266.62	0.15
RiverInflow	206.577*	24FEB2010 1100	15074.47	-25.00	5.59		5.78	0.000061	3.48	4328.68	266.54	0.15
RiverInflow	208.028*	24FEB2010 1100	15077.56	-25.00	5.60		5.79	0.000061	3.49	4325.61	266.41	0.15
RiverInflow	209.479*	24FEB2010 1100	15080.65	-25.00	5.61		5.80	0.000061	3.49	4323.94	266.34	0.15
RiverInflow	210.93*	24FEB2010 1100	15083.76	-25.00	5.62		5.81	0.000062	3.49	4320.88	266.21	0.15
RiverInflow	212.380*	24FEB2010 1100	15086.87	-25.00	5.63		5.82	0.000062	3.49	4317.83	266.08	0.15
RiverInflow	213.831*	24FEB2010 1100	15089.99	-25.00	5.64		5.83	0.000062	3.50	4316.17	266.00	0.15
RiverInflow	215.282*	24FEB2010 1100	15093.12	-25.00	5.65		5.84	0.000062	3.50	4313.13	265.87	0.15
RiverInflow	216.732*	24FEB2010 1100	15096.26	-25.00	5.66		5.85	0.000062	3.50	4311.48	265.80	0.15

ATTACHMENT 8 – UNSTEADY FLOW COMPUTATIONS – 15,000 CFS DIVERSION (continued)

HEC-RAS Plan: Plan 04 River: MRDiversionII Reach: RiverInflow Profile: 24FEB2010 1100 (Continued)

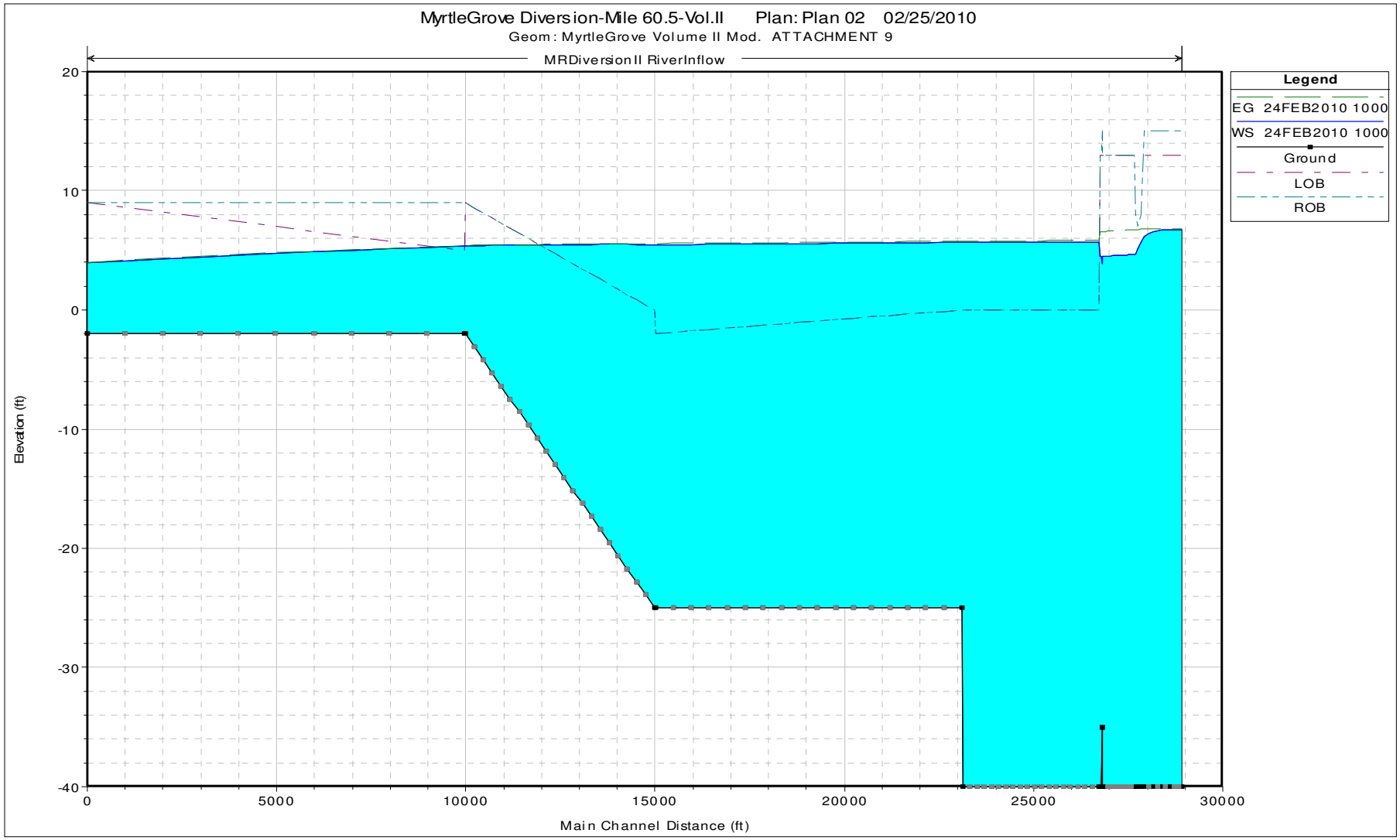
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
RiverInflow	218.183*	24FEB2010 1100	15099.40	-25.00	5.67		5.86	0.000062	3.50	4308.46	265.66	0.15
RiverInflow	219.634*	24FEB2010 1100	15102.56	-25.00	5.67		5.87	0.000062	3.51	4306.81	265.59	0.15
RiverInflow	221.085*	24FEB2010 1100	15105.73	-25.00	5.68		5.87	0.000062	3.51	4303.80	265.45	0.15
RiverInflow	222.535*	24FEB2010 1100	15108.90	-25.00	5.69		5.88	0.000063	3.51	4300.79	265.32	0.15
RiverInflow	223.986*	24FEB2010 1100	15112.08	-25.00	5.70		5.89	0.000063	3.52	4299.17	265.24	0.15
RiverInflow	225.437*	24FEB2010 1100	15115.27	-25.00	5.71		5.90	0.000063	3.52	4296.17	265.11	0.15
RiverInflow	226.887*	24FEB2010 1100	15118.47	-25.00	5.72		5.91	0.000063	3.52	4294.55	265.03	0.15
RiverInflow	228.338*	24FEB2010 1100	15121.68	-25.00	5.73		5.92	0.000063	3.52	4291.57	264.89	0.15
RiverInflow	229.789*	24FEB2010 1100	15124.90	-25.00	5.74		5.93	0.000063	3.53	4289.96	264.81	0.15
RiverInflow	231.24	24FEB2010 1100	15128.13	-25.00	5.75		5.94	0.000063	3.53	4286.99	264.67	0.15
RiverInflow	231.44	24FEB2010 1100	15128.53	-25.00	5.69		5.94	0.000062	4.04	3746.15	214.14	0.17
RiverInflow	232.881*	24FEB2010 1100	15131.09	-25.00	5.70		5.95	0.000062	4.04	3748.17	214.19	0.17
RiverInflow	234.322*	24FEB2010 1100	15133.67	-25.00	5.71		5.96	0.000062	4.04	3750.19	214.25	0.17
RiverInflow	235.763*	24FEB2010 1100	15136.25	-25.00	5.72		5.97	0.000062	4.03	3752.21	214.30	0.17
RiverInflow	237.205*	24FEB2010 1100	15138.85	-25.00	5.73		5.98	0.000062	4.03	3754.23	214.36	0.17
RiverInflow	238.646*	24FEB2010 1100	15141.46	-25.00	5.74		5.99	0.000062	4.03	3756.24	214.42	0.17
RiverInflow	240.087*	24FEB2010 1100	15144.07	-25.00	5.75		6.00	0.000062	4.03	3758.26	214.47	0.17
RiverInflow	241.529*	24FEB2010 1100	15146.69	-25.00	5.76		6.01	0.000062	4.03	3760.27	214.53	0.17
RiverInflow	242.970*	24FEB2010 1100	15149.33	-25.00	5.76		6.02	0.000062	4.03	3762.28	214.59	0.17
RiverInflow	244.411*	24FEB2010 1100	15151.97	-25.00	5.77		6.03	0.000062	4.03	3764.29	214.64	0.17
RiverInflow	245.853*	24FEB2010 1100	15154.63	-25.00	5.78		6.03	0.000062	4.02	3766.29	214.70	0.17
RiverInflow	247.294*	24FEB2010 1100	15157.29	-25.00	5.79		6.04	0.000062	4.02	3768.30	214.75	0.17
RiverInflow	248.735*	24FEB2010 1100	15159.96	-25.00	5.80		6.05	0.000062	4.02	3770.30	214.81	0.17
RiverInflow	250.177*	24FEB2010 1100	15162.65	-25.00	5.81		6.06	0.000062	4.02	3772.30	214.87	0.17
RiverInflow	251.618*	24FEB2010 1100	15165.34	-25.00	5.82		6.07	0.000061	4.02	3774.30	214.92	0.17
RiverInflow	253.059*	24FEB2010 1100	15168.04	-25.00	5.83		6.08	0.000061	4.02	3776.30	214.98	0.17
RiverInflow	254.500*	24FEB2010 1100	15170.75	-25.00	5.84		6.09	0.000061	4.02	3778.30	215.03	0.17
RiverInflow	255.942*	24FEB2010 1100	15173.48	-25.00	5.85		6.10	0.000061	4.01	3780.29	215.09	0.17
RiverInflow	257.383*	24FEB2010 1100	15176.21	-25.00	5.86		6.11	0.000061	4.01	3782.28	215.15	0.17
RiverInflow	258.824*	24FEB2010 1100	15178.95	-25.00	5.87		6.12	0.000061	4.01	3784.28	215.20	0.17
RiverInflow	260.266*	24FEB2010 1100	15181.70	-25.00	5.88		6.13	0.000061	4.01	3786.27	215.26	0.17
RiverInflow	261.707*	24FEB2010 1100	15184.46	-25.00	5.89		6.13	0.000061	4.01	3788.26	215.31	0.17
RiverInflow	263.148*	24FEB2010 1100	15187.23	-25.00	5.89		6.14	0.000061	4.01	3790.24	215.37	0.17
RiverInflow	264.59	24FEB2010 1100	15190.01	-25.00	5.90		6.15	0.000061	4.01	3792.23	215.42	0.17
RiverInflow	265.09*	24FEB2010 1100	15190.96	-27.50	5.92		6.16	0.000052	3.87	3926.50	204.96	0.16
RiverInflow	265.59*	24FEB2010 1100	15191.90	-30.00	5.96		6.16	0.000040	3.56	4266.64	207.29	0.14
RiverInflow	266.09*	24FEB2010 1100	15192.86	-32.50	5.99		6.16	0.000032	3.30	4606.09	209.34	0.12
RiverInflow	266.59*	24FEB2010 1100	15193.84	-35.00	6.01		6.16	0.000026	3.07	4945.10	211.14	0.11
RiverInflow	267.09*	24FEB2010 1100	15194.84	-37.50	6.03		6.16	0.000021	2.88	5283.85	212.75	0.10
RiverInflow	267.59	24FEB2010 1100	15195.84	-40.00	6.05		6.16	0.000017	2.70	5622.43	214.19	0.09
RiverInflow	267.79	24FEB2010 1100	15196.06	-40.00	4.91		6.89	0.000515	11.29	1346.56	30.06	0.30
RiverInflow	268.14	24FEB2010 1100	15196.13	-40.00	4.94		6.91	0.000514	11.28	1347.25	30.06	0.30
RiverInflow	268.19	24FEB2010 1100	15196.14	-35.00	4.36		6.93	0.000703	12.88	1179.86	30.06	0.36

ATTACHMENT 8 – UNSTEADY FLOW COMPUTATIONS – 15,000 CFS DIVERSION (continued)

HEC-RAS Plan: Plan 04 River: MRDiversionII Reach: RiverInflow Profile: 24FEB2010 1100 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
RiverInflow	268.39	24FEB2010 1100	15196.16	-35.00	4.37		6.95	0.000702	12.87	1180.34	30.06	0.36
RiverInflow	268.44	24FEB2010 1100	15196.17	-40.00	4.96		6.93	0.000514	11.27	1347.91	30.06	0.30
RiverInflow	268.89	24FEB2010 1100	15196.25	-40.00	4.98		6.95	0.000513	11.27	1348.68	30.06	0.30
RiverInflow	269.09	24FEB2010 1100	15196.29	-40.00	4.99		6.96	0.000513	11.26	1349.02	30.06	0.30
RiverInflow	269.784*	24FEB2010 1100	15196.41	-40.00	5.03		7.00	0.000512	11.25	1350.49	30.07	0.30
RiverInflow	270.479*	24FEB2010 1100	15196.54	-40.00	5.07		7.04	0.000510	11.24	1351.95	30.07	0.30
RiverInflow	271.173*	24FEB2010 1100	15196.68	-40.00	5.11		7.07	0.000509	11.23	1353.12	30.07	0.30
RiverInflow	271.868*	24FEB2010 1100	15196.81	-40.00	5.15		7.11	0.000508	11.22	1354.58	30.07	0.29
RiverInflow	272.562*	24FEB2010 1100	15196.95	-40.00	5.19		7.14	0.000507	11.21	1356.04	30.07	0.29
RiverInflow	273.257*	24FEB2010 1100	15197.10	-40.00	5.23		7.18	0.000505	11.19	1357.49	30.07	0.29
RiverInflow	273.951*	24FEB2010 1100	15197.24	-40.00	5.27		7.21	0.000504	11.18	1358.94	30.08	0.29
RiverInflow	274.646*	24FEB2010 1100	15197.39	-40.00	5.31		7.25	0.000503	11.17	1360.39	30.08	0.29
RiverInflow	275.340*	24FEB2010 1100	15197.55	-40.00	5.35		7.28	0.000502	11.16	1361.54	30.08	0.29
RiverInflow	276.035*	24FEB2010 1100	15197.70	-40.00	5.39		7.32	0.000500	11.15	1362.98	30.08	0.29
RiverInflow	276.73	24FEB2010 1100	15197.86	-40.00	5.43		7.35	0.000499	11.14	1364.42	30.08	0.29
RiverInflow	276.93	24FEB2010 1100	15197.91	-40.00	5.44		7.37	0.001177	11.15	1362.78	30.08	0.29
RiverInflow	277.73	24FEB2010 1100	15198.24	-40.00	6.47		7.38	0.000366	7.62	1994.19	50.82	0.21
RiverInflow	278.55	24FEB2010 1100	15199.03	-40.00	7.02		7.35	0.000085	4.64	3277.03	99.56	0.14
RiverInflow	279.50	24FEB2010 1100	15202.94	-40.00	7.23		7.25	0.000005	1.25	12131.82	494.78	0.04
RiverInflow	280.59*	24FEB2010 1100	15212.03	-40.00	7.24		7.25	0.000003	0.95	15974.45	709.41	0.04
RiverInflow	281.68	24FEB2010 1100	15223.52	-40.00	7.24		7.25	0.000002	0.74	20685.84	813.61	0.03
RiverInflow	282.91*	24FEB2010 1100	15238.69	-40.00	7.25		7.25	0.000001	0.57	26771.75	968.83	0.02
RiverInflow	284.14	24FEB2010 1100	15256.54	-40.00	7.25		7.25	0.000001	0.45	33610.57	1124.96	0.01
RiverInflow	285.09*	24FEB2010 1100	15271.74	-40.00	7.25		7.25	0.000000	0.42	36200.52	1182.42	0.01
RiverInflow	286.04	24FEB2010 1100	15287.72	-40.00	7.25		7.25	0.000000	0.39	38849.47	1239.97	0.01
RiverInflow	287.183*	24FEB2010 1100	15308.12	-40.00	7.25		7.25	0.000000	0.36	42446.23	1332.41	0.01
RiverInflow	288.326*	24FEB2010 1100	15330.20	-40.00	7.25		7.25	0.000000	0.33	46340.63	1424.91	0.01
RiverInflow	289.47	24FEB2010 1100	15353.55	-40.00	7.25		7.25	0.000000	0.30	50542.67	1517.67	0.01

ATTACHMENT 8 – UNSTEADY FLOW COMPUTATIONS – 15,000 CFS DIVERSION (continued)



ATTACHMENT 9 – MODULAR DESIGN DISCHARGE 70,000 CFS

HEC-RAS Plan: Plan 04 River: MRDiversionII Reach: RiverInflow Profile: 24FEB2010 0700

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
RiverInflow	0.00	24FEB2010 0700	50521.04	-2.00	3.04	-0.65	3.09	0.000200	1.75	28945.16	5840.04	0.14
RiverInflow	9.96000*	24FEB2010 0700	51080.45	-2.00	3.24		3.28	0.000180	1.70	30097.48	5848.94	0.13
RiverInflow	19.92*	24FEB2010 0700	51641.83	-2.00	3.41		3.45	0.000164	1.66	31144.01	5856.62	0.13
RiverInflow	29.88*	24FEB2010 0700	52206.37	-2.00	3.57		3.61	0.000152	1.63	32108.02	5863.37	0.12
RiverInflow	39.84*	24FEB2010 0700	52774.32	-2.00	3.72		3.76	0.000142	1.60	33006.18	5869.08	0.12
RiverInflow	49.8*	24FEB2010 0700	53345.52	-2.00	3.86		3.90	0.000133	1.58	33849.27	5874.18	0.12
RiverInflow	59.76*	24FEB2010 0700	53920.31	-2.00	3.99		4.02	0.000126	1.56	34646.46	5878.70	0.11
RiverInflow	69.72*	24FEB2010 0700	54498.48	-2.00	4.11		4.15	0.000120	1.54	35404.62	5882.48	0.11
RiverInflow	79.68*	24FEB2010 0700	55079.61	-2.00	4.23		4.27	0.000115	1.52	36129.34	5885.86	0.11
RiverInflow	89.64*	24FEB2010 0700	55663.65	-2.00	4.34		4.38	0.000110	1.51	36825.14	5888.81	0.11
RiverInflow	99.60	24FEB2010 0700	56250.61	-2.00	4.45		4.49	0.000106	1.50	37494.10	5891.25	0.10
RiverInflow	99.80	24FEB2010 0700	56262.45	-2.00	4.45		4.49	0.000138	1.50	37548.90	5903.67	0.10
RiverInflow	102.181*	24FEB2010 0700	56401.08	-3.10	4.49		4.52	0.000089	1.33	42340.85	5693.45	0.09
RiverInflow	104.561*	24FEB2010 0700	56535.07	-4.19	4.51		4.53	0.000062	1.22	46484.78	5482.38	0.07
RiverInflow	106.942*	24FEB2010 0700	56664.34	-5.29	4.53		4.55	0.000044	1.13	50119.82	5265.78	0.06
RiverInflow	109.323*	24FEB2010 0700	56788.77	-6.38	4.54		4.56	0.000034	1.07	53161.27	5049.72	0.06
RiverInflow	111.704*	24FEB2010 0700	56908.27	-7.48	4.55		4.56	0.000028	1.02	55716.90	4834.22	0.05
RiverInflow	114.085*	24FEB2010 0700	57022.77	-8.57	4.55		4.57	0.000023	0.99	57701.52	4619.44	0.05
RiverInflow	116.466*	24FEB2010 0700	57132.22	-9.67	4.56		4.58	0.000019	0.97	59203.71	4405.36	0.05
RiverInflow	118.847*	24FEB2010 0700	57236.62	-10.76	4.57		4.58	0.000017	0.95	60146.27	4192.19	0.04
RiverInflow	121.228*	24FEB2010 0700	57336.03	-11.86	4.57		4.58	0.000016	0.95	60606.53	3979.96	0.04
RiverInflow	123.609*	24FEB2010 0700	57430.40	-12.95	4.57		4.59	0.000015	0.95	60516.57	3768.89	0.04
RiverInflow	125.990*	24FEB2010 0700	57519.67	-14.05	4.58		4.59	0.000013	0.96	59941.24	3557.57	0.04
RiverInflow	128.371*	24FEB2010 0700	57603.85	-15.14	4.58		4.59	0.000013	0.98	58822.44	3345.86	0.04
RiverInflow	130.752*	24FEB2010 0700	57682.90	-16.24	4.58		4.60	0.000013	1.01	57214.18	3134.16	0.04
RiverInflow	133.133*	24FEB2010 0700	57756.82	-17.33	4.58		4.60	0.000014	1.05	55068.64	2922.45	0.04
RiverInflow	135.514*	24FEB2010 0700	57825.64	-18.43	4.59		4.60	0.000014	1.10	52428.17	2710.75	0.04
RiverInflow	137.895*	24FEB2010 0700	57889.38	-19.52	4.59		4.61	0.000015	1.18	49255.83	2499.02	0.05
RiverInflow	140.276*	24FEB2010 0700	57947.91	-20.62	4.59		4.61	0.000017	1.27	45583.43	2287.31	0.05
RiverInflow	142.657*	24FEB2010 0700	58001.24	-21.71	4.59		4.62	0.000021	1.40	41383.50	2075.51	0.05
RiverInflow	145.038*	24FEB2010 0700	58049.37	-22.81	4.58		4.62	0.000026	1.58	36677.01	1863.76	0.06
RiverInflow	147.419*	24FEB2010 0700	58092.27	-23.90	4.58		4.63	0.000036	1.85	31446.34	1651.98	0.07
RiverInflow	149.80	24FEB2010 0700	58129.84	-25.00	4.56		4.64	0.000059	2.27	25698.26	1440.12	0.09
RiverInflow	150.00	24FEB2010 0700	58132.78	-25.00	4.57		4.64	0.000018	2.16	27044.65	1450.57	0.09
RiverInflow	154.778*	24FEB2010 0700	58203.42	-25.00	4.58		4.65	0.000018	2.17	26976.99	1450.10	0.09
RiverInflow	159.557*	24FEB2010 0700	58274.23	-25.00	4.59		4.66	0.000018	2.17	26909.49	1449.62	0.09
RiverInflow	164.336*	24FEB2010 0700	58345.21	-25.00	4.60		4.67	0.000019	2.18	26848.88	1449.18	0.09
RiverInflow	169.115*	24FEB2010 0700	58416.37	-25.00	4.61		4.68	0.000019	2.19	26781.67	1448.69	0.09
RiverInflow	173.894*	24FEB2010 0700	58487.70	-25.00	4.62		4.69	0.000019	2.20	26714.63	1448.19	0.09
RiverInflow	178.672*	24FEB2010 0700	58559.21	-25.00	4.63		4.70	0.000019	2.21	26647.74	1447.68	0.09
RiverInflow	183.451*	24FEB2010 0700	58630.90	-25.00	4.64		4.71	0.000019	2.21	26587.76	1447.21	0.09
RiverInflow	188.230*	24FEB2010 0700	58702.77	-25.00	4.64		4.72	0.000020	2.22	26521.19	1446.69	0.09
RiverInflow	193.009*	24FEB2010 0700	58774.80	-25.00	4.65		4.73	0.000020	2.23	26454.79	1446.16	0.09

ATTACHMENT 10 – UNSTEADY FLOW COMPUTATIONS – 50,000 TO 60,000 CFS DIVERSION

HEC-RAS Plan: Plan 04 River: MRDiversionII Reach: RiverInflow Profile: 24FEB2010 0700 (Continued)

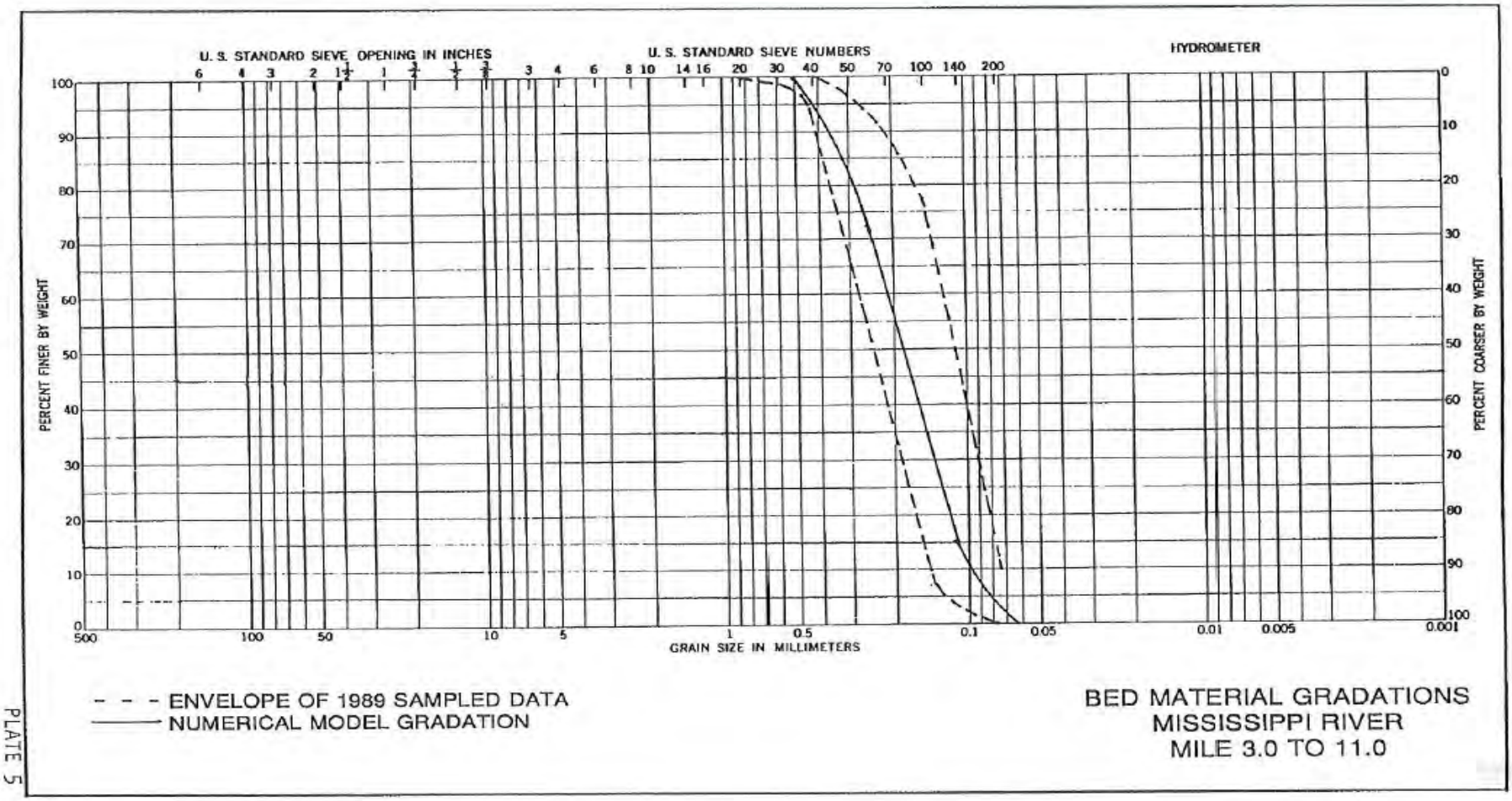
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
RiverInflow	197.788*	24FEB2010 0700	58847.03	-25.00	4.66		4.74	0.000020	2.24	26388.56	1445.61	0.09
RiverInflow	202.567*	24FEB2010 0700	58919.43	-25.00	4.67		4.75	0.000020	2.25	26329.23	1445.12	0.09
RiverInflow	207.345*	24FEB2010 0700	58992.01	-25.00	4.68		4.76	0.000020	2.25	26263.34	1444.56	0.09
RiverInflow	212.124*	24FEB2010 0700	59064.77	-25.00	4.69		4.77	0.000021	2.26	26197.62	1444.00	0.09
RiverInflow	216.903*	24FEB2010 0700	59137.71	-25.00	4.70		4.78	0.000021	2.27	26132.08	1443.42	0.09
RiverInflow	221.682*	24FEB2010 0700	59210.83	-25.00	4.71		4.79	0.000021	2.28	26073.43	1442.89	0.09
RiverInflow	226.461*	24FEB2010 0700	59284.13	-25.00	4.72		4.80	0.000021	2.29	26008.25	1442.30	0.09
RiverInflow	231.24	24FEB2010 0700	59357.62	-25.00	4.73		4.82	0.000022	2.29	25943.25	1441.69	0.09
RiverInflow	231.44	24FEB2010 0700	59360.13	-40.00	4.73		4.82	0.000011	2.40	24733.25	904.30	0.08
RiverInflow	233.342*	24FEB2010 0700	59378.47	-40.00	4.73		4.82	0.000011	2.40	24735.42	904.31	0.08
RiverInflow	235.245*	24FEB2010 0700	59396.83	-40.00	4.73		4.82	0.000011	2.40	24737.59	904.32	0.08
RiverInflow	237.147*	24FEB2010 0700	59415.20	-40.00	4.73		4.82	0.000011	2.41	24739.76	904.33	0.08
RiverInflow	239.050*	24FEB2010 0700	59433.59	-40.00	4.74		4.83	0.000011	2.41	24741.94	904.34	0.08
RiverInflow	240.953*	24FEB2010 0700	59452.00	-40.00	4.74		4.83	0.000011	2.41	24744.11	904.35	0.08
RiverInflow	242.855*	24FEB2010 0700	59470.41	-40.00	4.74		4.83	0.000011	2.41	24746.28	904.36	0.08
RiverInflow	244.758*	24FEB2010 0700	59488.85	-40.00	4.74		4.83	0.000011	2.41	24748.45	904.37	0.08
RiverInflow	246.661*	24FEB2010 0700	59507.29	-40.00	4.75		4.84	0.000011	2.41	24750.63	904.38	0.08
RiverInflow	248.563*	24FEB2010 0700	59525.75	-40.00	4.75		4.84	0.000011	2.41	24752.80	904.39	0.08
RiverInflow	250.466*	24FEB2010 0700	59544.23	-40.00	4.75		4.84	0.000011	2.41	24754.98	904.40	0.08
RiverInflow	252.368*	24FEB2010 0700	59562.71	-40.00	4.75		4.84	0.000011	2.41	24757.15	904.41	0.08
RiverInflow	254.271*	24FEB2010 0700	59581.22	-40.00	4.76		4.85	0.000011	2.41	24759.33	904.42	0.08
RiverInflow	256.174*	24FEB2010 0700	59599.73	-40.00	4.76		4.85	0.000011	2.41	24761.50	904.43	0.08
RiverInflow	258.076*	24FEB2010 0700	59618.27	-40.00	4.76		4.85	0.000011	2.41	24763.68	904.44	0.08
RiverInflow	259.979*	24FEB2010 0700	59636.81	-40.00	4.76		4.85	0.000011	2.41	24765.86	904.45	0.08
RiverInflow	261.882*	24FEB2010 0700	59655.38	-40.00	4.76		4.86	0.000011	2.41	24768.04	904.46	0.08
RiverInflow	263.784*	24FEB2010 0700	59673.95	-40.00	4.77		4.86	0.000011	2.41	24770.22	904.47	0.08
RiverInflow	265.687*	24FEB2010 0700	59692.54	-40.00	4.77		4.86	0.000011	2.41	24772.40	904.48	0.08
RiverInflow	267.59	24FEB2010 0700	59711.14	-40.00	4.77		4.86	0.000011	2.41	24774.58	904.49	0.08
RiverInflow	267.79	24FEB2010 0700	59712.22	-40.00	3.91		5.39	0.000120	9.75	6126.96	139.72	0.26
RiverInflow	268.14	24FEB2010 0700	59712.63	-40.00	3.92		5.39	0.000120	9.74	6127.71	139.72	0.26
RiverInflow	268.19	24FEB2010 0700	59712.68	-35.00	3.49		5.41	0.000174	11.12	5369.51	139.70	0.32
RiverInflow	268.39	24FEB2010 0700	59712.83	-35.00	3.49		5.41	0.000174	11.12	5370.23	139.71	0.32
RiverInflow	268.44	24FEB2010 0700	59712.88	-40.00	3.92		5.40	0.000120	9.74	6128.48	139.72	0.26
RiverInflow	268.89	24FEB2010 0700	59713.34	-40.00	3.93		5.40	0.000120	9.74	6129.32	139.72	0.26
RiverInflow	269.09	24FEB2010 0700	59713.54	-40.00	3.93		5.40	0.000120	9.74	6129.70	139.72	0.26
RiverInflow	269.784*	24FEB2010 0700	59714.27	-40.00	3.94		5.41	0.000119	9.74	6131.55	139.72	0.26
RiverInflow	270.479*	24FEB2010 0700	59714.99	-40.00	3.95		5.42	0.000119	9.74	6133.13	139.73	0.26
RiverInflow	271.173*	24FEB2010 0700	59715.72	-40.00	3.96		5.43	0.000119	9.73	6134.98	139.73	0.26
RiverInflow	271.868*	24FEB2010 0700	59716.45	-40.00	3.97		5.44	0.000119	9.73	6136.83	139.73	0.26
RiverInflow	272.562*	24FEB2010 0700	59717.19	-40.00	3.98		5.45	0.000119	9.73	6138.40	139.74	0.26
RiverInflow	273.257*	24FEB2010 0700	59717.93	-40.00	3.99		5.46	0.000119	9.73	6140.25	139.74	0.26
RiverInflow	273.951*	24FEB2010 0700	59718.68	-40.00	4.00		5.47	0.000119	9.72	6141.82	139.74	0.26
RiverInflow	274.646*	24FEB2010 0700	59719.43	-40.00	4.01		5.47	0.000119	9.72	6143.67	139.74	0.26

ATTACHMENT 10 – UNSTEADY FLOW COMPUTATIONS – 50,000 TO 60,000 CFS DIVERSION (continued)

HEC-RAS Plan: Plan 04 River: MRDiversionII Reach: RiverInflow Profile: 24FEB2010 0700 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
RiverInflow	275.340*	24FEB2010 0700	59720.18	-40.00	4.02		5.48	0.000119	9.72	6145.51	139.75	0.26
RiverInflow	276.035*	24FEB2010 0700	59720.94	-40.00	4.03		5.49	0.000119	9.72	6147.08	139.75	0.26
RiverInflow	276.73	24FEB2010 0700	59721.70	-40.00	4.04		5.50	0.000119	9.71	6148.93	139.75	0.26
RiverInflow	276.93	24FEB2010 0700	59721.92	-40.00	4.04		5.50	0.000278	9.71	6150.23	139.96	0.26
RiverInflow	277.73	24FEB2010 0700	59723.00	-40.00	4.38		5.51	0.000193	8.56	6979.38	169.55	0.24
RiverInflow	278.55	24FEB2010 0700	59724.68	-40.00	4.72		5.52	0.000133	7.15	8349.01	233.37	0.21
RiverInflow	279.50	24FEB2010 0700	59728.62	-40.00	5.06		5.51	0.000105	5.38	11098.26	466.16	0.19
RiverInflow	280.59*	24FEB2010 0700	59736.52	-40.00	5.26		5.52	0.000068	4.09	14602.84	673.13	0.15
RiverInflow	281.68	24FEB2010 0700	59747.07	-40.00	5.37		5.52	0.000034	3.11	19181.38	791.38	0.11
RiverInflow	282.91*	24FEB2010 0700	59761.48	-40.00	5.43		5.52	0.000018	2.39	25034.90	945.45	0.08
RiverInflow	284.14	24FEB2010 0700	59778.66	-40.00	5.47		5.52	0.000010	1.89	31627.18	1100.37	0.06
RiverInflow	285.09*	24FEB2010 0700	59793.41	-40.00	5.47		5.52	0.000008	1.75	34123.90	1158.21	0.06
RiverInflow	286.04	24FEB2010 0700	59808.93	-40.00	5.48		5.52	0.000007	1.63	36678.61	1216.15	0.05
RiverInflow	287.183*	24FEB2010 0700	59828.92	-40.00	5.49		5.52	0.000006	1.49	40121.12	1308.73	0.05
RiverInflow	288.326*	24FEB2010 0700	59850.33	-40.00	5.49		5.52	0.000005	1.36	43860.78	1401.38	0.04
RiverInflow	289.47	24FEB2010 0700	59873.27	-40.00	5.50		5.52	0.000004	1.25	47907.17	1494.33	0.04

ATTACHMENT 10 – UNSTEADY FLOW COMPUTATIONS – 50,000 TO 60,000 CFS DIVERSION (continued)



ATTACHMENT 11 – TYPICAL BED MATERIAL GRAIN SIZES IN THE LOWER MISSISSIPPI RIVER

SandCalc - [Total Load - Currents - van Rijn (formula)]

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

01/03/10 09:16:19

OUTPUT PARAMETERS	
total volumetric sediment transport rate in current direction (m^2/s)	1.217E-05

INPUT PARAMETERS	
water depth (m)	14
kinematic viscosity of water (m^2/s)	1.156E-06
ratio of densities of grain and water (-)	2.65
median grain diameter d50 (mm)	.19
d90 grain diameter (mm)	.3441
depth-averaged current speed (m/s)	.61

ATTACHMENT 12 – DEPTH AVERAGED VELOCITY OF 2 FPS

SandCalc - [Total Load - Currents - van Rijn (formula)]

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

01/03/10 09:52:51

OUTPUT PARAMETERS

total volumetric sediment transport rate in current direction (m^2/s)	.0000614

INPUT PARAMETERS

water depth (m)	14
kinematic viscosity of water (m^2/s)	1.156E-06
ratio of densities of grain and water (-)	2.65
median grain diameter d50 (mm)	.19
d90 grain diameter (mm)	.3441
depth-averaged current speed (m/s)	.76

ATTACHMENT 13 – DEPTH AVERAGED VELOCITY OF 2.5 FPS

SandCalc - [Total Load - Currents - van Rijn (formula)]

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

01/03/10 10:02:18

OUTPUT PARAMETERS

total volumetric sediment transport rate in current direction (m^2/s)	.0001822

INPUT PARAMETERS

water depth (m)	14
kinematic viscosity of water (m^2/s)	1.156E-06
ratio of densities of grain and water (-)	2.65
median grain diameter d50 (mm)	.19
d90 grain diameter (mm)	.3441
depth-averaged current speed (m/s)	.915

ATTACHMENT 14 – DEPTH AVERAGED VELOCITY OF 3 FPS

SandCalc - [Total Load - Currents - van Rijn (formula)]

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

01/03/10 10:09:04

OUTPUT PARAMETERS	
total volumetric sediment transport rate in current direction (m^2/s)	.002059

INPUT PARAMETERS	
water depth (m)	14
kinematic viscosity of water (m^2/s)	1.156E-06
ratio of densities of grain and water (-)	2.65
median grain diameter d50 (mm)	.19
d90 grain diameter (mm)	.3441
depth-averaged current speed (m/s)	1.52

ATTACHMENT 15 – DEPTH AVERAGED VELOCITY OF 5 FPS

SandCalc - [Total Load - Currents - van Rijn (formula)]

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

01/03/10 10:12:50

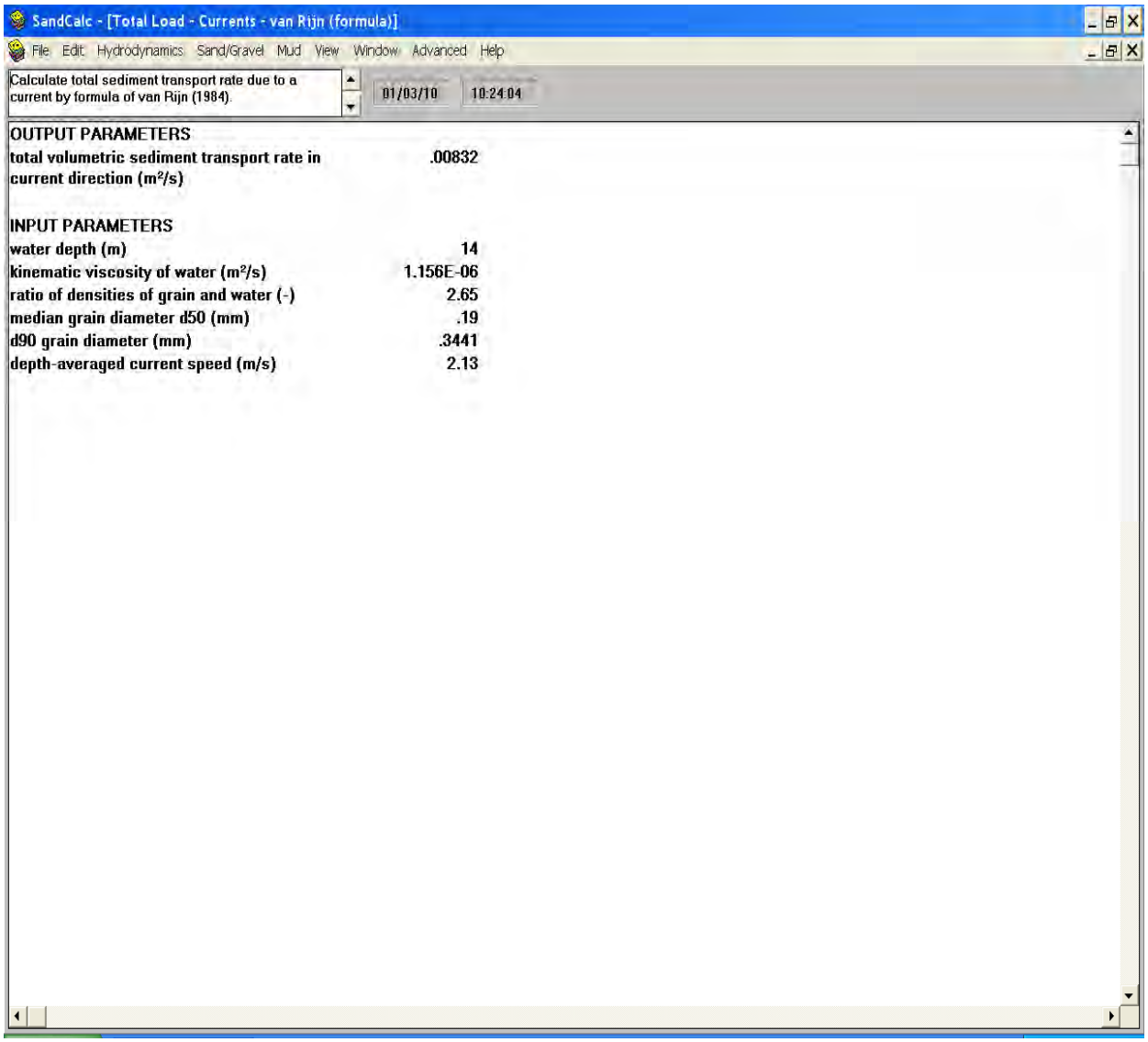
OUTPUT PARAMETERS

total volumetric sediment transport rate in current direction (m^2/s)	.004498

INPUT PARAMETERS

water depth (m)	14
kinematic viscosity of water (m^2/s)	1.156E-06
ratio of densities of grain and water (-)	2.65
median grain diameter d50 (mm)	.19
d90 grain diameter (mm)	.3441
depth-averaged current speed (m/s)	1.83

ATTACHMENT 16 – DEPTH AVERAGED VELOCITY OF 6 FPS



SandCalc - [Total Load - Currents - van Rijn (formula)]

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

01/03/10 10:24:04

OUTPUT PARAMETERS

total volumetric sediment transport rate in current direction (m^2/s)	.00832

INPUT PARAMETERS

water depth (m)	14
kinematic viscosity of water (m^2/s)	1.156E-06
ratio of densities of grain and water (-)	2.65
median grain diameter d50 (mm)	.19
d90 grain diameter (mm)	.3441
depth-averaged current speed (m/s)	2.13

ATTACHMENT 17 – DEPTH AVERAGED VELOCITY OF 7 FPS

SandCalc - [Total Load - Currents - van Rijn (formula)]

File Edit Hydrodynamics Sand/Gravel Mud View Window Advanced Help

Calculate total sediment transport rate due to a current by formula of van Rijn (1984).

01/03/10 10:17:56

OUTPUT PARAMETERS

total volumetric sediment transport rate in current direction (m^2/s)	.01422

INPUT PARAMETERS

water depth (m)	14
kinematic viscosity of water (m^2/s)	1.156E-06
ratio of densities of grain and water (-)	2.65
median grain diameter d50 (mm)	.19
d90 grain diameter (mm)	.3441
depth-averaged current speed (m/s)	2.44

ATTACHMENT 18 – DEPTH AVERAGED VELOCITY OF 8 FPS

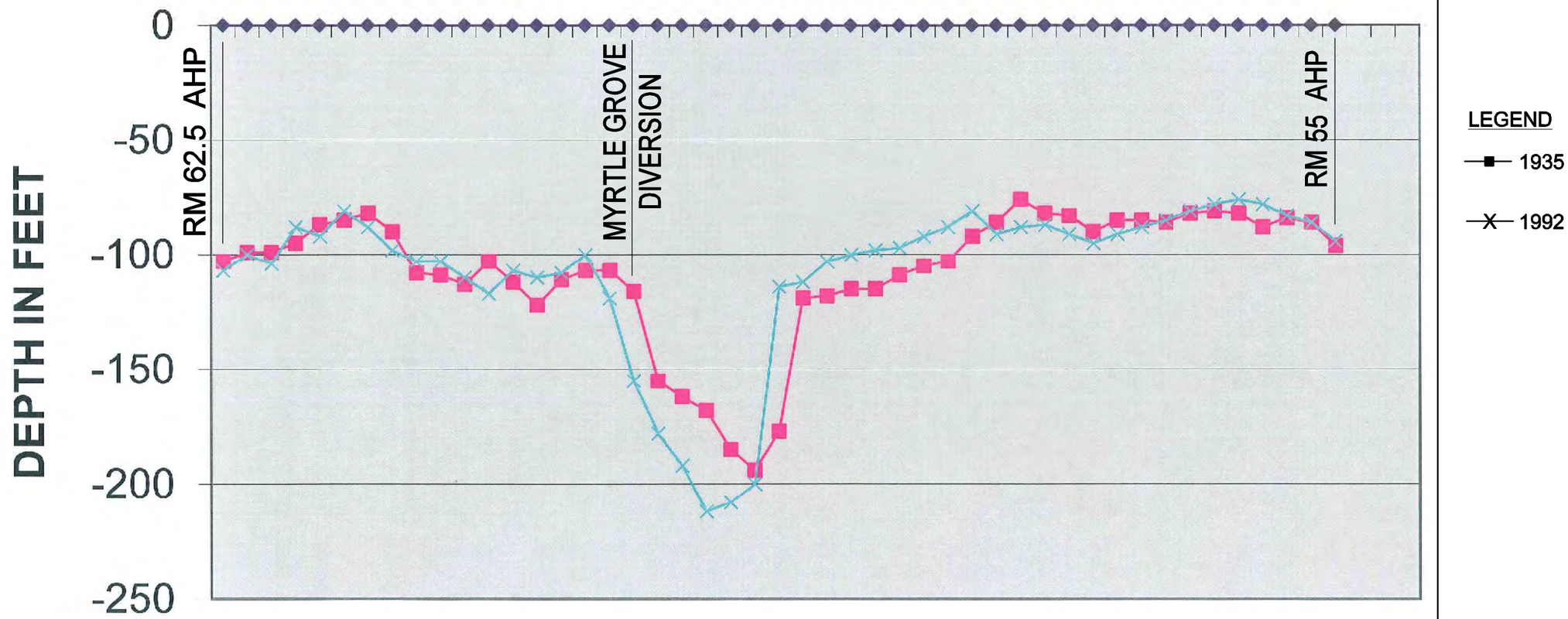
ATTACHMENT 19 – PLOT OF DIVERSION FLOW VERSUS DAYS FLOW EXCEEDS 600,000 CFS IN MISSISSIPPI.

ATTACHMENT 20 – UNSTEADY FLOW DIVERSION MODEL FOR 15,000 CFS AT MILE 60.7 – MYRTLE GROVE, LA

APPENDIX B

Drawings 1 to 17

THALWEG PROFILE, MISS. RIVER, MILE 62.5 TO 55.0, 1935 vs 1992



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OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Carmouch
PLOT SCALE: 1" = 1000'
PLOT DATE: Mar 2010
SIZE: ANSI B
FILE NAME: G-9530cover.dgn

BCG Engineering & Consulting, Inc. Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

THALWEG PROFILE, MISSISSIPPI RIVER
MILE 62.5 TO 55.0, 1935 vs 1992
VOLUME II

SHEET IDENTIFICATION
1

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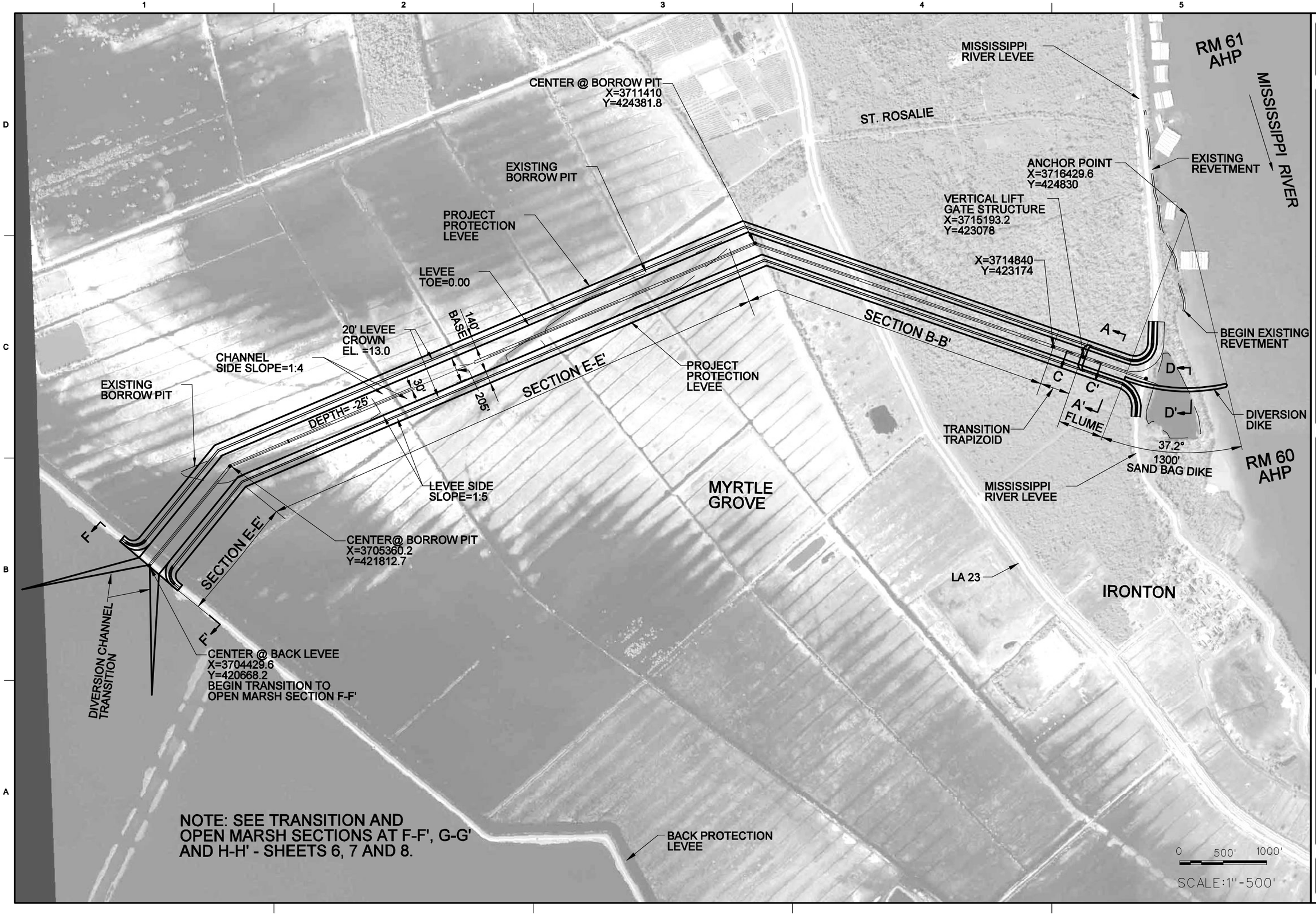

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COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Gernoch
PLOT SCALE: 1:1
PLOT DATE: 1-11-2011
SIZE: ANSI D
FILE NAME: G-SBDxxx.dgn

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MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

MYRTLE GROVE AUTHORIZED DIVERSION CHANNEL

CONCEPTUAL PLAN - VOLUME II

SHEET IDENTIFICATION
2

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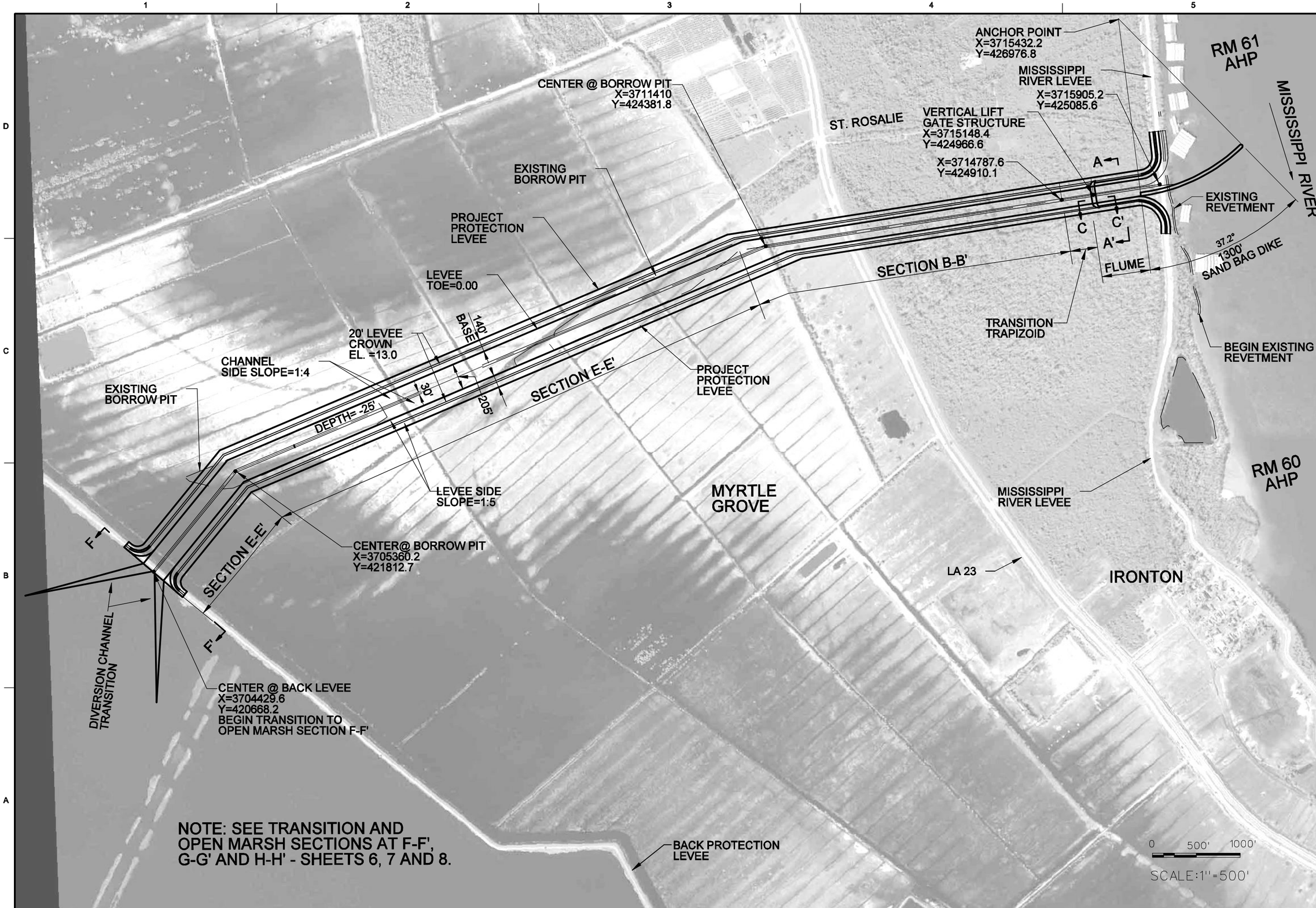

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Gemsch
PLOT SCALE: 1:1
PLOT DATE: 1/11/2011
DATE: MAY, 2010
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MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

MYRTLE GROVE OPTION 2
DIVERSION CHANNEL

CONCEPTUAL PLAN - VOLUME II

SHEET IDENTIFICATION
2A

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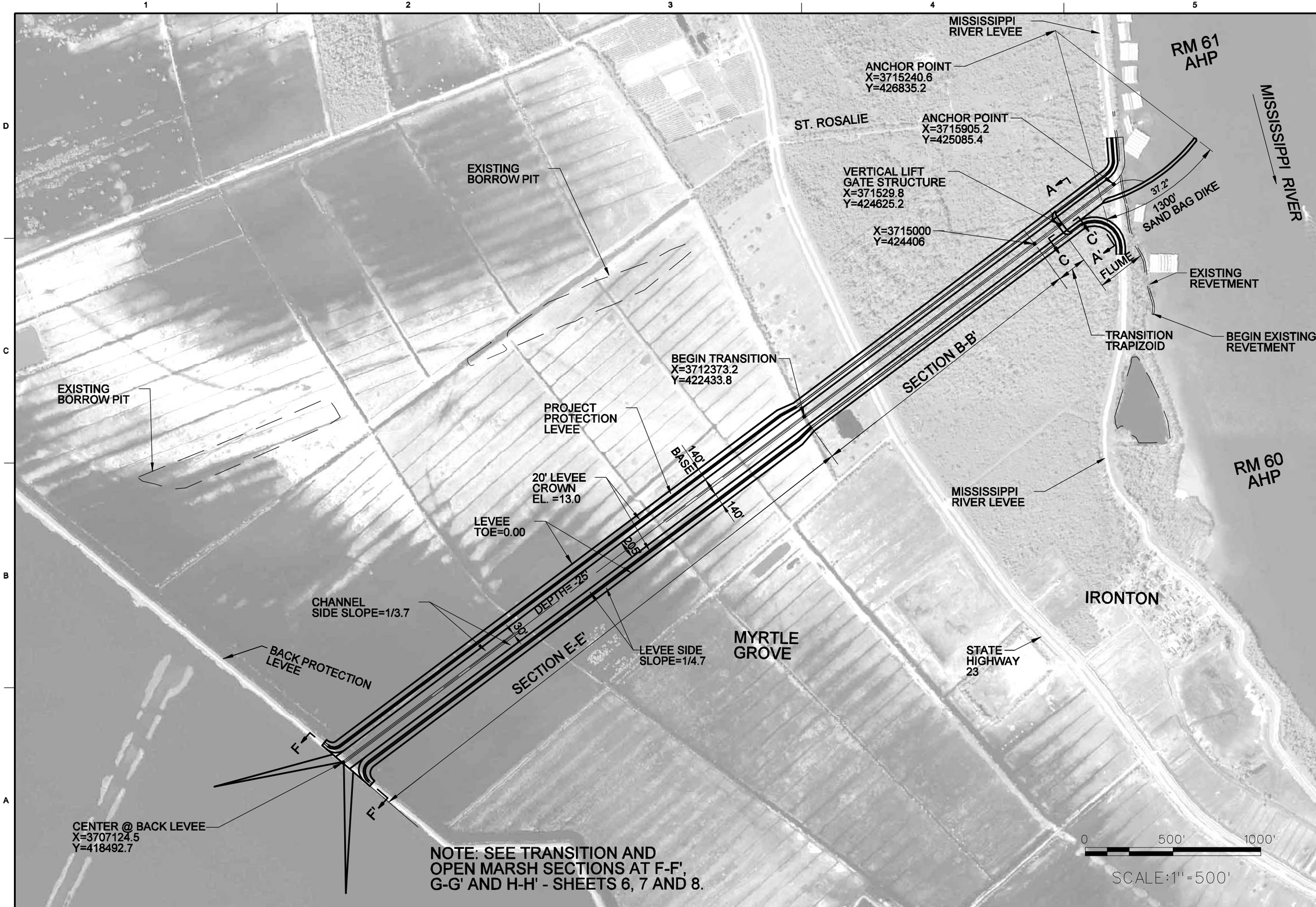

NOTE: SEE TRANSITION AND OPEN MARSH SECTIONS AT F-F', G-G' AND H-H' - SHEETS 6, 7 AND 8.

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
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DESIGNED BY: Cecil Boileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Germach
PLOT SCALE: 1" = 500'
PLOT DATE: May 2010
SIZE: ANSI D
FILE NAME: G-B2Doc.dgn

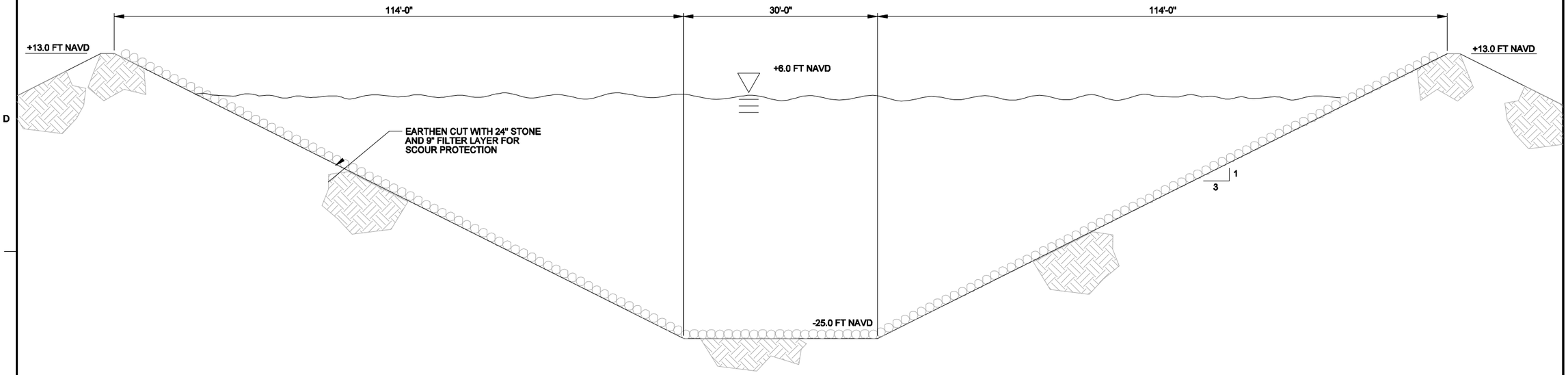
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

MYRTLE GROVE OPTION 3
DIVERSION CHANNEL

CONCEPTUAL PLAN - VOLUME II

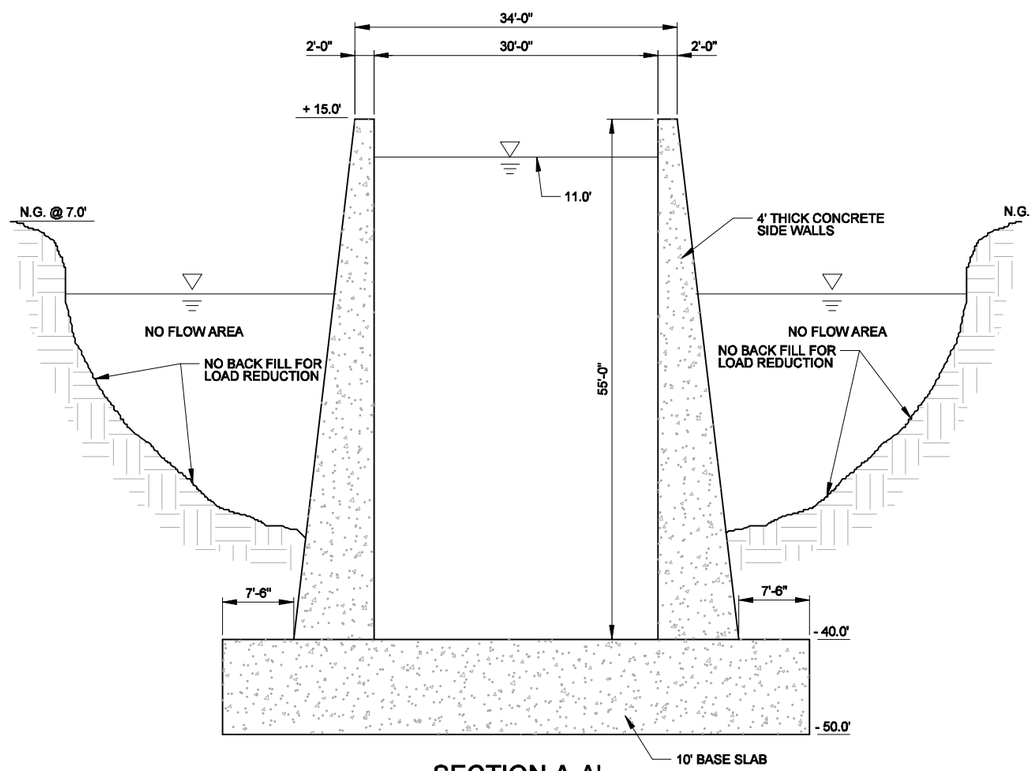
SHEET IDENTIFICATION
2B

auto_time



TYPICAL SECTION B-B'

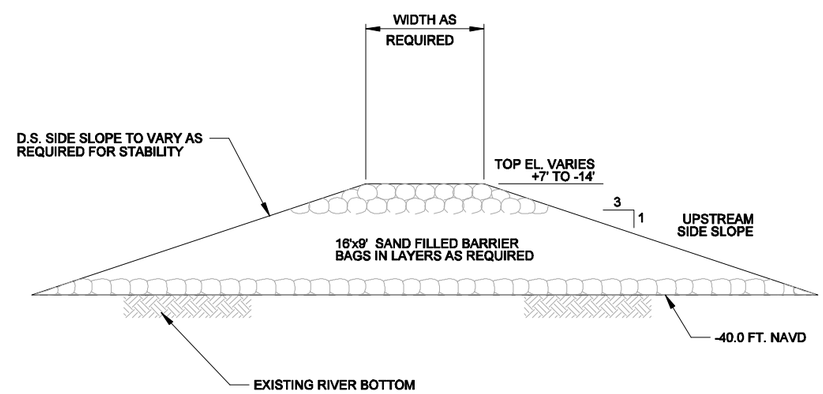
N.T.S.



SECTION A-A'

30' X 49' DIVERSION CANAL BETWEEN MISSISSIPPI RIVER AND CONTROL STRUCTURE

N.T.S.



TYPICAL SECTION D-D'

N.T.S.

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Cannon
PLOT SCALE: NTS
PLOT DATE: May 2010
SIZE: ANSI D
FILE NAME: FIG 2 40X45 DIVERSION CANAL

BCG Engineering & Consulting, Inc. Excellence in Engineering

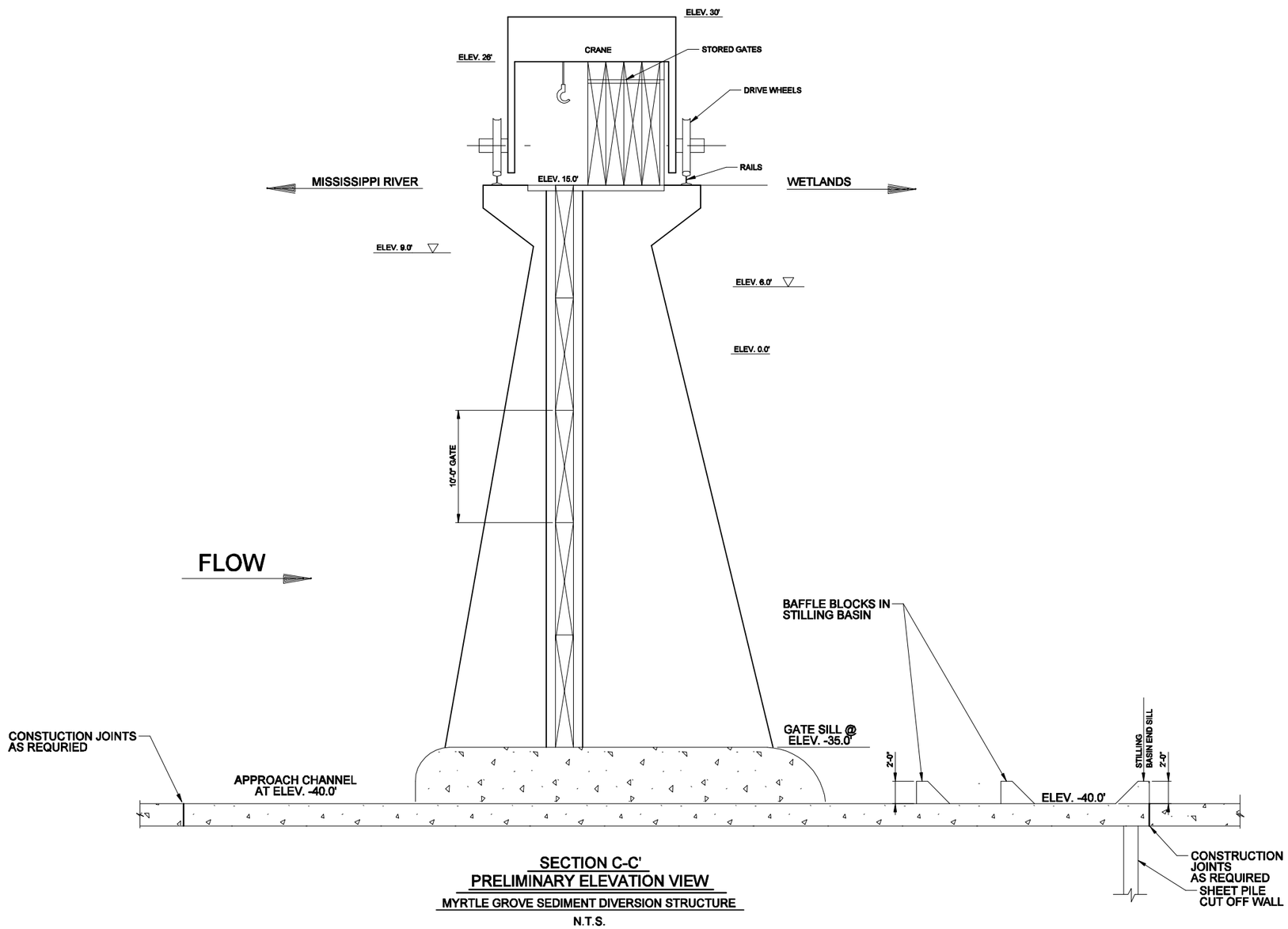
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

TYPICAL SECTION
VOLUME II

SHEET IDENTIFICATION
3

auto_time

...\FIG 3_40 X 45 DIVERSION CANAL.dgn 01/11/2011 2:48:41 PM



SECTION C-C'
PRELIMINARY ELEVATION VIEW
MYRTLE GROVE SEDIMENT DIVERSION STRUCTURE
N.T.S.

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Cernach
PLOT SCALE: 1:1 or 1" = 1'-0"
PLOT DATE: Wednesday
SIZE: ANSI D
FILE NAME: FIG 3

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Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

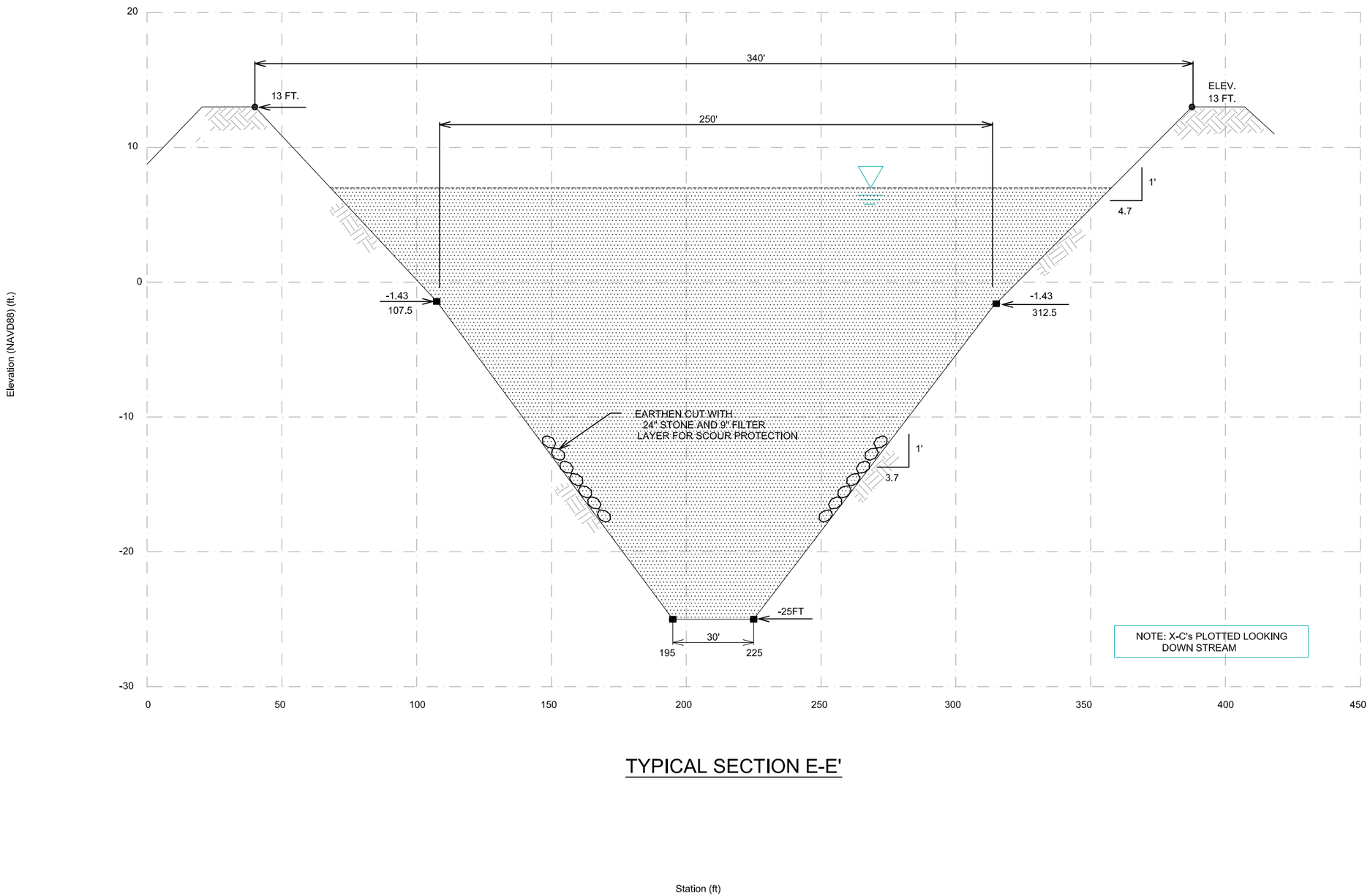
TYPICAL SECTION
VOLUME II

SHEET IDENTIFICATION
4

...\VOLUME 2\FIG 4.dgn 01/11/2011 2:49:33 PM

RS=228.338

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



Legend

- EG Max WS.
- WS Max WS.
- Ground.
- Bank Sta.

TYPICAL SECTION E-E'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: CWS
DWN BY: EWS
CKD BY: DWS
SUBMITTED BY: Rodney J. Guensch
DATE: MAY 2010
PLOT SCALE:
PLOT DATE: MAY 2010
SIZE: ANSI D
FILE NAME:

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=228.338

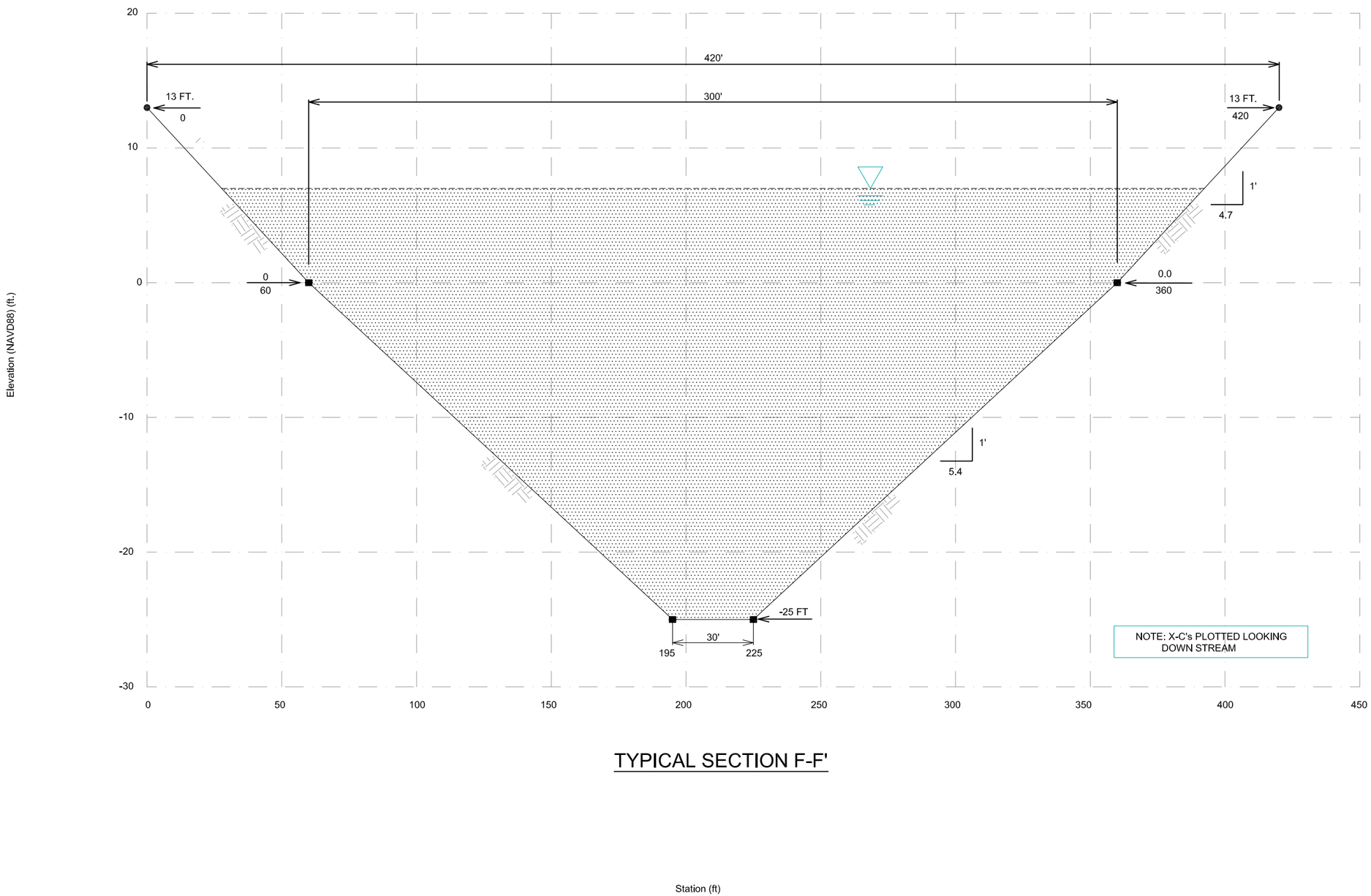
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION

SHEET IDENTIFICATION
5

auto_time

RS=149.80 BEGIN TRANSITION TO OPEN MARSH

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



TYPICAL SECTION F-F'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: CWS
DWN BY: CWS
CKD BY: CWS
SUBMITTED BY: Rodney J. Guenich
DATE: MAY 2010
PLOT SCALE:
PLOT DATE: MAY 2010
SIZE: ANSI D
FILE NAME:

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

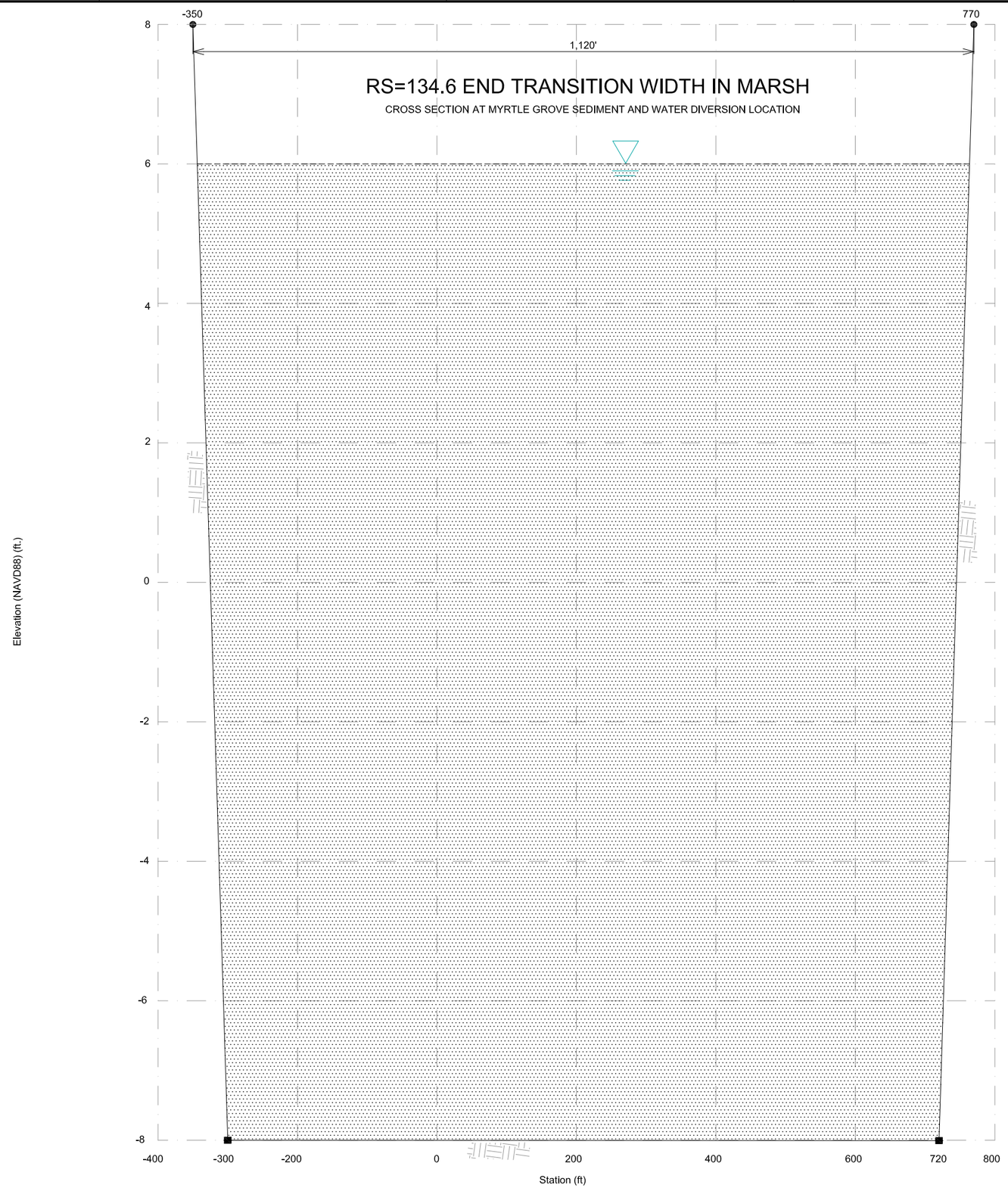
BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=149.80 BEGIN TRANSITION TO OPEN MARSH
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION

SHEET IDENTIFICATION
6

auto_time



Legend
EG Max WS.
WS Max WS.
Ground.
Bank Sta.

NOTE: X-C's PLOTTED LOOKING DOWN STREAM

TYPICAL SECTION G-G'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

DESIGNED BY: CWS
DWN BY: DOS
CKD BY: CWS
SUBMITTED BY: Rodney J. Guenich
DATE: MAY 2010
PLOT SCALE:
PLOT DATE: MAY 2010
SIZE: ANSI D
FILE NAME:

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

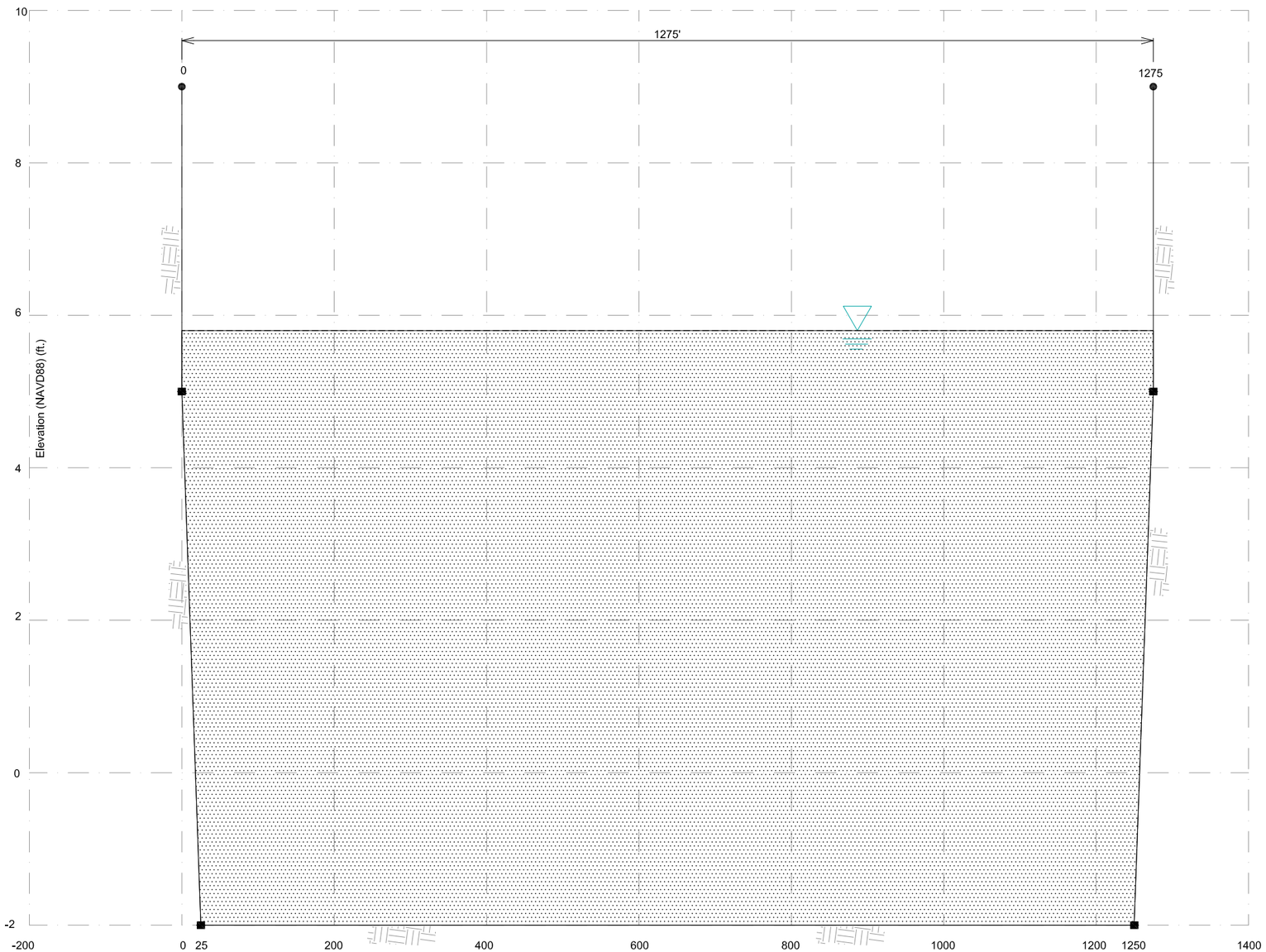
RS=134.6 END TRANSITION WIDTH IN MARSH
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION

SHEET IDENTIFICATION
7

auto_time

RS=99.80 END TRANSITION IN MARSH

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



TYPICAL SECTION H-H'

Legend
EG Max WS.
WS Max WS.
Ground.
Bank Sta.

NOTE: X-C's PLOTTED LOOKING DOWN STREAM

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY

OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: CWS | DATE: MAY 2010
DWN BY: CWS | CKD BY: CWS
SUBMITTED BY: Rodney J. Guenich
PLOT SCALE: | PLOT DATE: MAY 2010
SIZE: ANSI D | FILE NAME:

BCG Engineering & Consulting, Inc. Excellence in Engineering

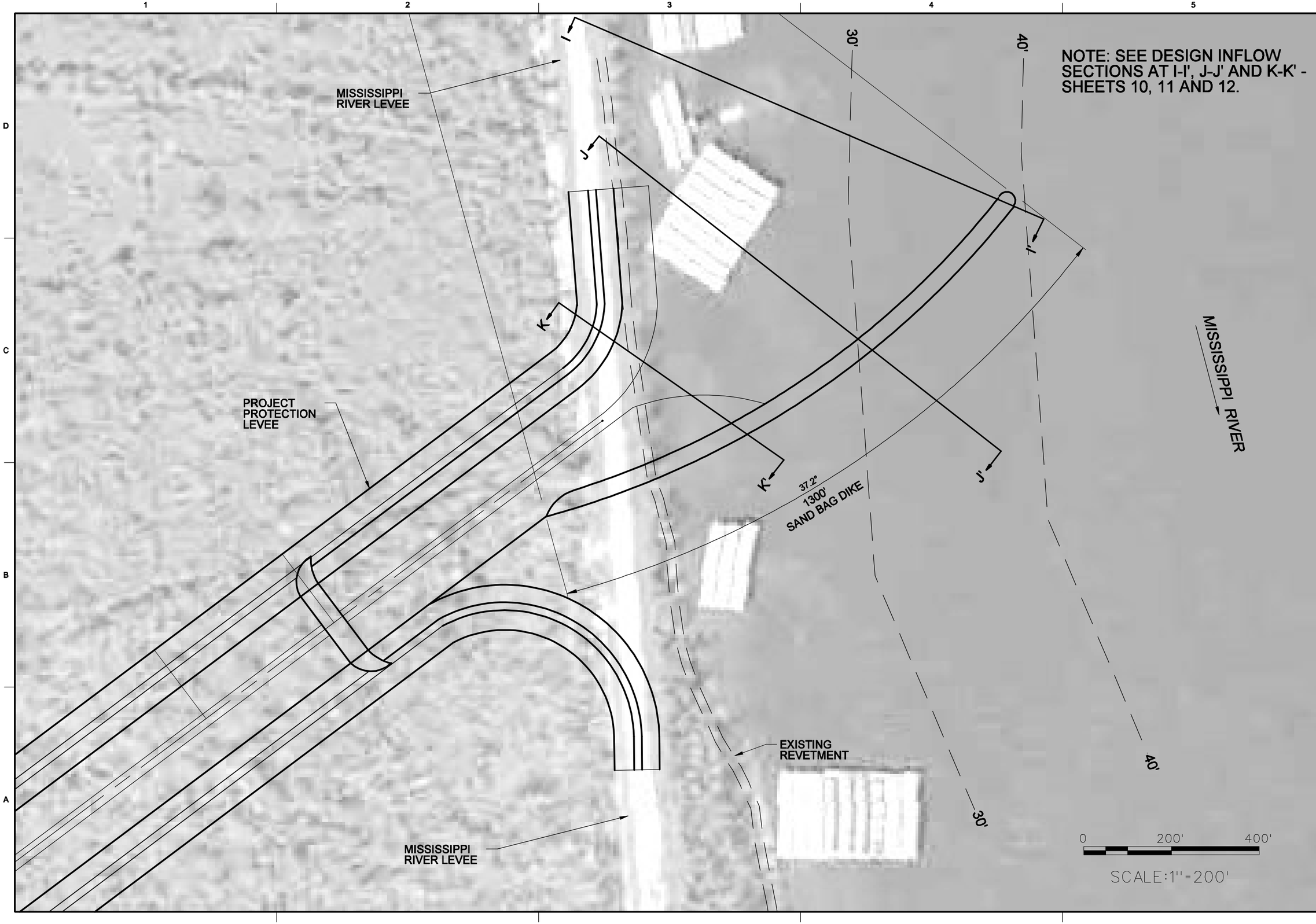
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=99.80 END TRANSITION IN MARSH

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION

SHEET IDENTIFICATION
8

auto_time



MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Gernoch
PLOT SCALE: 1" = 200'
PLOT DATE: May 2010
SIZE: ANSI D
FILE NAME: Q-480hoc.dgn

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Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

MYRTLE GROVE OPTION 3 - RIVER TIE-IN DIVERSION CHANNEL

CONCEPTUAL PLAN - VOLUME II

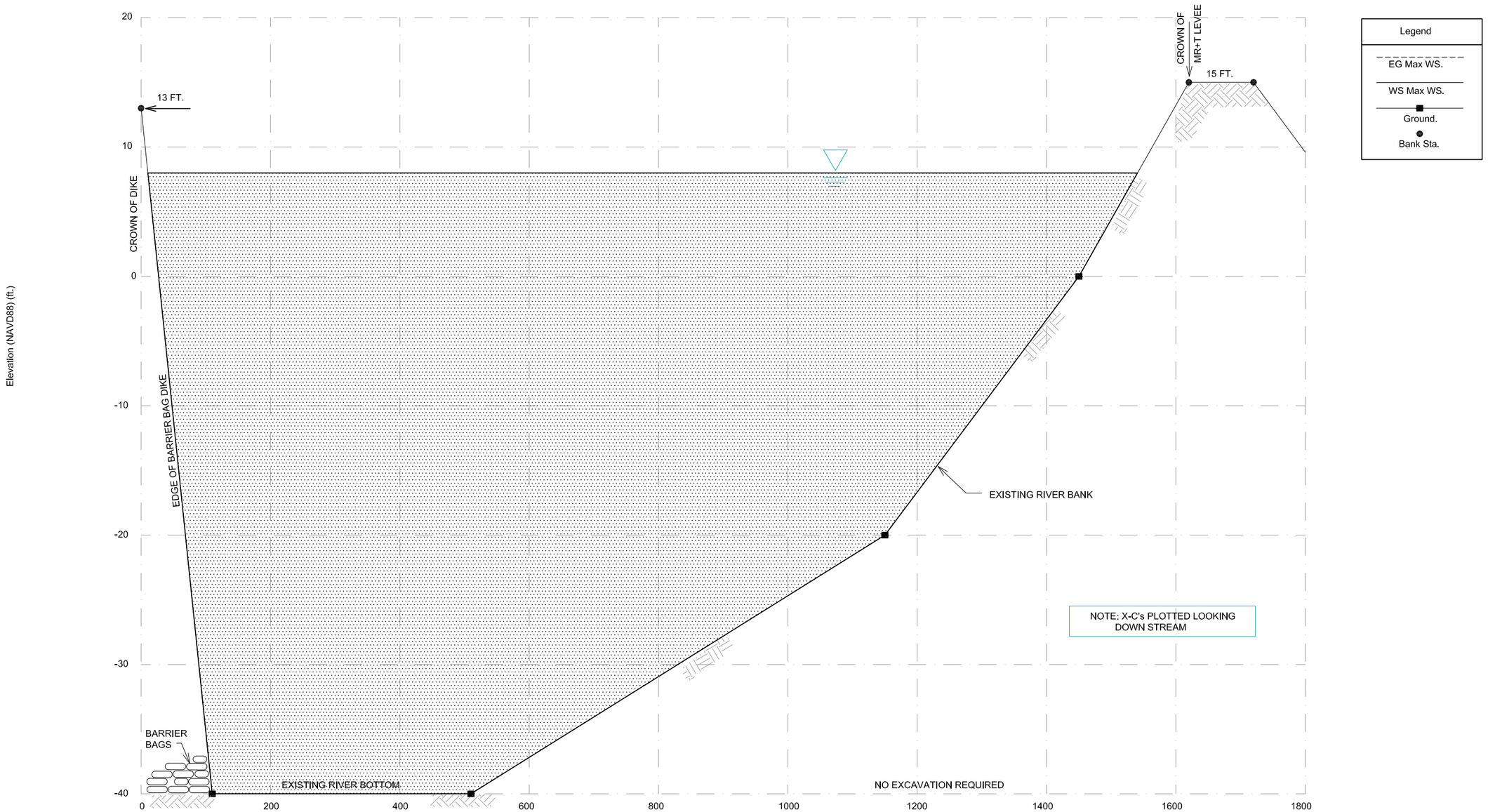
SHEET IDENTIFICATION
9

auto_time

...\FIG 9 PL OPT 3 blowup.dgn 01/11/2011 2:59:29 PM

RS=289.47 MISSISSIPPI RIVER INFLOW

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



SECTION I-I'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY

OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc. Excellence in Engineering

DESIGNED BY: CWS | DATE: MAY 2010

DWN BY: CWS | CKD BY: CWS

SUBMITTED BY: Rodney J. Guimuch

PLOT SCALE: | PLOT DATE: MAY 2010

SIZE: ANSI D | FILE NAME:

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=289.47 MISSISSIPPI RIVER INFLOW

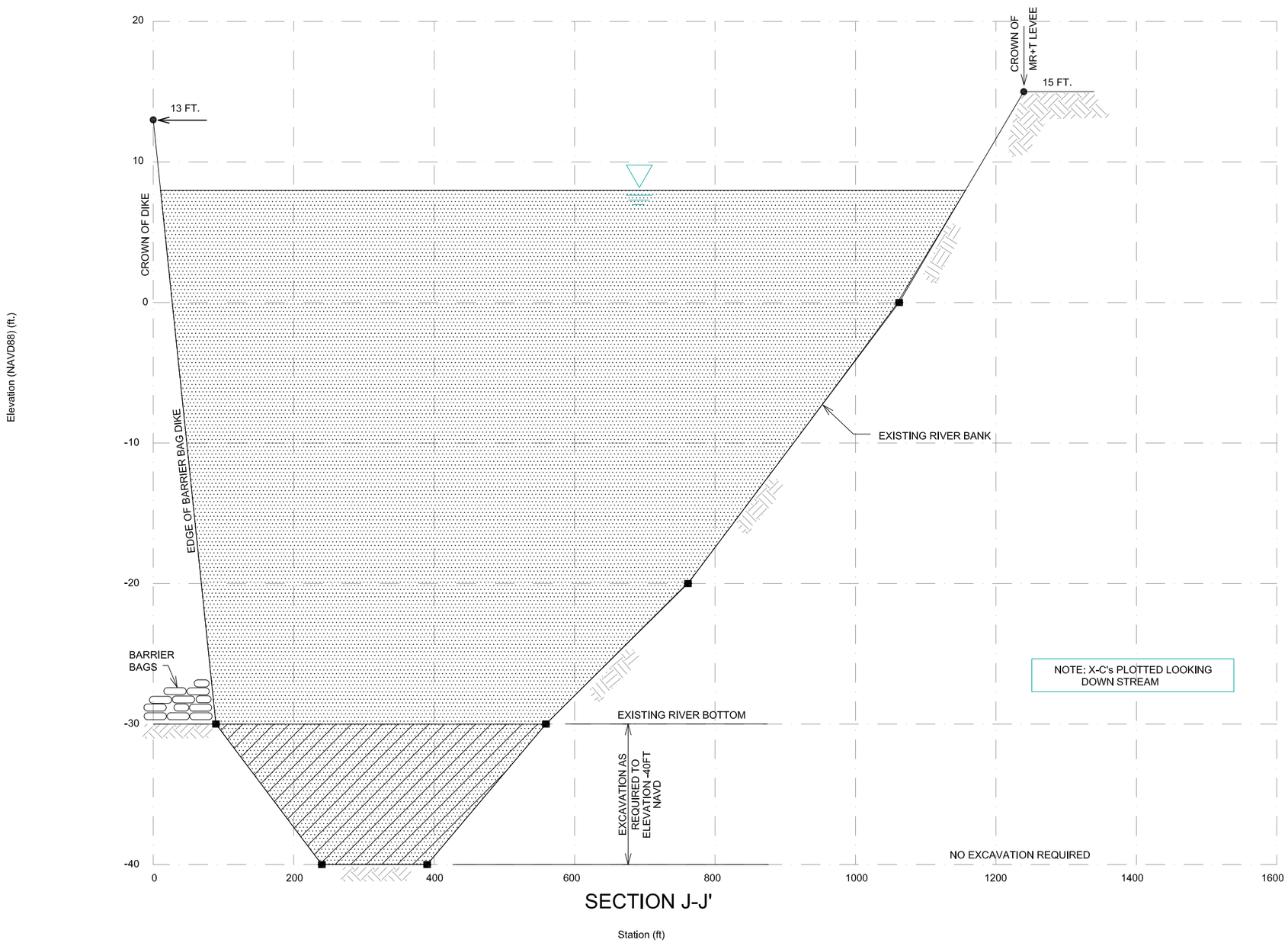
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION

SHEET IDENTIFICATION
10

auto_time

RS=284.14 M.R. INFLOW TO FLUME

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



SECTION J-J'

Legend

- EG Max WS.
- WS Max WS.
- Ground.
- Bank Sta.

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY

OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc. Excellence in Engineering

DESIGNED BY: CWS | DATE: MAY 2010

DWN BY: CWS | CKD BY: CWS

SUBMITTED BY: Rodney J. Gaunch

PLOT SCALE: | PLOT DATE: MAY 2010

SIZE: ANSI D | FILE NAME:

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=284.14 MISSISSIPPI RIVER INFLOW TO FLUME

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION

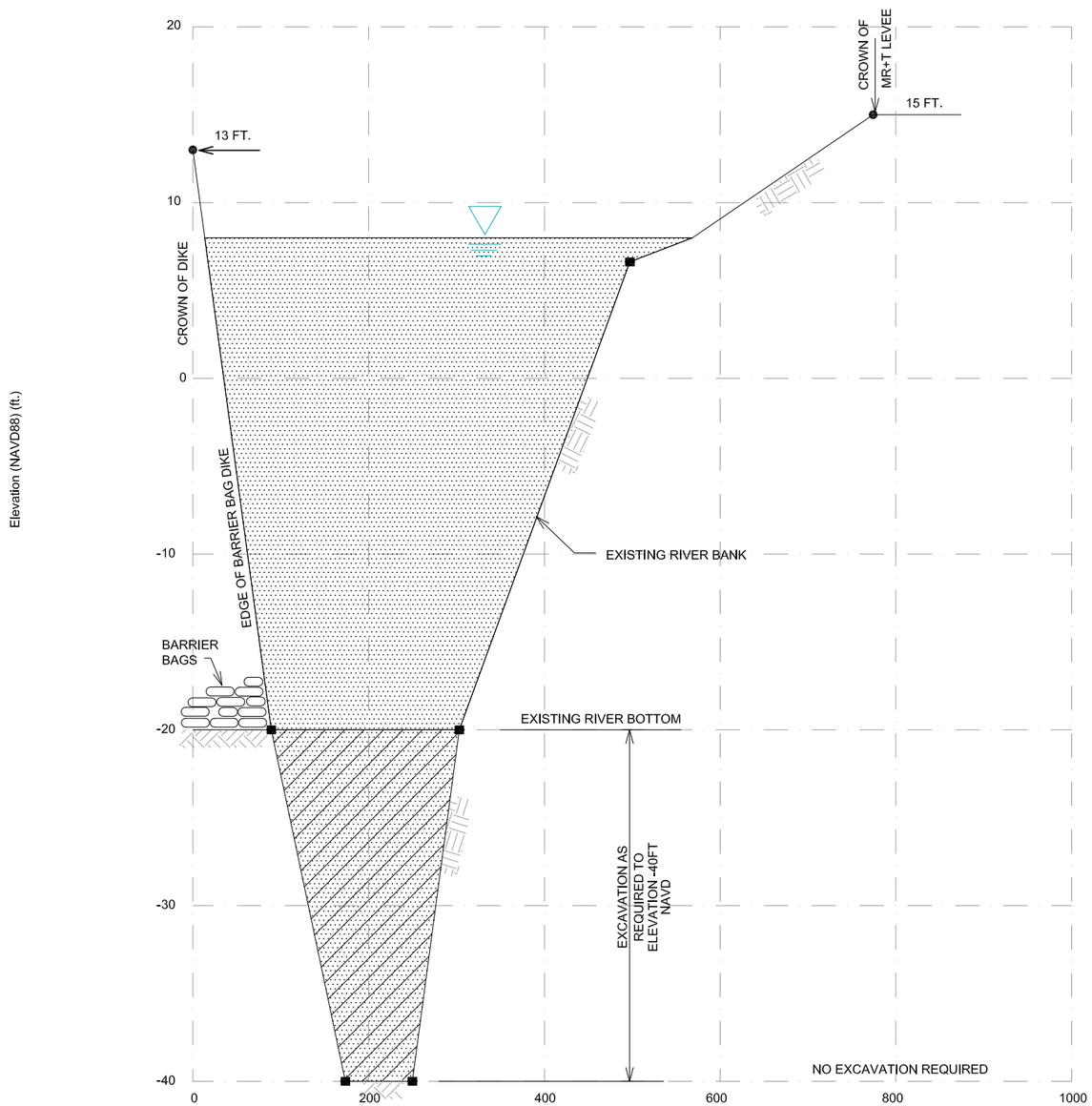
SHEET IDENTIFICATION

11

auto_time

RS=279.50

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



SECTION K-K'

Station (ft)

Legend
EG Max WS.
WS Max WS.
Ground.
Bank Sta.

NOTE: X-C's PLOTTED LOOKING DOWN STREAM

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY

OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc. Excellence in Engineering

DESIGNED BY: CWS | DATE: MAY 2010

DWN BY: CWS | CKD BY: CWS

SUBMITTED BY: Rodney J Guenich

PLOT SCALE: | PLOT DATE: MAY 2010

SIZE: ANSI D | FILE NAME:

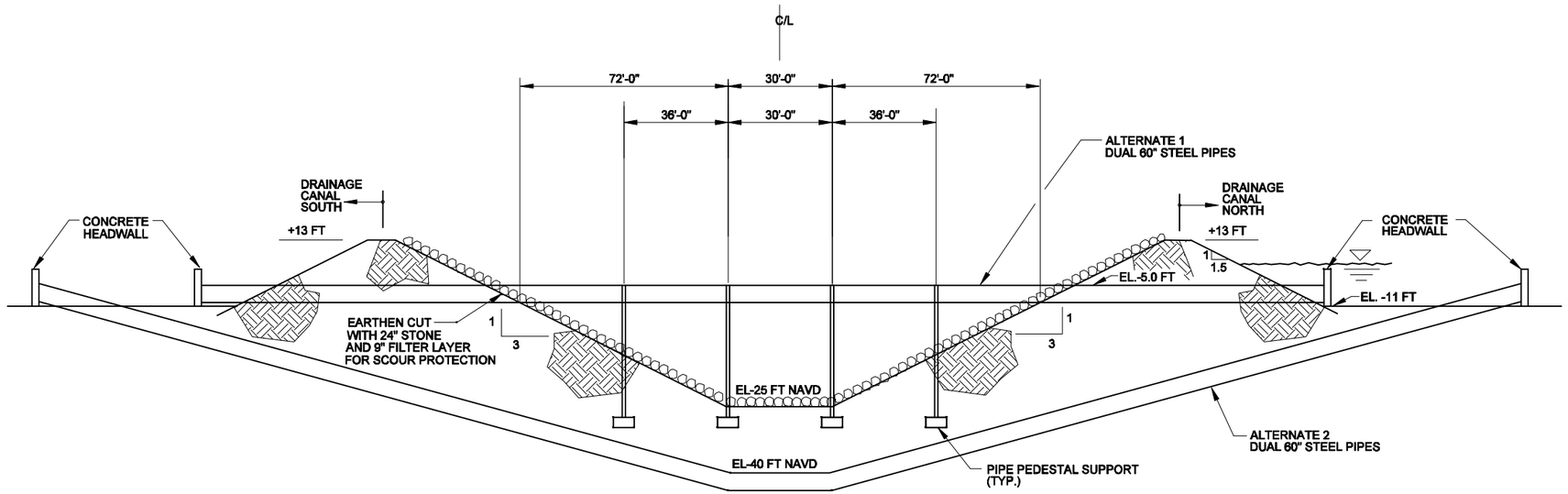
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=279.50

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION

SHEET IDENTIFICATION
12

auto_time



TYPICAL INVERTED SIPHON ALTERNATIVES
DRAINAGE CANAL CROSSINGS
MYRTLE GROVE WATER AND SEDIMENT DIVERSION PROJECT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DATE: MAY, 2010

DESIGNED BY: Cecil Soileau
DWN BY: BC3
CKD BY: CWS
SUBMITTED BY: Rodney J. Dormuth
PLOT SCALE: N.Ts.
PLOT DATE: May 2010
SIZE: ANSI D
FILE NAME: p-48Sbcc.dgn

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BOG Engineering & Consulting, Inc.
Excellence in Engineering

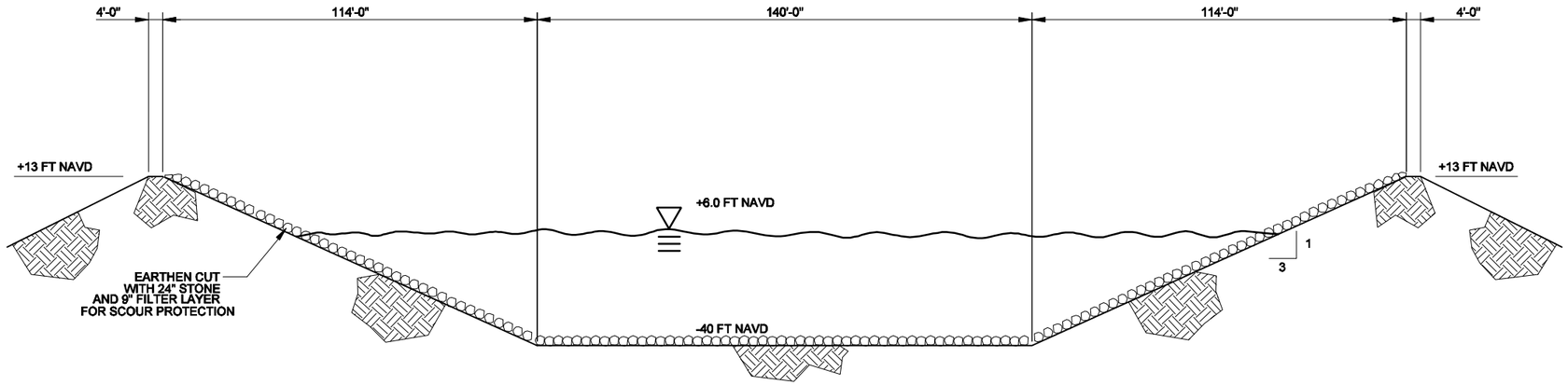
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

TYPICAL SECTIONS
VOLUME II

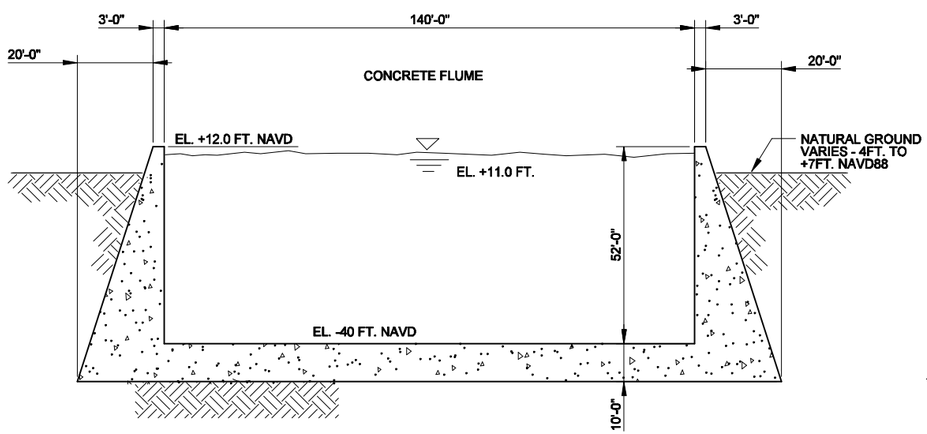
SHEET IDENTIFICATION
13

auto_time

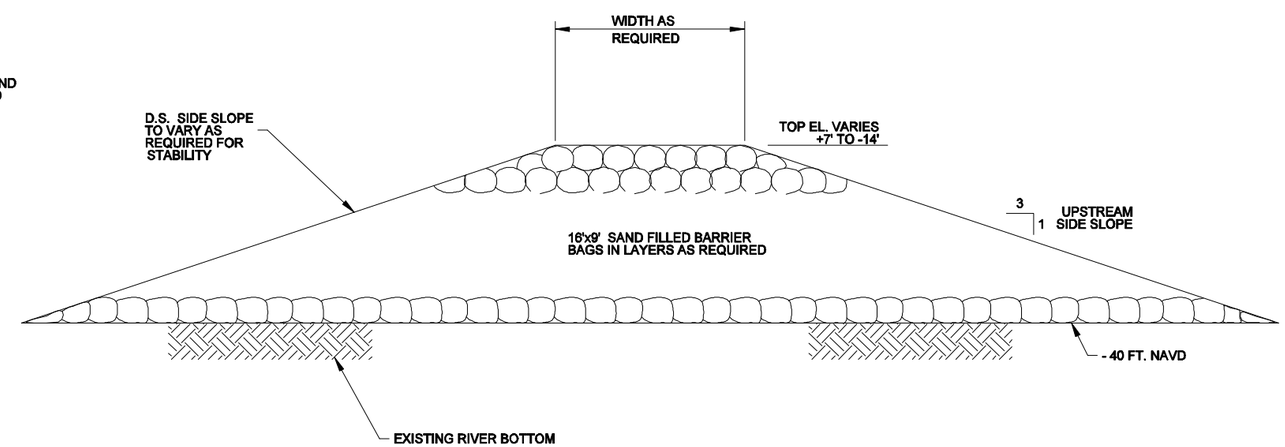
...\FIG 13_TYP. INVERT SIPHON.dgn 01/11/2011 2:59:56 PM



TYPICAL SECTION B-B'



TYPICAL SECTION A-A'



TYPICAL SECTION D-D'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Cernoch
PLOT SCALE: 1:1 "as-is"
PLOT DATE: Wednesday
SIZE: ANSI D
FILE NAME: p-s50xxx.dgn

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

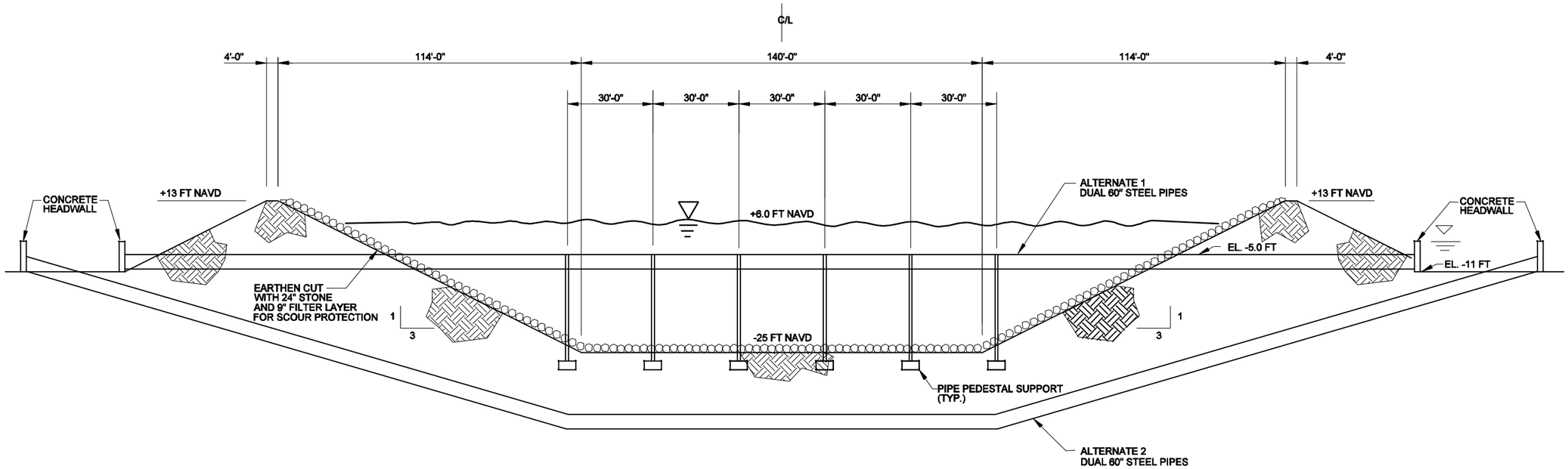
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

TYPICAL SECTIONS
VOLUME II

SHEET IDENTIFICATION
14

auto_time

...\FIG 14_FLUME SECTIONS.dgn 01/11/2011 3:00:11 PM



TYPICAL INVERTED SIPHON ALTERNATIVES
DRAINAGE CANAL CROSSINGS
MYRTLE GROVE WATER AND SEDIMENT DIVERSION PROJECT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Dernoch
PLOT SCALE: 1:1 "=1'-0"
PLOT DATE: Nov-2010
SIZE: ANSI D
FILE NAME: p-585box.dgn

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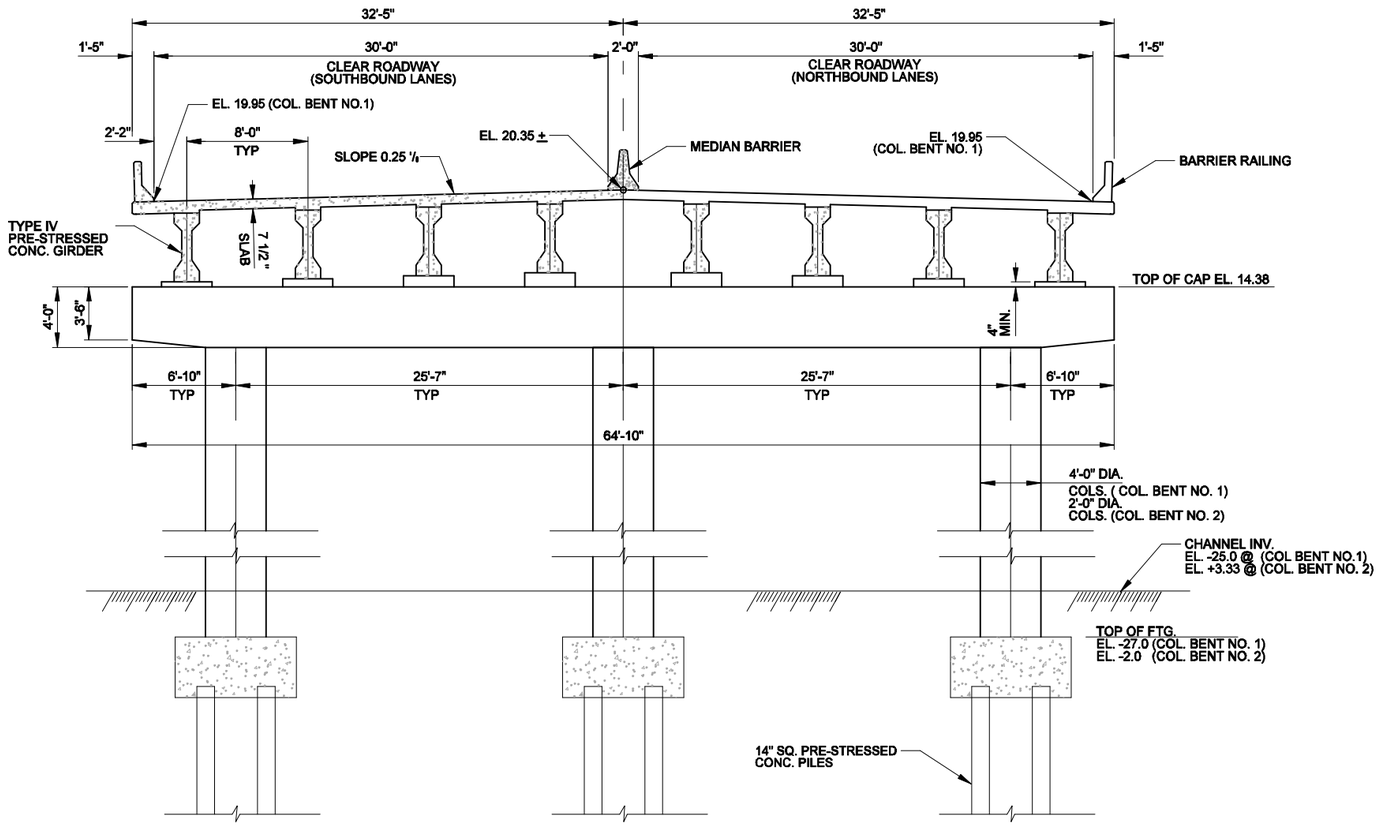
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

TYPICAL SECTIONS
VOLUME II

SHEET IDENTIFICATION
15

auto_time

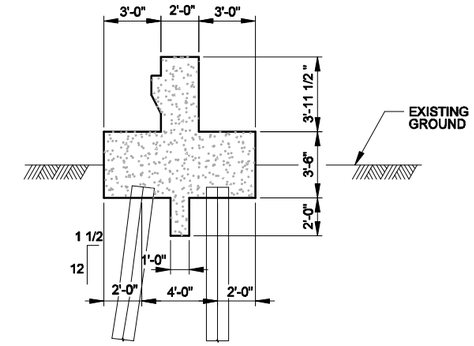


TYPICAL COLUMN BENT

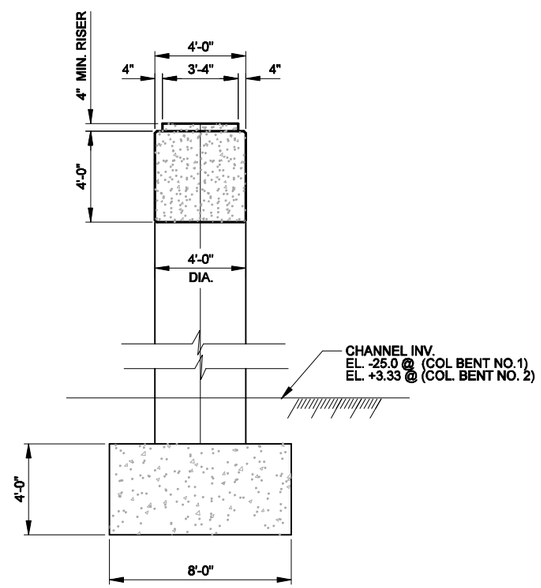
COL. BENT NO. 1, ONE(1) REQUIRED
COL. BENT NO. 2, TWO (2) REQUIRED

SCALE: 1" = 5'-0"

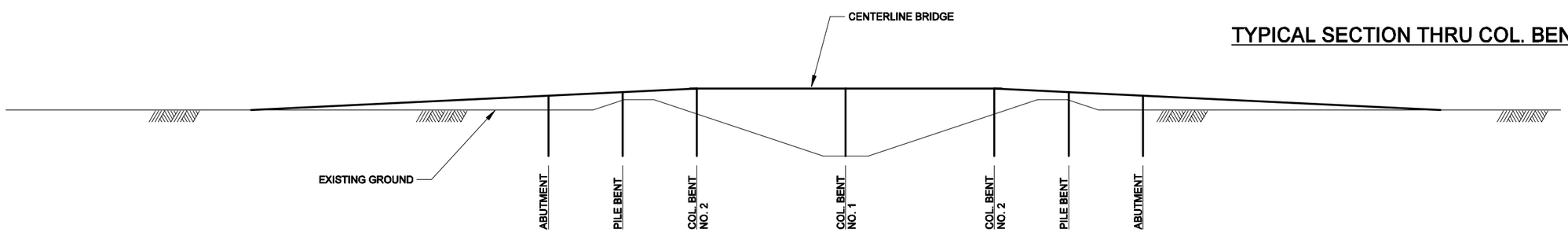
5' 0 5' 10'



TYPICAL ABUTMENT



TYPICAL SECTION THRU COL. BENT NO. 1



TYPICAL PROFILE ALONG C/L BRIDGE

SCALE: 1" = 40'-0"

40' 20' 0 40' 80'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Genovich
PLOT SCALE: 1" = 10'-0"
PLOT DATE: November 2010
SIZE: ANSI B
FILE NAME: Q-480hzxc.dgn
DATE: MAY, 2010

BCG Engineering & Consulting, Inc. Excellence in Engineering

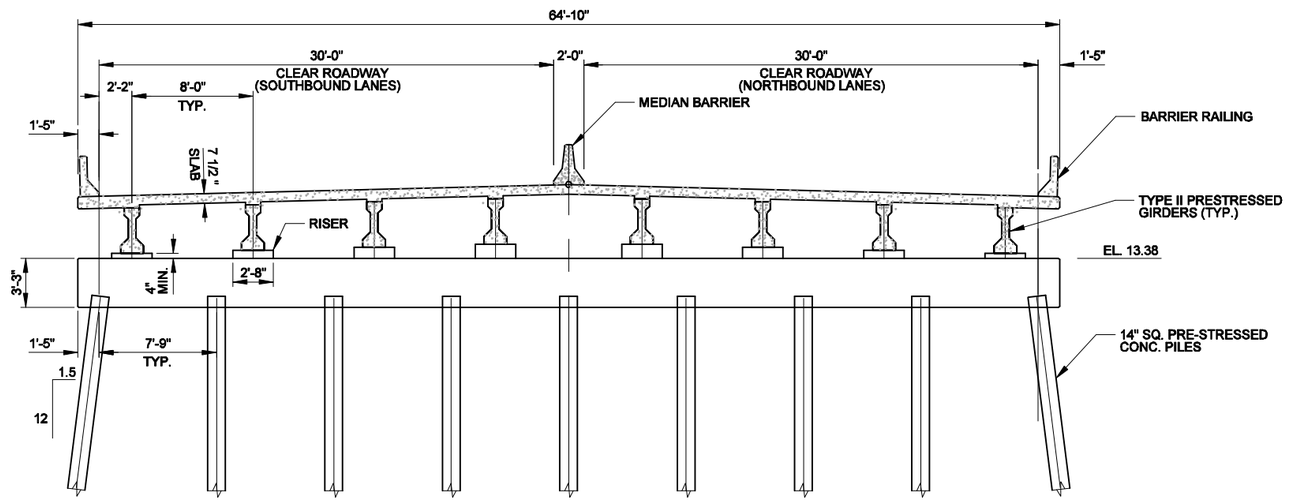
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

CROSS SECTION AND PROFILE
LA. HWY 23 BRIDGE

SHEET IDENTIFICATION
16

auto_time

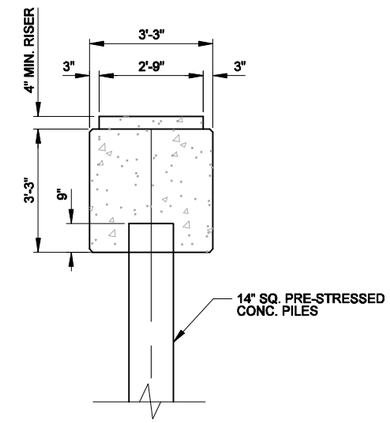
...\FIG 16 bridge section.DGN 01/11/2011 3:00:44 PM



TYPICAL PILE BENT

SCALE: 1" = 5'-0"

5' 0 5' 10'



TYPICAL SECTION THRU PILE BENT

SCALE: ½" = 1'- 0"

12" 0 2' 4' 6' 8'

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Cormier
PLOT SCALE: 1" = 1'-0"
PLOT DATE: MAY, 2010
SIZE: ANSI B
FILE NAME: G-BSDxxx.dgn

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

CROSS SECTION - PILE BENT
LA. HWY. 23 BRIDGE

SHEET IDENTIFICATION
17

auto_time

...\VOLUME 2\FIG 17 PILE BENT.DGN 01/11/2011 3:01:01 PM

APPENDIX C

TEXT AND COST ESTIMATE
Drawings 1C to 15C

APPENDIX C

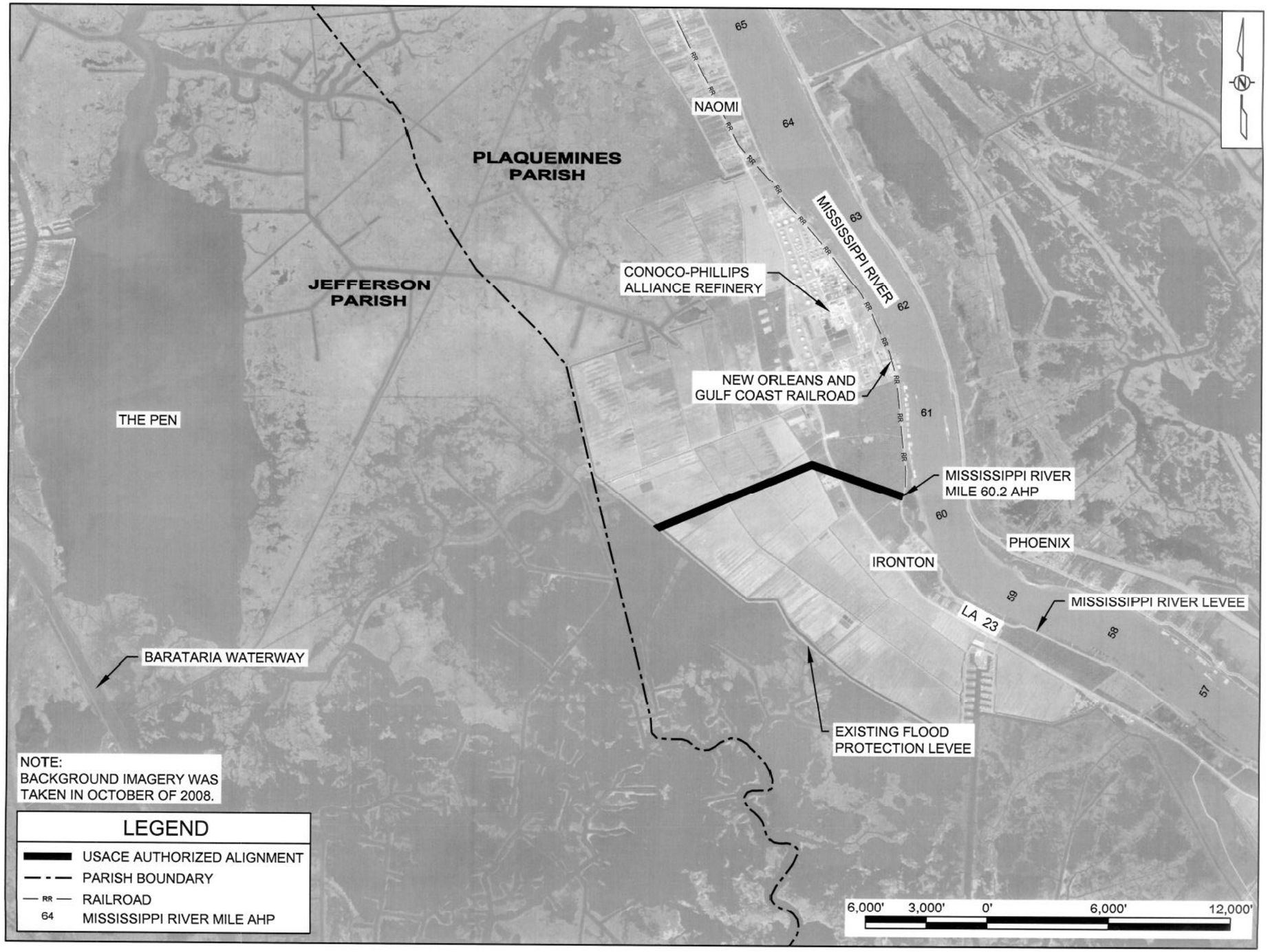
1. INTRODUCTION. It is the intent of the OCPRA that the variability of possible sizes of water and sediment diversions be analyzed within a reasonable set of limits between 15,000 cfs and 45,000 cfs, to bracket the likely benefits as well as the effects to the receiving side of the diversion. To achieve this OCPRA has requested the BCG produce drawings of the proposed structure and diversion channel dimension, a one dimensional unsteady flow model of the diversion project, which extends from the Mississippi River to some three (3) miles into the receiving marsh, and a construction cost estimate for that project. Finally BCG has been requested to revise a draft report dated April 2010 to incorporate this additional alternative. It is to be noted that the Draft Report of April 2010 already included an alternative 50 to 60,000 cfs diversion structure with model results and drawing of that plan, but no cost estimate. Also all of the hydrologic data included in the draft report is also applicable to this conceptual design; therefore none of that data is presented in this Appendix C. The model hydraulic output for the 45,000 CFS alternative is presented in Appendix D.

2. CONCLUSIONS. See the Drawings 1C for a Vicinity Map of the project area and the original alignment as proposed by the U.S. Army Corps of Engineers for a freshwater only conceptual design. Drawings 2C through 13C show the dimensions of the gated structure, dikes, flume, and diversion channel sized to pass 45,000 cfs of water and sediment over the useful life of the project, which is expected to be 100 years. Drawings 14C and 15C show the required dimensions of a highway bridge on LA 23 that would be required at the crossing of the Diversion Channel. Finally Table C-1 is a construction cost estimate for this alternative, which is $330 million.

TABLE 1-C

BUDGETARY COST ESTIMATE
MYRTLE GROVE DIVERSION CHANNEL

ITEM NO.	DESCRIPTION	UNIT	QUANITY	UNIT PRICE	AMOUNT
1.	Mobilization	Lump			$3,000,000.00
2.	Excavation	CY	2,118,091.00	$10.00	$21,180,910.00
3.	Dredging	CY	4,216,685.00	$25.00	$105,417,125.00
4.	Levees	CY	1,255,864.55	$14.00	$17,582,103.74
5.	Rip-rap	Tons	463,522.00	$70.00	$32,446,540.00
6.	Crushed Stones	CY	122,963.00	$55.00	$6,762,965.00
7.	Geo-textile Fabric	SY	448,589.00	$10.00	$4,485,890.00
8.	Dike (Sand-filled Geo-tubes)	CY	365,251.82	$20.00	$7,305,036.47
8a.	Spread Excess Excavation on Site	CY	862,226.45	2.00	$1,724,452.89
	Total Channel Construction				**$199,905,023.10**
9.	Reinforced Concrete				
	A. Flume:				
	1. Base Slab	CY	38,266.00	$500.00	$19,133,000.00
	2. Walls	CY	10,022.00	$700.00	$7,015,400.00
	B. T-walls:				
	1. Base Slab	CY	700.00	$500.00	$350,000.00
	2. Walls	CY	570.00	$700.00	$399,000.00
	C. Gate Structure:				
	1. Base Slab	CY	1,750	$500.00	$875,000.00
	2. Piers	CY	744.00	$700.00	$520,800.00
	D. Pile Caps for 60 inch Pipe	CY	21.00	$700.00	$14,700.00
	Total Reinforced Concrete				**$28,307,900.00**
10.	Structural Steel Gates	Tons	81.00	$30,000.00	$2,430,000.00
	Total Gates				**$2,430,000.00**
11.	Piling				
	HP 14 x 73	L.F.	355,800.00	$75.00	$26,685,000.00
	12 inch dia. Steel Pipe Piles	L.F.	800.00	$75.00	$60,000.00
12.	PZ-22 Cut-off	SF	8,130.00	$45.00	$365,850.00
	Total Piling				**$27,110,850.00**
13.	60 inch dia. x 3/8 inch w.t.				
	AWWA C200 Pipe	L.F.	620.00	$1,200.00	$744,000.00
	Total Steel Pipe				**$744,000.00**
14.	20 inch Dia. Sched 80 Shell Oil Overhead Pipe Crossing and Pipe Supports	L.F.	450.00	$450.00	$202,500.00
	Total 20 inch Diameter Pipe				**$202,500.00**
15.	40 T Gantry Crane	Each	1.00	$500,000.00	$500,000.00
	Total Gantry Crane				**$500,000.00**
16.	LA 23 Bridge	L.F.	1,060.00	$14,625.00	$15,502,500.00
	Total LA 23 Bridge				**$15,502,500.00**
17.	Demobilization	Lump		$1,000,000.00	$1,000,000.00
	Total Demobilization				**$1,000,000.00**
	Total W/O Contingencies				**$275,702,773.10**
18.	Contingencies (20%)	Lump			$55,140,554.62
	TOTAL PROJECT COST				**$330,843,327.73**
NOTES:					
1. All prices are in current dollars and do not include future inflation.					
2. Quantities shown are based on preliminary design only and are not all inclusive.					
3. Land acquisition costs not included.					



NOTE:
BACKGROUND IMAGERY WAS TAKEN IN OCTOBER OF 2008.

LEGEND

- USACE AUTHORIZED ALIGNMENT
- PARISH BOUNDARY
- RR — RAILROAD
- 64 — MISSISSIPPI RIVER MILE AHP

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DATE: AUGUST, 2010
DWN BY: BCS
CKD BY: CWS
SUBMITTED BY: Rodney J. Gernoch
PLOT SCALE: AS NOTED
PLOT DATE: August 2010
SIZE: ANSI D
FILE NAME: VICINITY.dgn

BOG Engineering & Consulting, Inc.
Excellence in Engineering

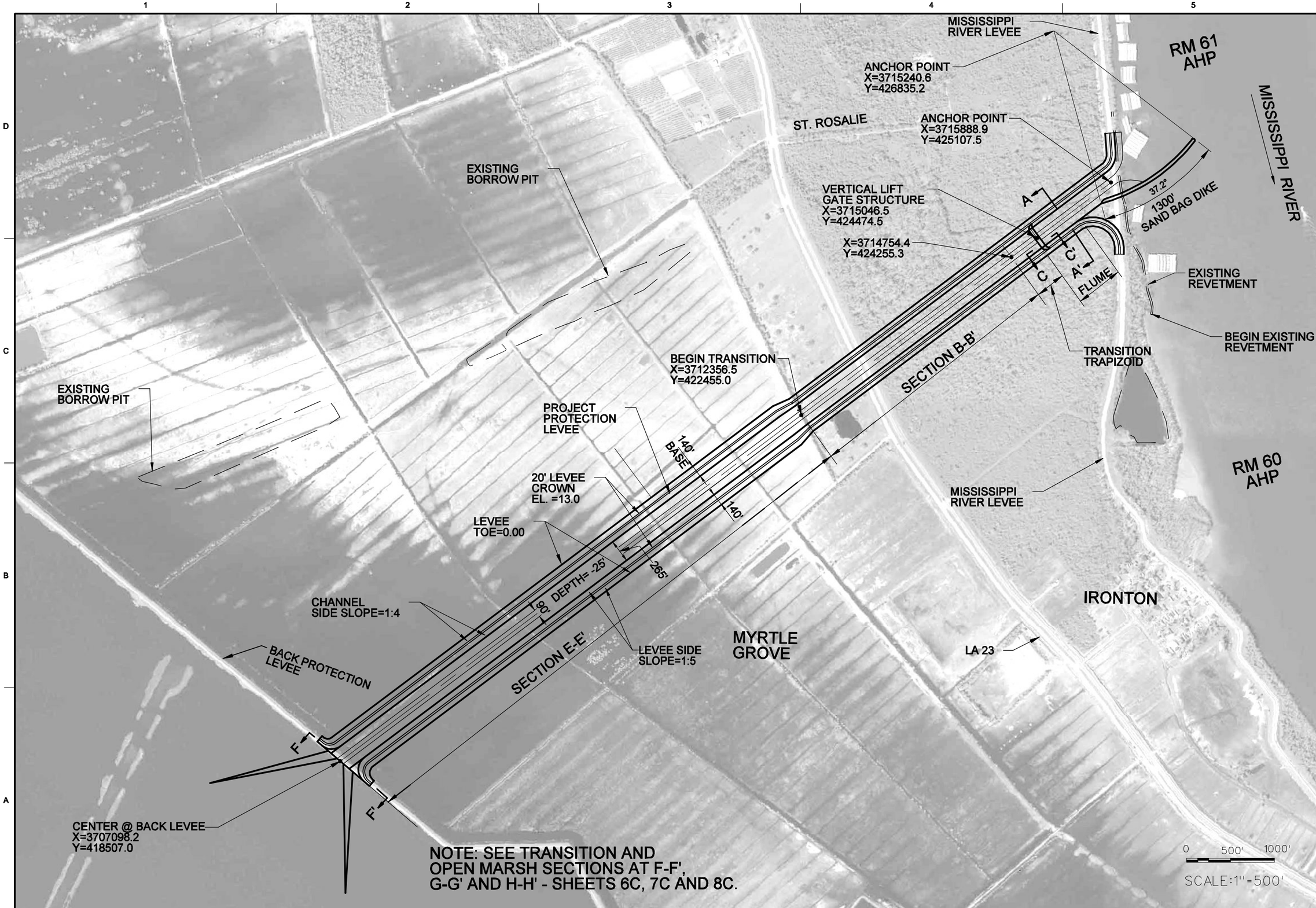
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

VICINITY MAP &
CORPS OF ENGINEERS ALIGNMENT

SHEET IDENTIFICATION
1C

auto_time

...\VICINITY.dgn 01/11/2011 3:02:12 PM



MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Gemsch
DATE: AUGUST, 2010
PLOT SCALE: 1" = 500'
PLOT DATE: August 2010
SIZE: ANSI D
FILE NAME: FIG 2C.DGN

BCG Engineering & Consulting, Inc. Excellence in Engineering

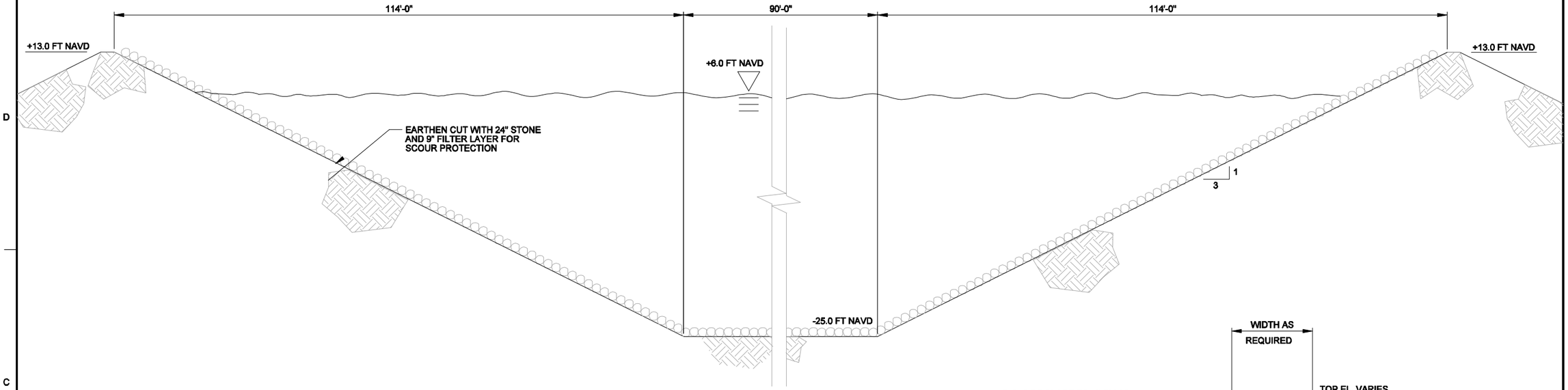
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

MYRTLE GROVE OPTION 4 DIVERSION CHANNEL

CONCEPTUAL PLAN - VOLUME III

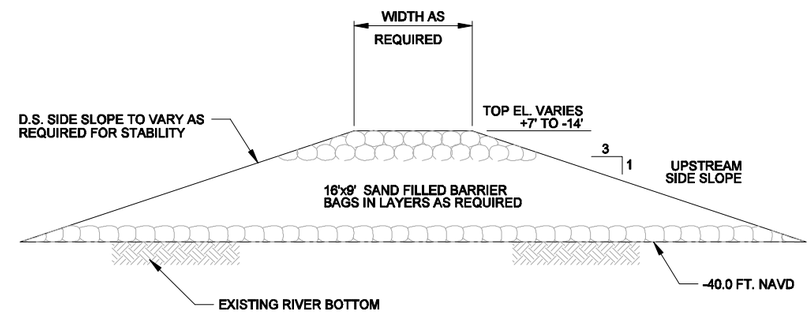
SHEET IDENTIFICATION
2C

...\FIG 2c PL OPT 3.dgn 01/11/2011 3:02:50 PM



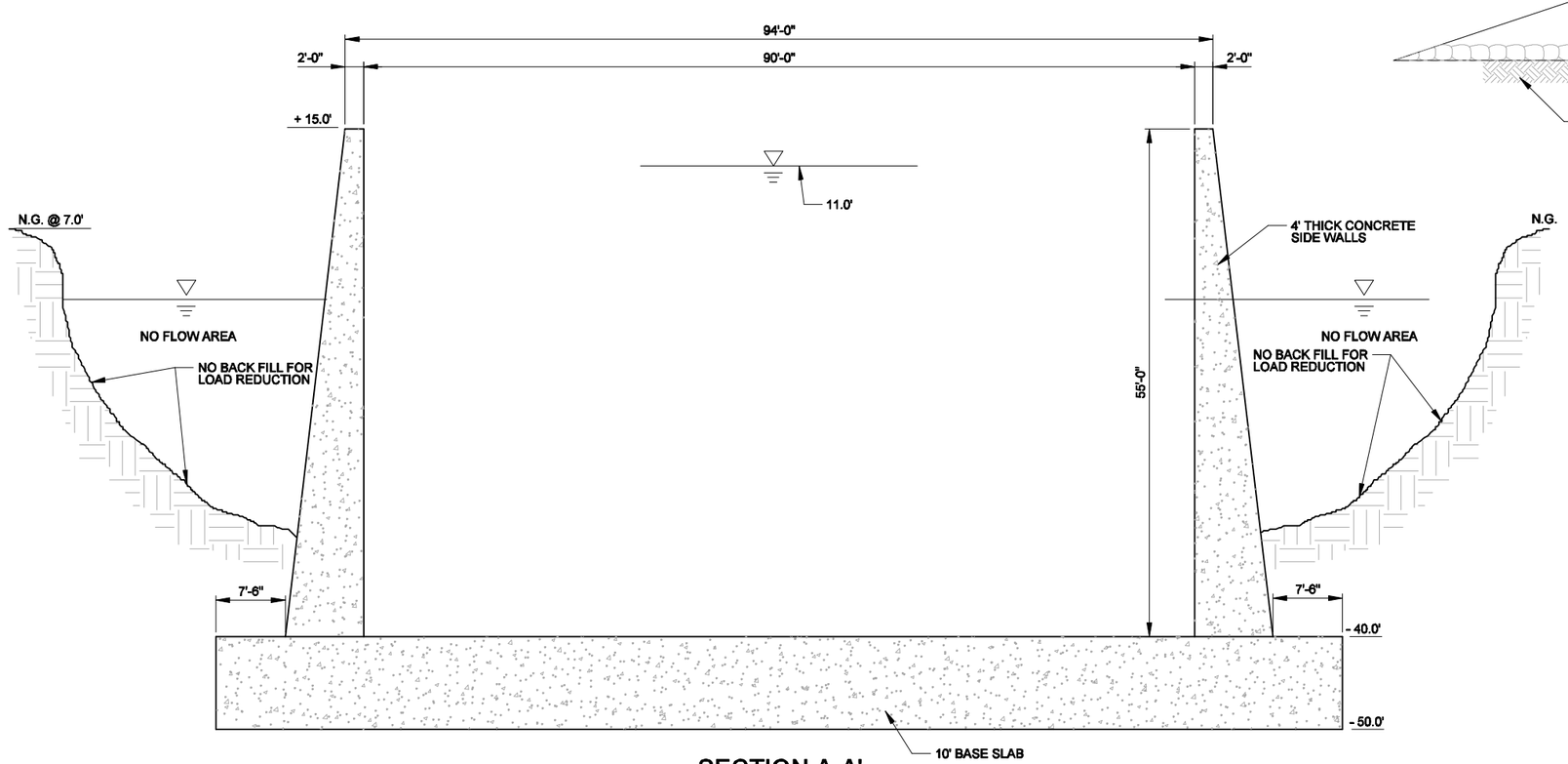
TYPICAL SECTION B-B'

N.T.S.



TYPICAL SECTION D-D'

N.T.S.



SECTION A-A'

30' X 49' DIVERSION CANAL BETWEEN MISSISSIPPI RIVER AND CONTROL STRUCTURE

N.T.S.

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: Cecil Soileau
DATE: AUGUST, 2010
DWN BY: BC3
CKD BY: CWS
SUBMITTED BY: Rodney J. Cormier
PLOT SCALE: NTS
PLOT DATE: August, 2010
SIZE: ANSI D
FILE NAME: FIG 3_40X45 DIVERSION CANAL

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

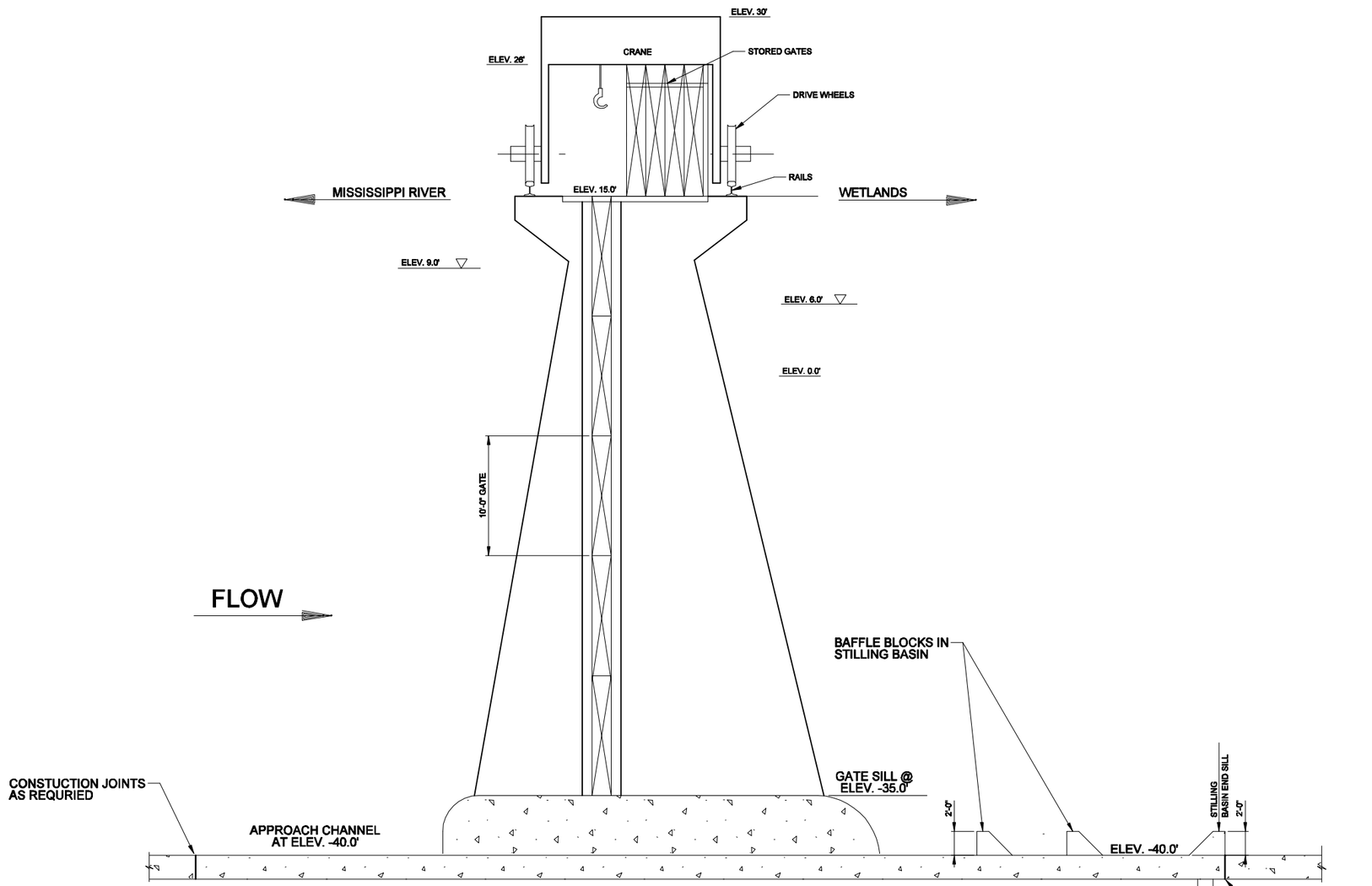
BOG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

TYPICAL SECTION
VOLUME III

SHEET IDENTIFICATION
3C

auto_time



SECTION C-C'
PRELIMINARY ELEVATION VIEW

MYRTLE GROVE SEDIMENT DIVERSION STRUCTURE

N.T.S.

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BOC Engineering & Consulting, Inc.
Excellence in Engineering

DESIGNED BY: Cecil Soileau
DATE: AUGUST, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Cannoch
PLOT SCALE: As Noted
PLOT DATE: August 2010
SIZE: ANSI D
FILE NAME: FIG 4.DGN

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

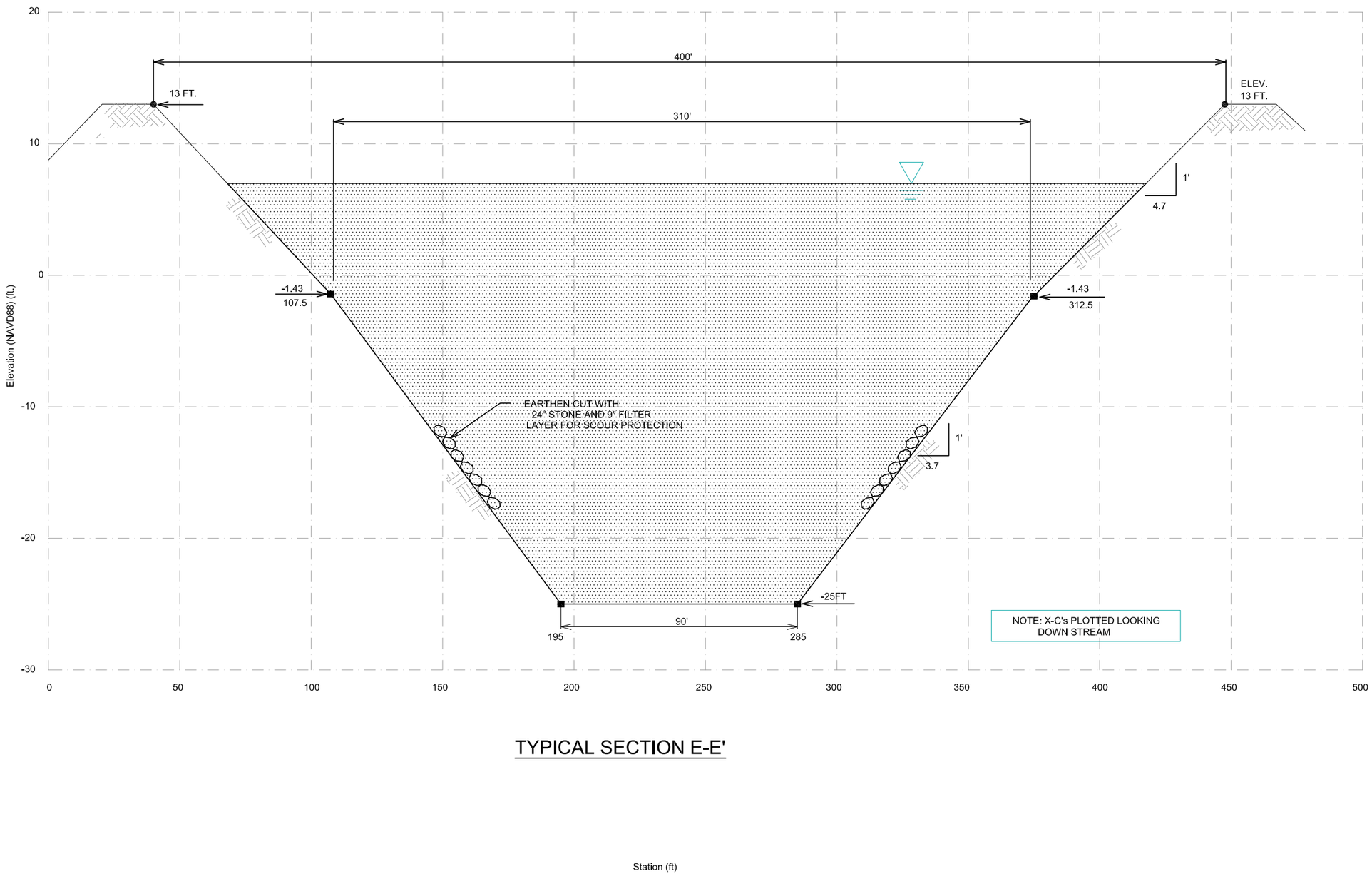
TYPICAL SECTION
CONCEPTUAL PLAN - VOLUME III

SHEET IDENTIFICATION
4C

...\FIG 4C.dgn 01/11/2011 3:03:36 PM

RS=228.338

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



TYPICAL SECTION E-E'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: CWS
DWN BY: ESCOBAR
CKD BY: CWS
SUBMITTED BY: Rodney J. Gaenisch
DATE: AUGUST 2010
PLOT SCALE:
PLOT DATE: August, 2010
SIZE: ANSI D
FILE NAME:

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

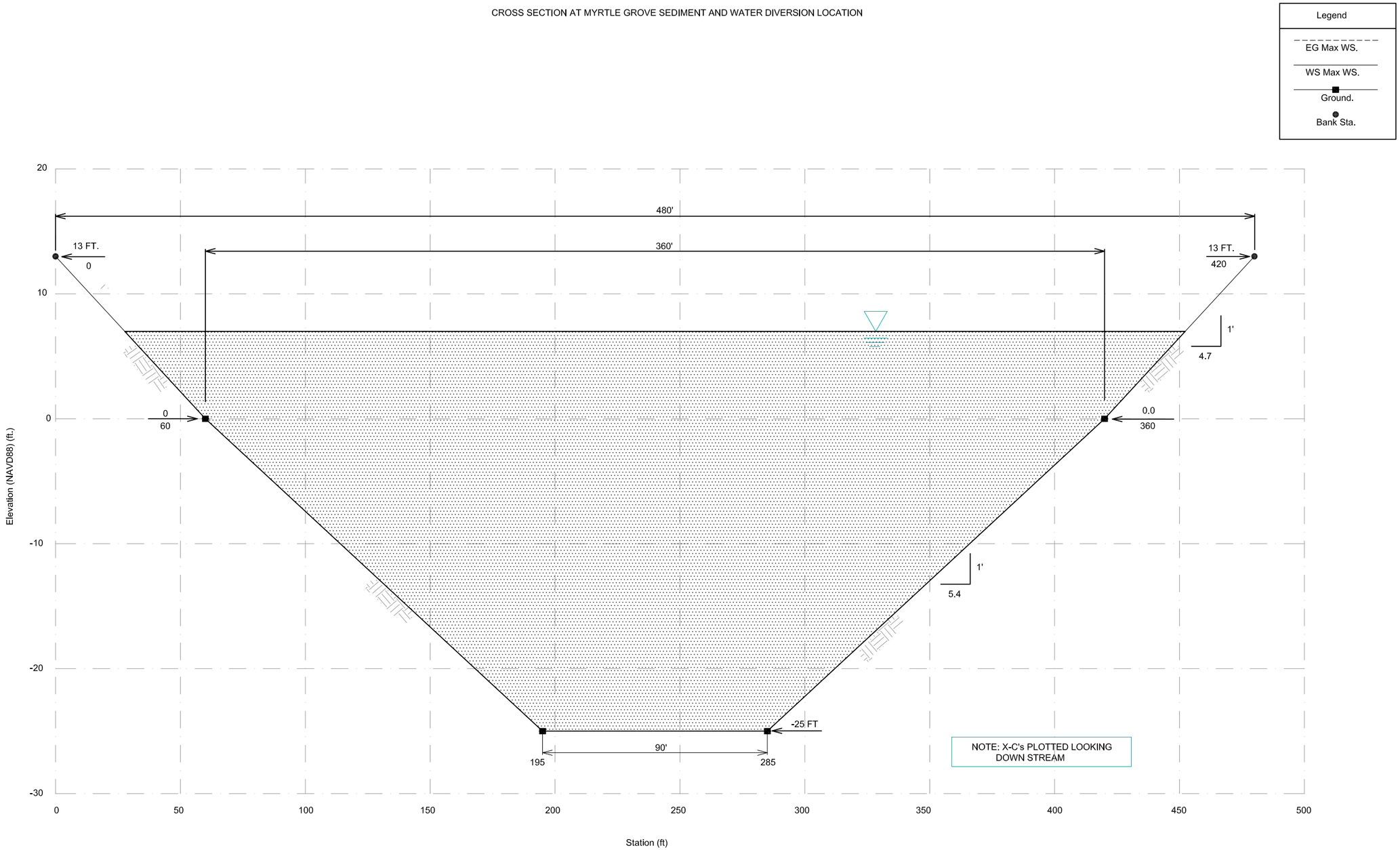
RS=228.338
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION
CONCEPTUAL PLAN - VOLUME III

SHEET IDENTIFICATION
5C

auto_time

RS=149.80 BEGIN TRANSITION TO OPEN MARSH

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



TYPICAL SECTION F-F'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: CWS
DATE: AUGUST 2010
DWN BY: CWS
CKD BY: CWS
SUBMITTED BY: Rodney J. Guenich
PLOT SCALE: 1
PLOT DATE: August, 2010
SIZE: ANSI D
FILE NAME:

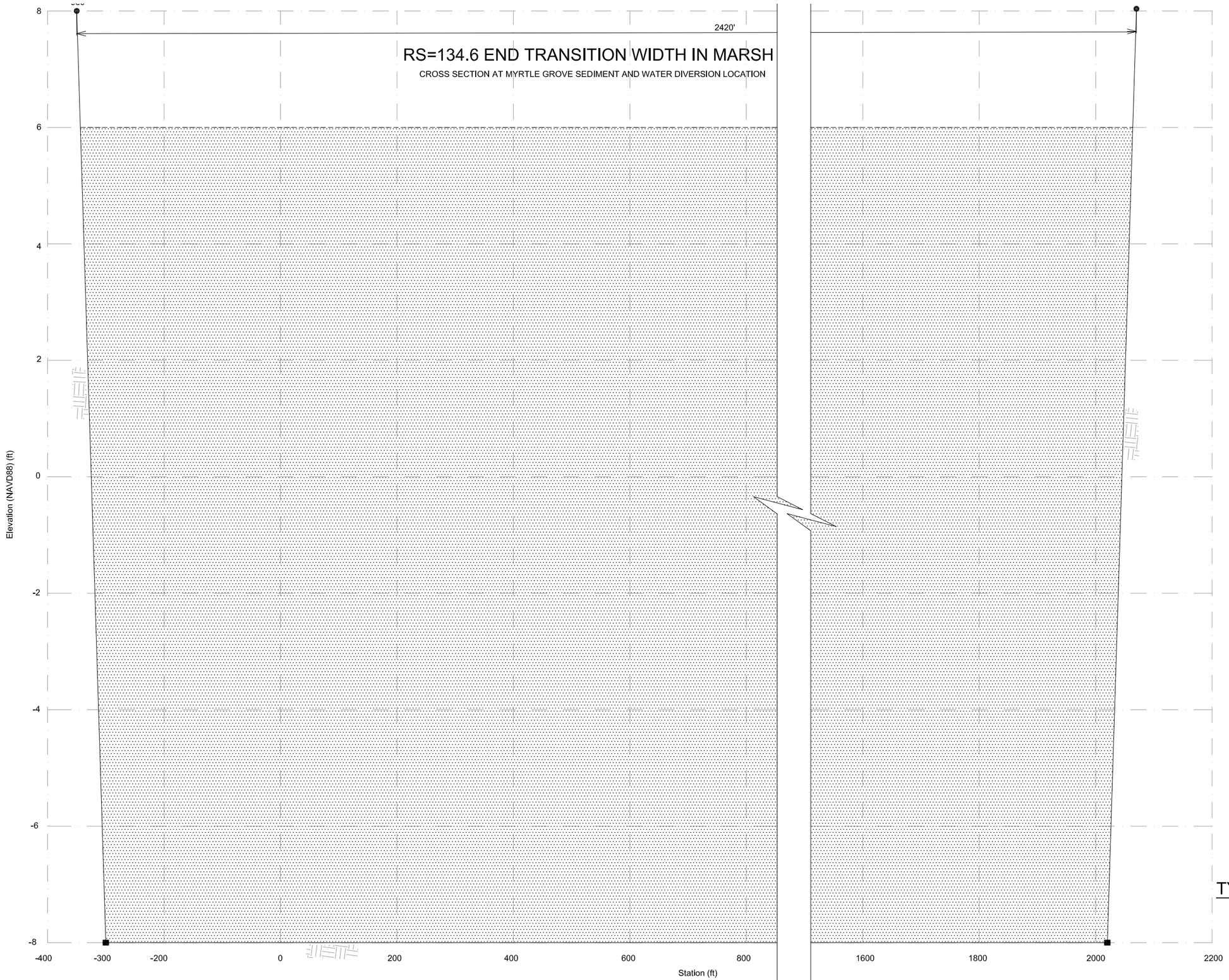
BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=149.80 BEGIN TRANSITION TO OPEN MARSH
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION
VOLUME III

SHEET IDENTIFICATION
6C

auto_time



Legend

EG Max WS.

WS Max WS.

Ground.

Bank Sta.

NOTE: X-C's PLOTTED LOOKING DOWN STREAM

TYPICAL SECTION G-G'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY

OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

DESIGNED BY: CWS
DATE: AUGUST 2010
DWN BY: CWS
CKD BY: CWS
SUBMITTED BY: Rodney J. Guerisch
PLOT SCALE:
PLOT DATE: August, 2010
SIZE: ANSI D
FILE NAME:

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=134.6 END TRANSITION WIDTH IN MARSH

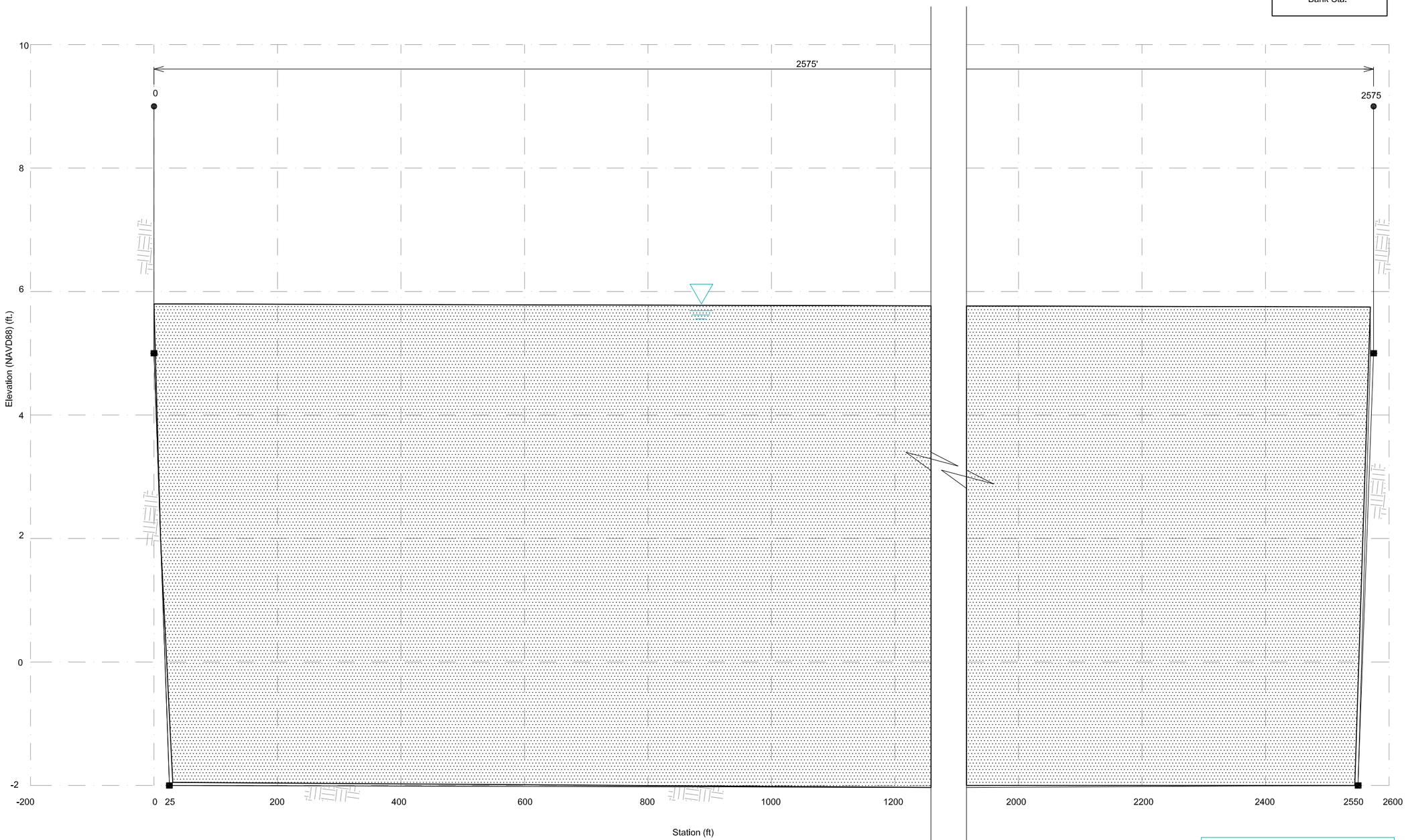
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION

SHEET IDENTIFICATION
7C

auto_time

RS=99.80 END TRANSITION IN MARSH

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



TYPICAL SECTION H-H'

NOTE: X-C's PLOTTED LOOKING DOWN STREAM

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

DESIGNED BY: CWS
DATE: AUGUST 2010
DWN BY: CWS
CKD BY: CWS
SUBMITTED BY: Rodney J Guenich
PLOT SCALE:
PLOT DATE: August, 2010
SIZE: ANSI D
FILE NAME:

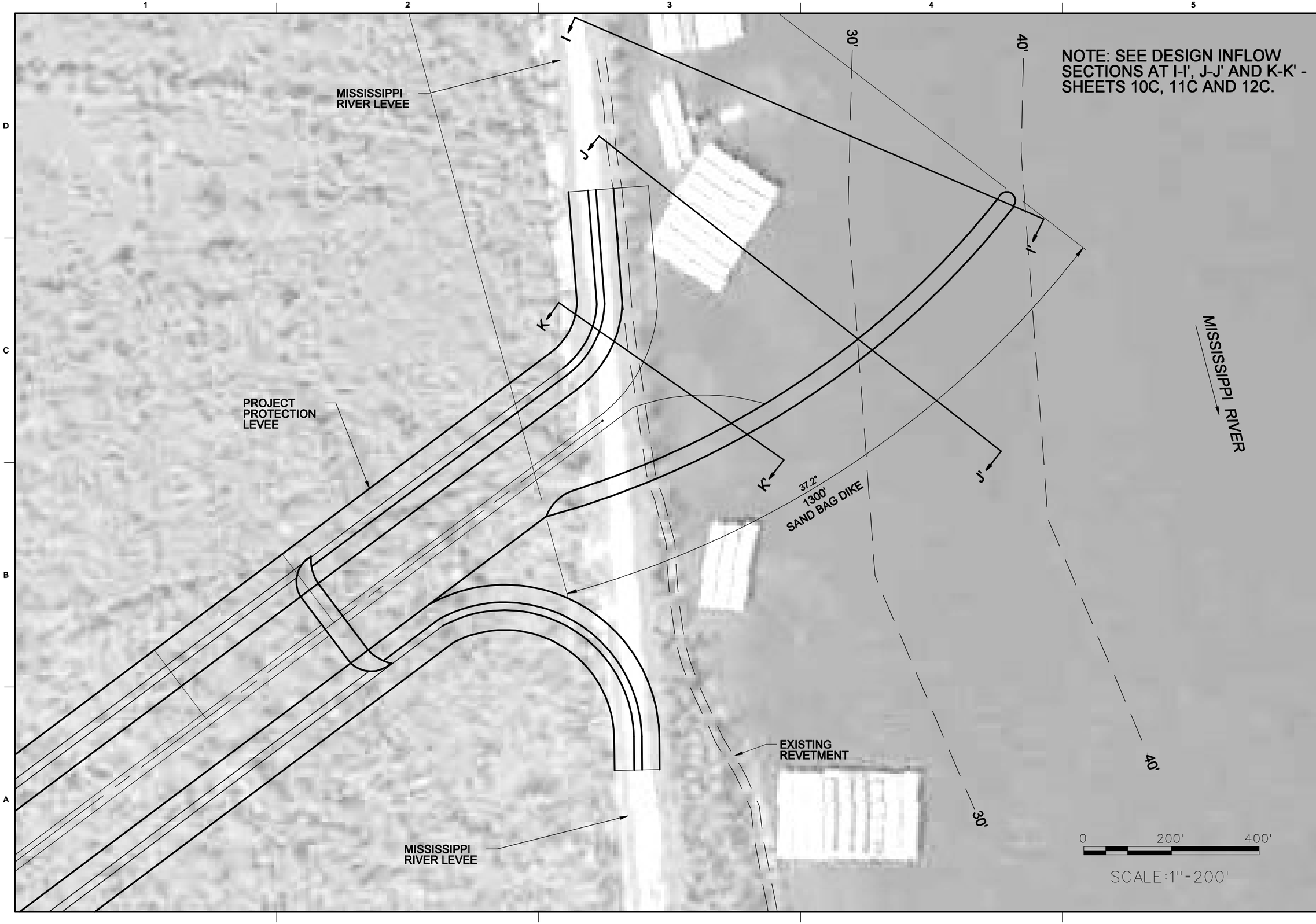
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=99.80 END TRANSITION IN MARSH

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION

SHEET IDENTIFICATION
8C

auto_time



MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Gernoch
PLOT SCALE: 1" = 200'
PLOT DATE: August 2010
SIZE: ANSI D
FILE NAME: Q-9B2loo.dgn
DATE: AUGUST, 2010

BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

MYRTLE GROVE OPTION 3 - RIVER TIE-IN DIVERSION CHANNEL

CONCEPTUAL PLAN - VOLUME II

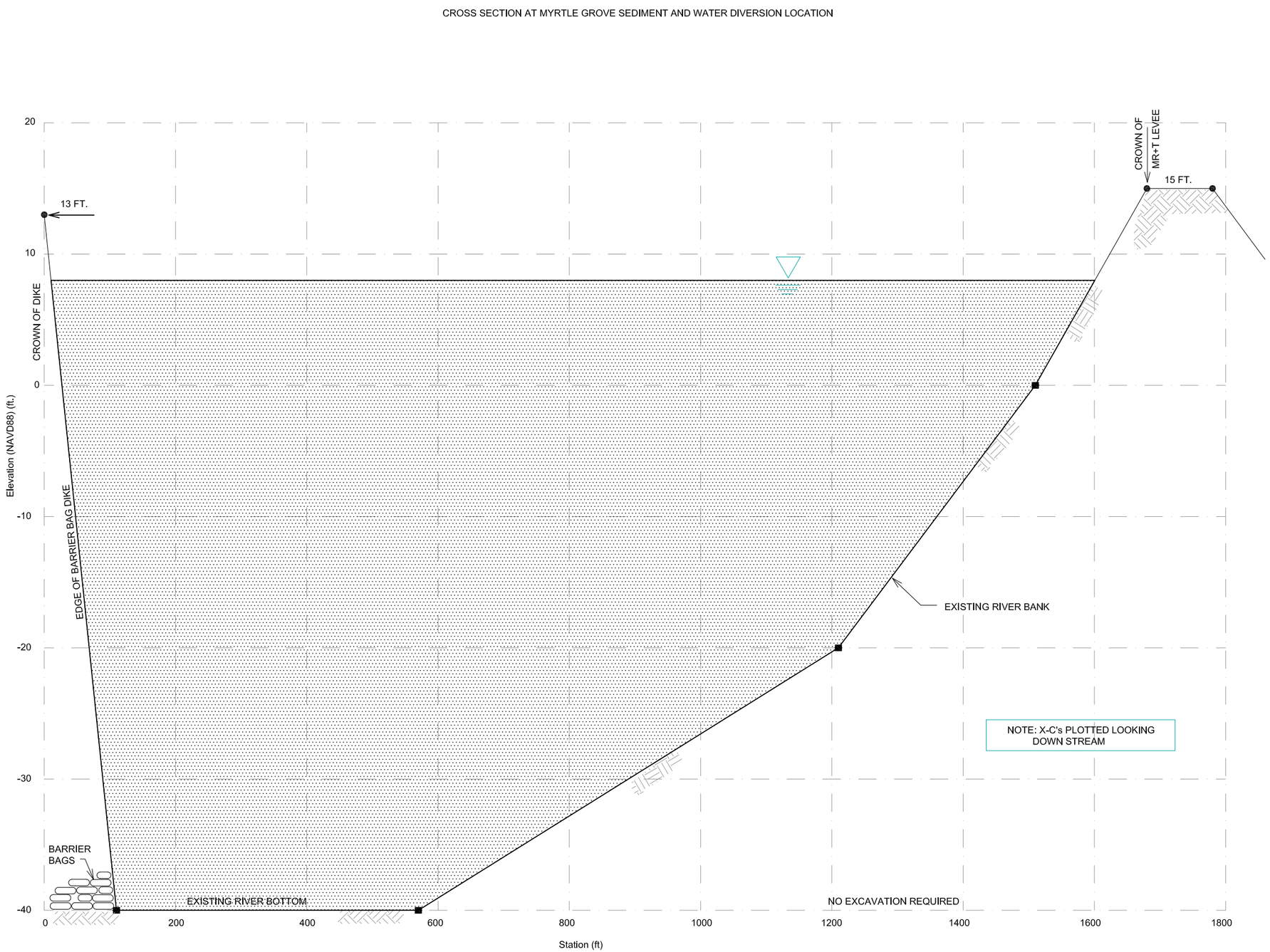
SHEET IDENTIFICATION
9C

auto_time

...\FIG 9C PL OPT 3 blowup.dgn 01/11/2011 3:07:35 PM

RS=289.47 MISSISSIPPI RIVER INFLOW

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



SECTION I-I'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: CWS
DWN BY: CWS
CKD BY: CWS
SUBMITTED BY: Rodney J Guinsch
DATE: AUGUST 2010
PLOT SCALE: 1
PLOT DATE: August, 2010
SIZE: ANSI D
FILE NAME:

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

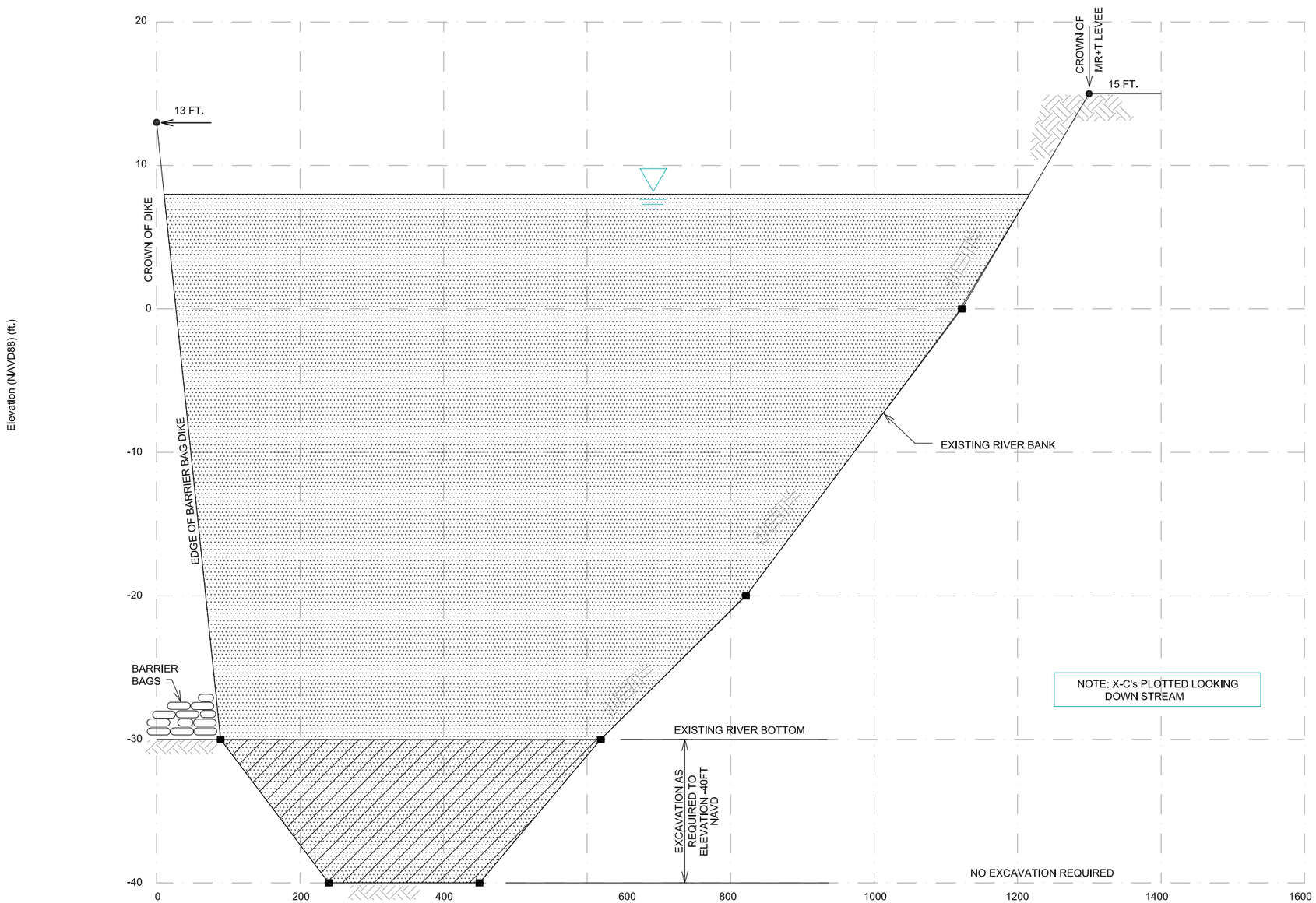
RS=289.47 MISSISSIPPI RIVER INFLOW
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION

SHEET IDENTIFICATION
10C

auto_time

RS=284.14 M.R. INFLOW TO FLUME

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



SECTION J-J'

Legend
EG Max WS.
WS Max WS.
Ground.
Bank Sta.

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: CWS | DATE: AUGUST 2010
DWN BY: CWS | CKD BY: CWS
SUBMITTED BY: Rodney J Guimuch
PLOT SCALE: | PLOT DATE: August, 2010
SIZE: ANSI D | FILE NAME:

BCG Engineering & Consulting, Inc. Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

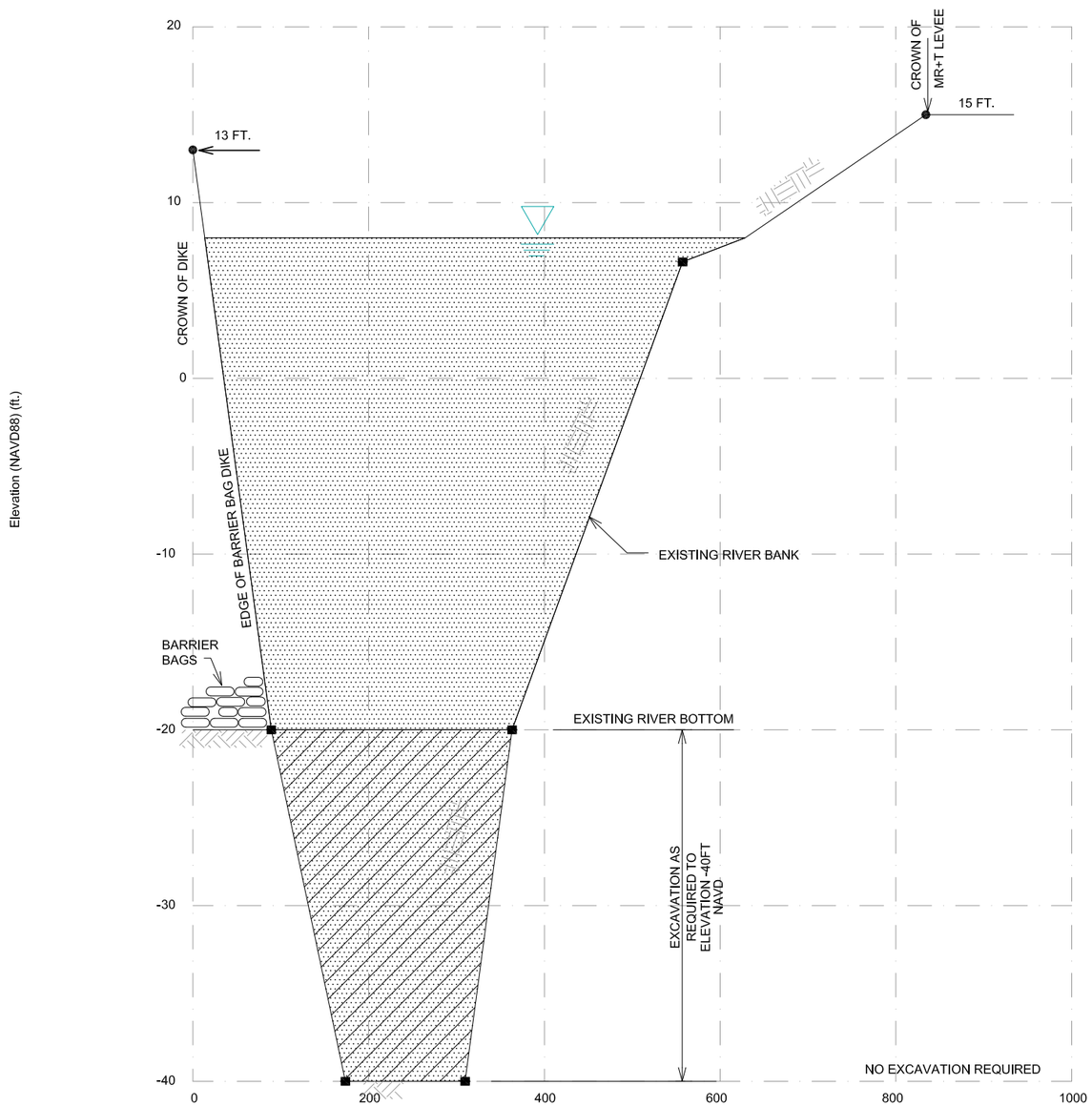
RS=284.14 MISSISSIPPI RIVER INFLOW TO FLUME
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION

SHEET IDENTIFICATION
11C

auto_time

RS=279.50

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



SECTION K-K'

Station (ft)

Legend

- EG Max WS.
- WS Max WS.
- Ground.
- Bank Sta.

NOTE: X-C's PLOTTED LOOKING DOWN STREAM

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc. Excellence in Engineering

DESIGNED BY: CWS | DATE: AUGUST 2010
DWN BY: CWS | CKD BY: CWS
SUBMITTED BY: Rodney J Guenich
PLOT SCALE: | PLOT DATE: August, 2010
SIZE: ANSI D | FILE NAME:

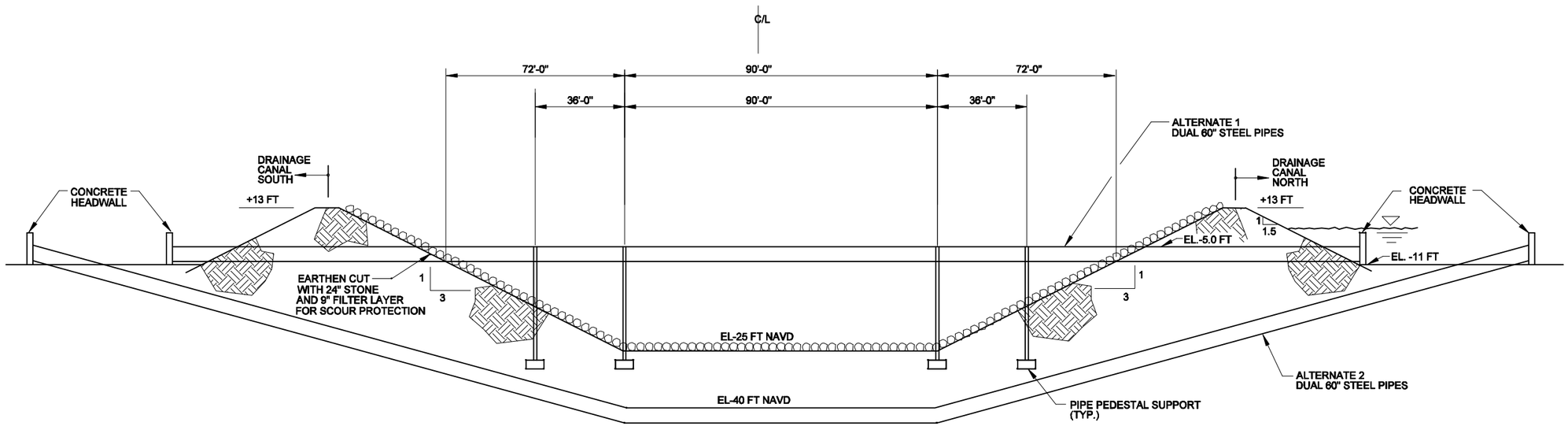
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=279.50

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION

SHEET IDENTIFICATION
12C

auto_time



TYPICAL INVERTED SIPHON ALTERNATIVES
DRAINAGE CANAL CROSSINGS
MYRTLE GROVE WATER AND SEDIMENT DIVERSION PROJECT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DATE: AUGUST, 2010
DWN BY: BC3
CKD BY: CWS
SUBMITTED BY: Rodney J. Cormier
PLOT SCALE: As Noted
PLOT DATE: August, 2010
SIZE: ANSI D
FILE NAME: FIG 13_TYP INVERT SIPHON.DGN

BCG Engineering & Consulting, Inc.
Excellence in Engineering

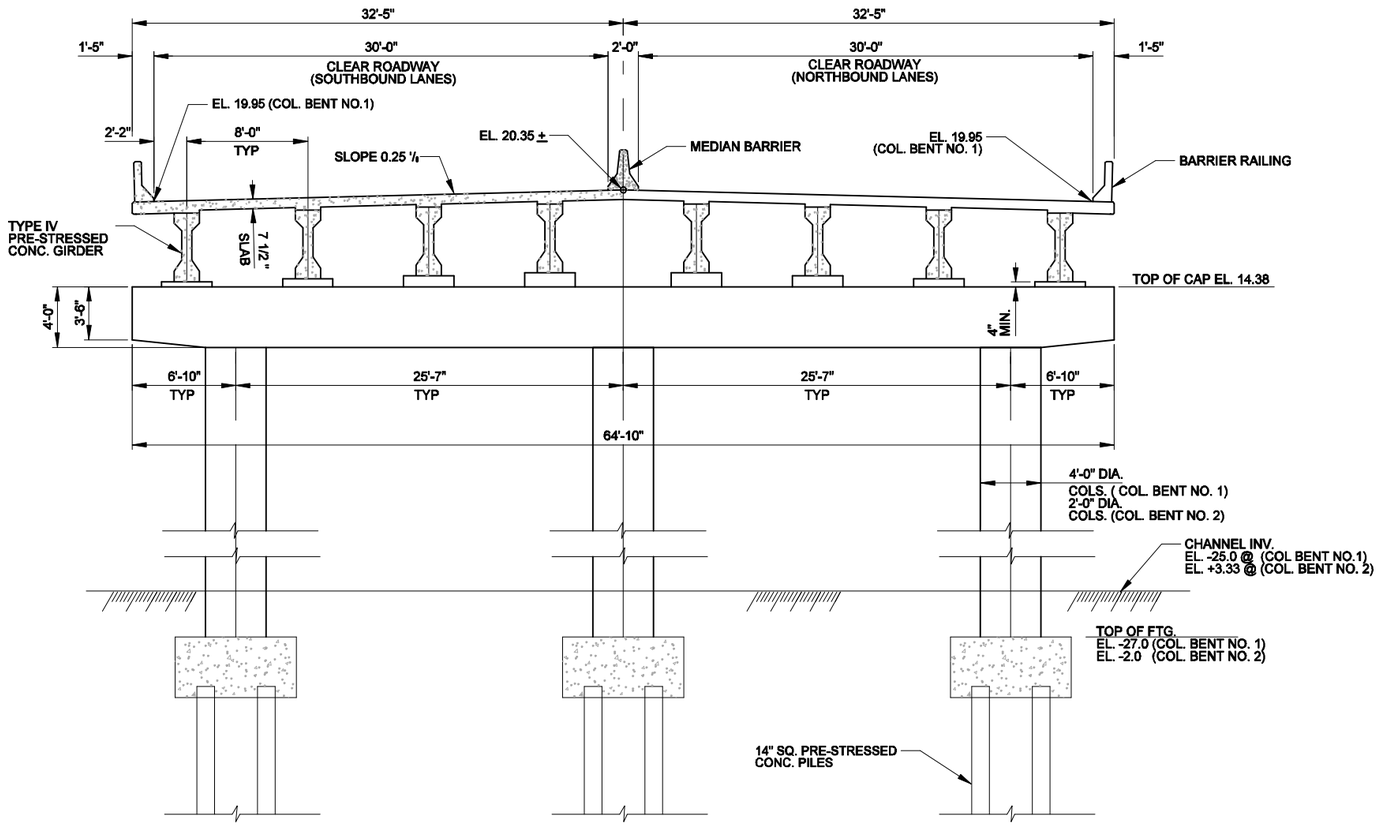
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

TYPICAL SECTIONS
CONCEPTUAL PLAN – VOLUME III

SHEET IDENTIFICATION
13C

auto_time

..\FIG 13_TYP. INVERT SIPHON.dgn 01/11/2011 3:07:51 PM

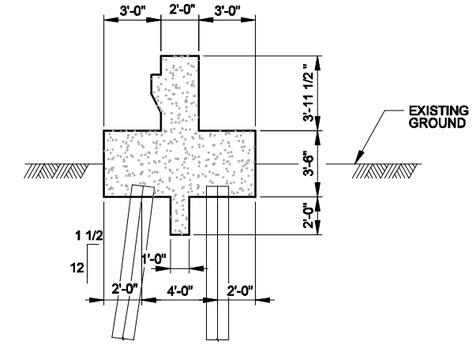


TYPICAL COLUMN BENT

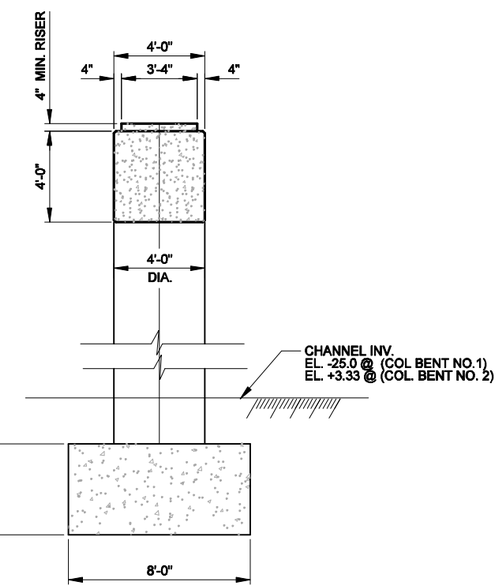
COL. BENT NO. 1, ONE(1) REQUIRED
COL. BENT NO. 2, TWO (2) REQUIRED

SCALE: 1" = 5'-0"

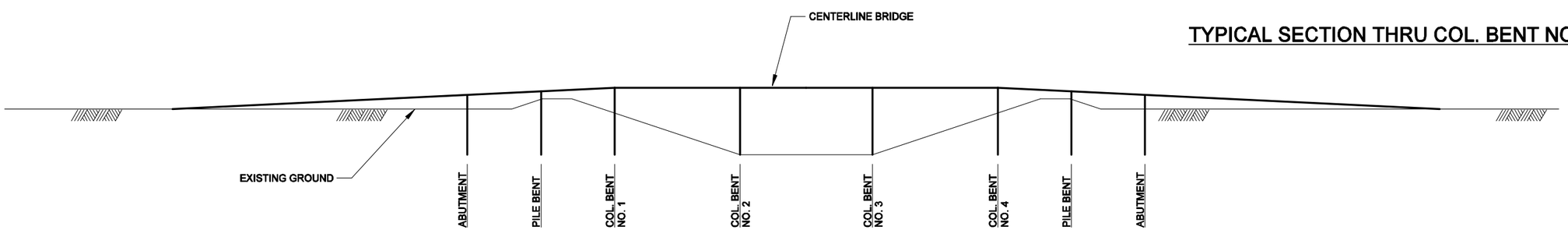
5' 0 5' 10'



TYPICAL ABUTMENT



TYPICAL SECTION THRU COL. BENT NO. 1



TYPICAL PROFILE ALONG C/L BRIDGE

SCALE: 1" = 40'-0"

40' 20' 0 40' 80'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Cannoch
PLOT SCALE: 1" = 10'-0"
PLOT DATE: August 2010
SIZE: ANSI B
FILE NAME: Q-480cross.dgn
DATE: AUGUST, 2010

BCG Engineering & Consulting, Inc.
Excellence in Engineering

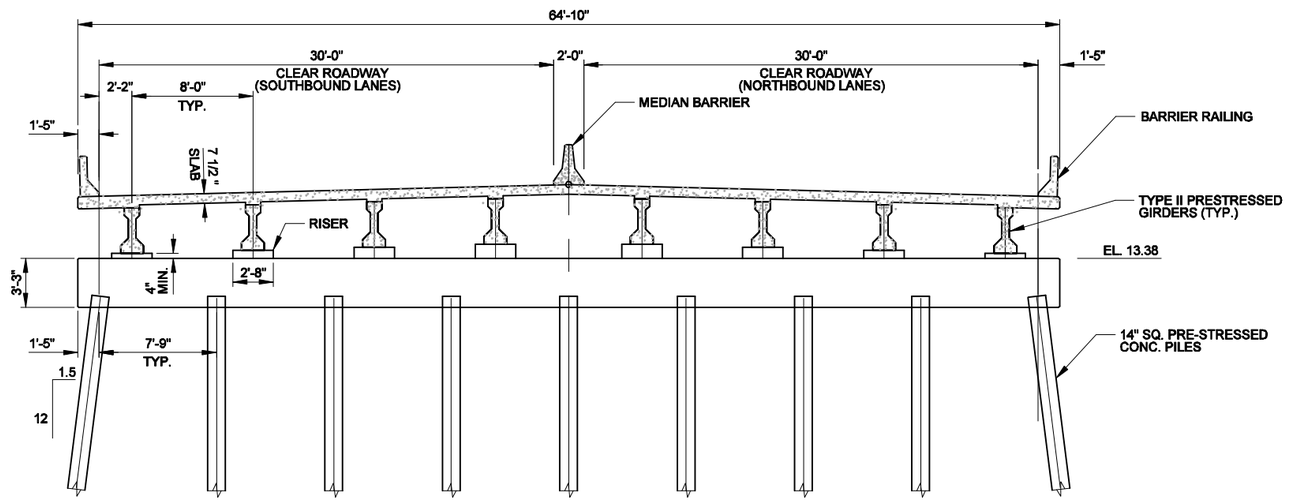
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

CROSS SECTION AND PROFILE
LA. HWY 23 BRIDGE

SHEET IDENTIFICATION
14C

auto_time

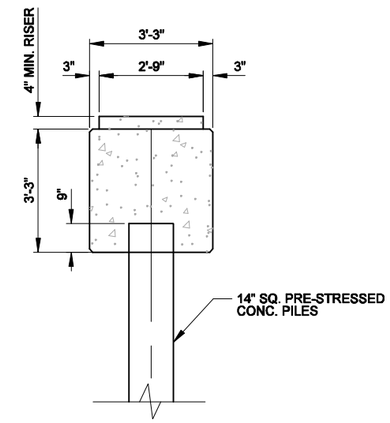
...\FIG 14C bridge section.dgn 01/11/2011 3:08:10 PM



TYPICAL PILE BENT

SCALE: 1" = 5'-0"

5' 0 5' 10'



TYPICAL SECTION THRU PILE BENT

SCALE: ½" = 1'- 0"

12" 0 2' 4' 6' 8'

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: Cecil Soileau
DATE: AUGUST, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Cormier
PLOT SCALE: 1" = 1'-0"
PLOT DATE: August, 2010
SIZE: ANSI B
FILE NAME: G-BSDxxx.dgn

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

CROSS SECTION - PILE BENT
LA. HWY. 23 BRIDGE

SHEET IDENTIFICATION
15C

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...\FIG 15C PILE BENT.dgn 01/11/2011 3:08:28 PM

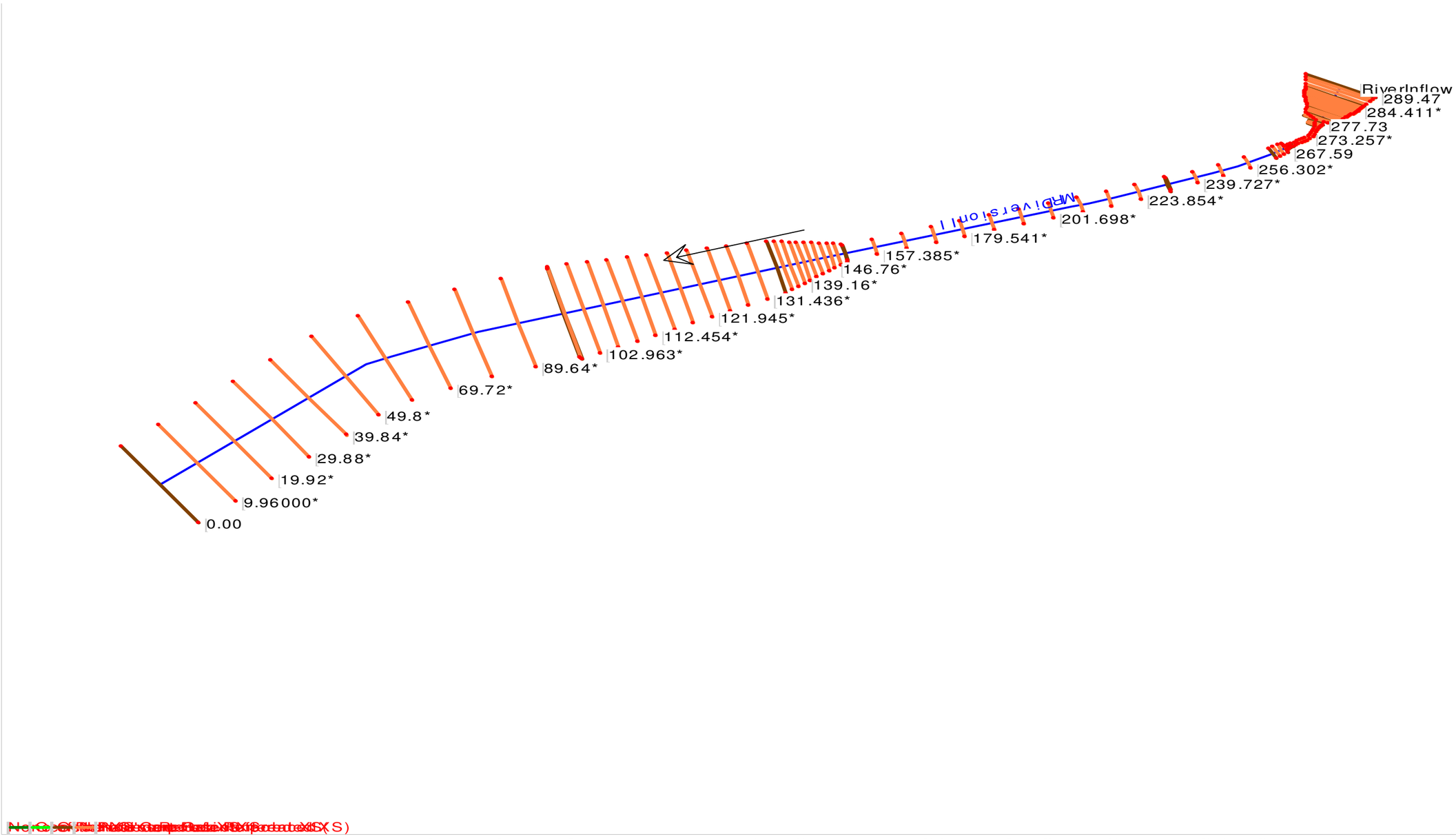
APPENDIX D

HYDRAULIC MODEL OUTPUT

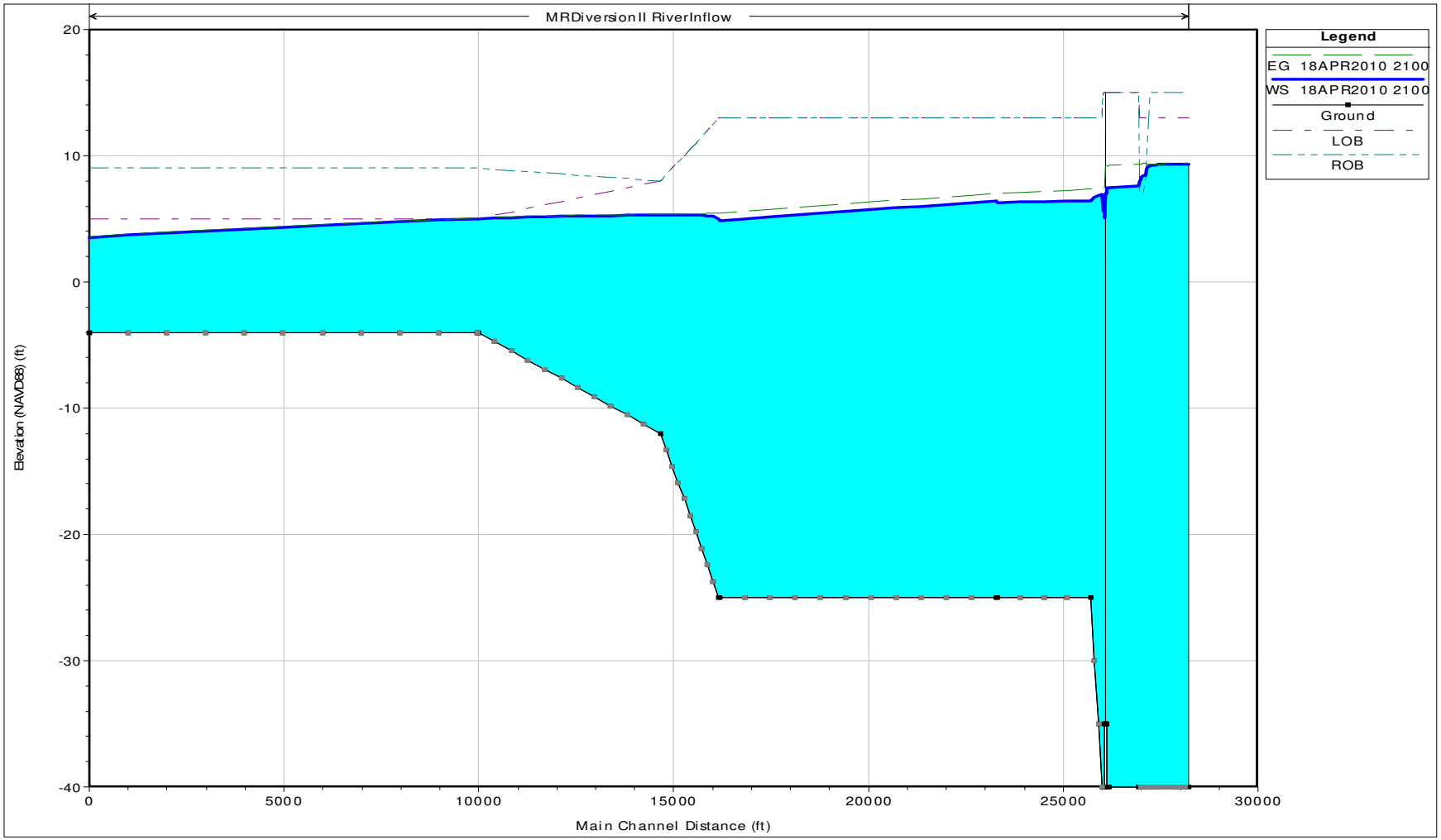
FOR

45,000 CFS ALTERNATIVE

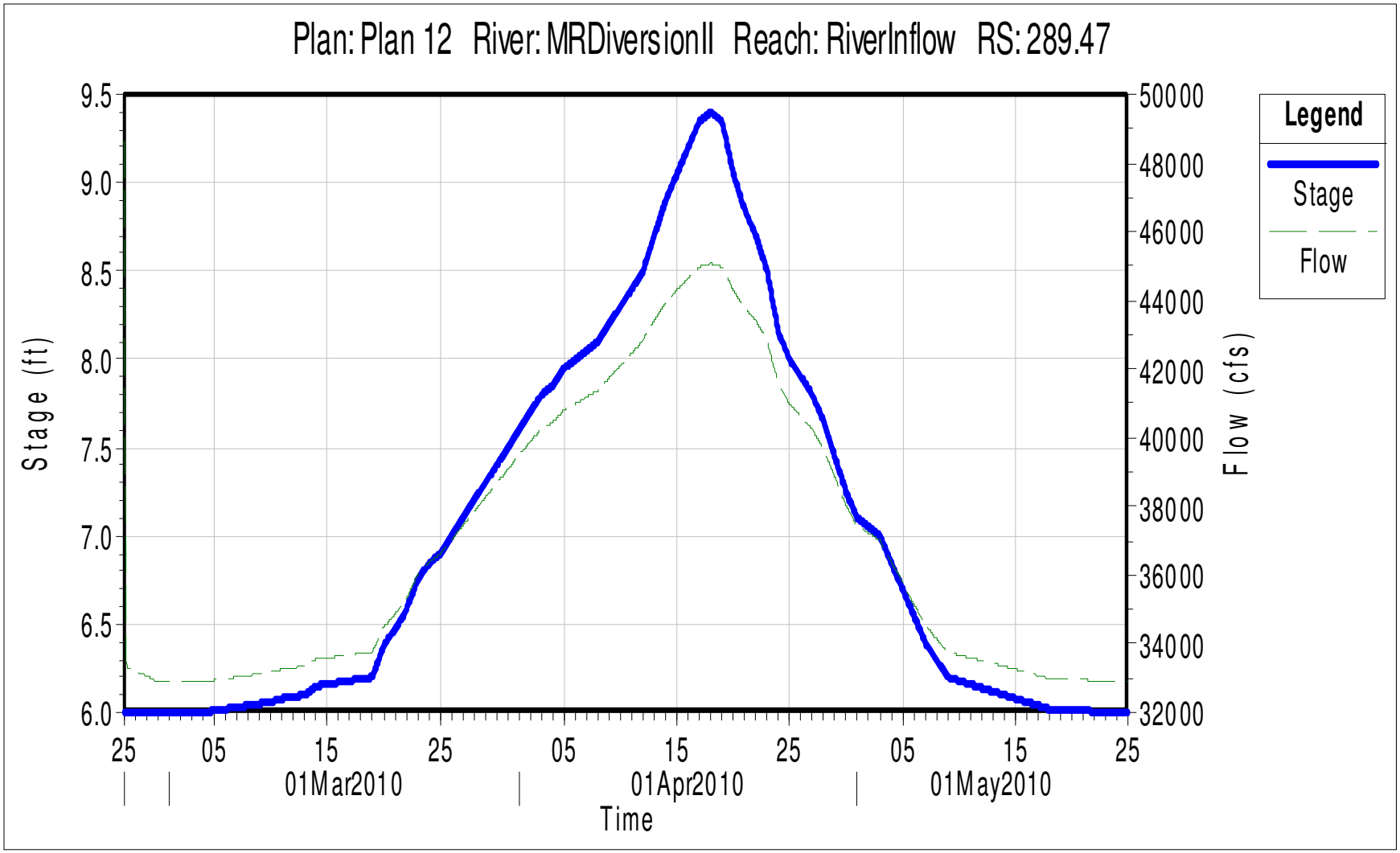
AT RIVER MILE 60.7 NEAR MYRTLE GROVE



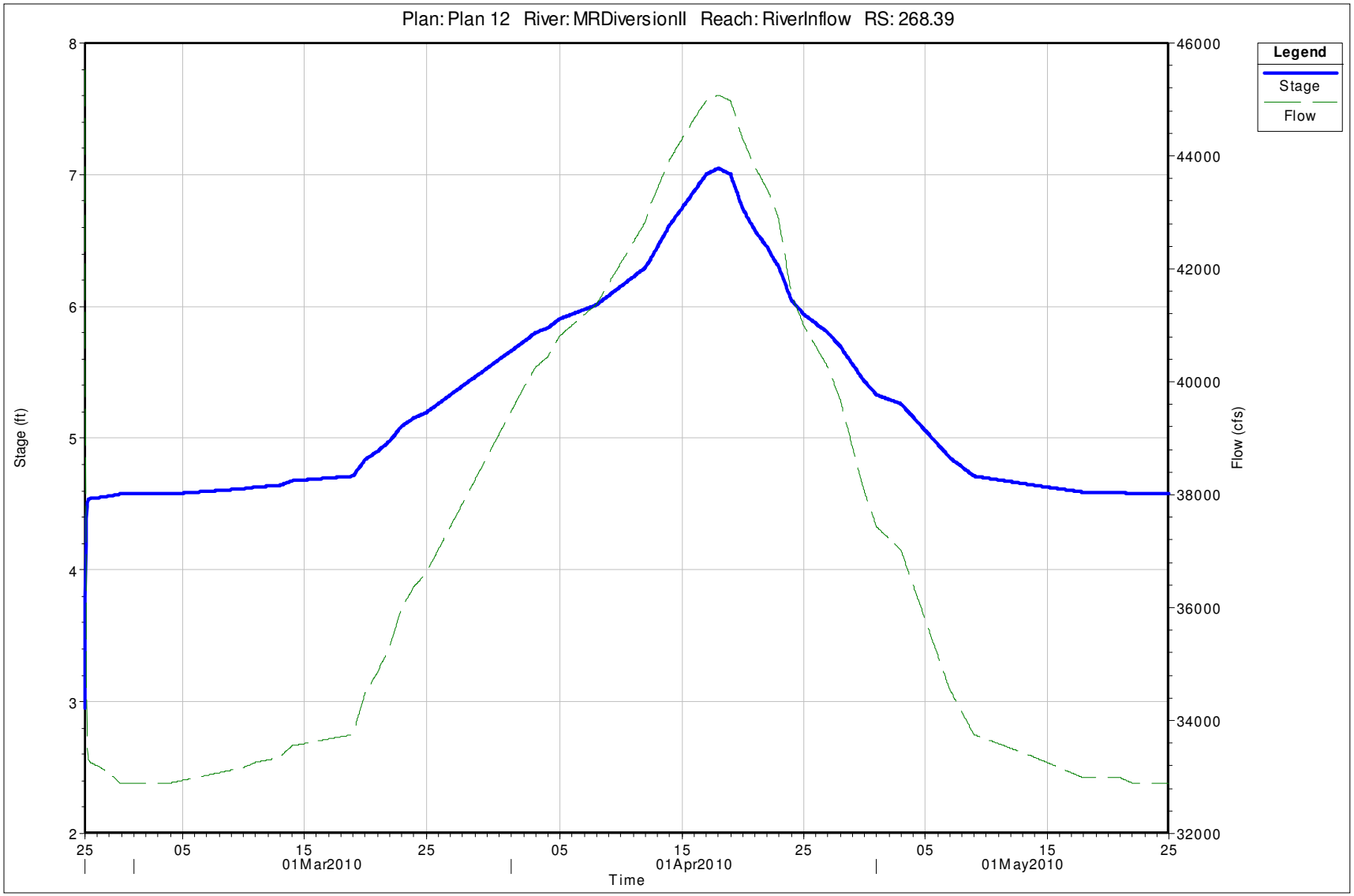
LAYOUT OF 45,000CFS UNSTEADY FLOW MODEL OF PERFERRED ALIGNMENT---CONTROL STRUCTURE IS AT STATION 268.29 AND TRANSITION INTO MARSH BEGINS AT STATION 150.00



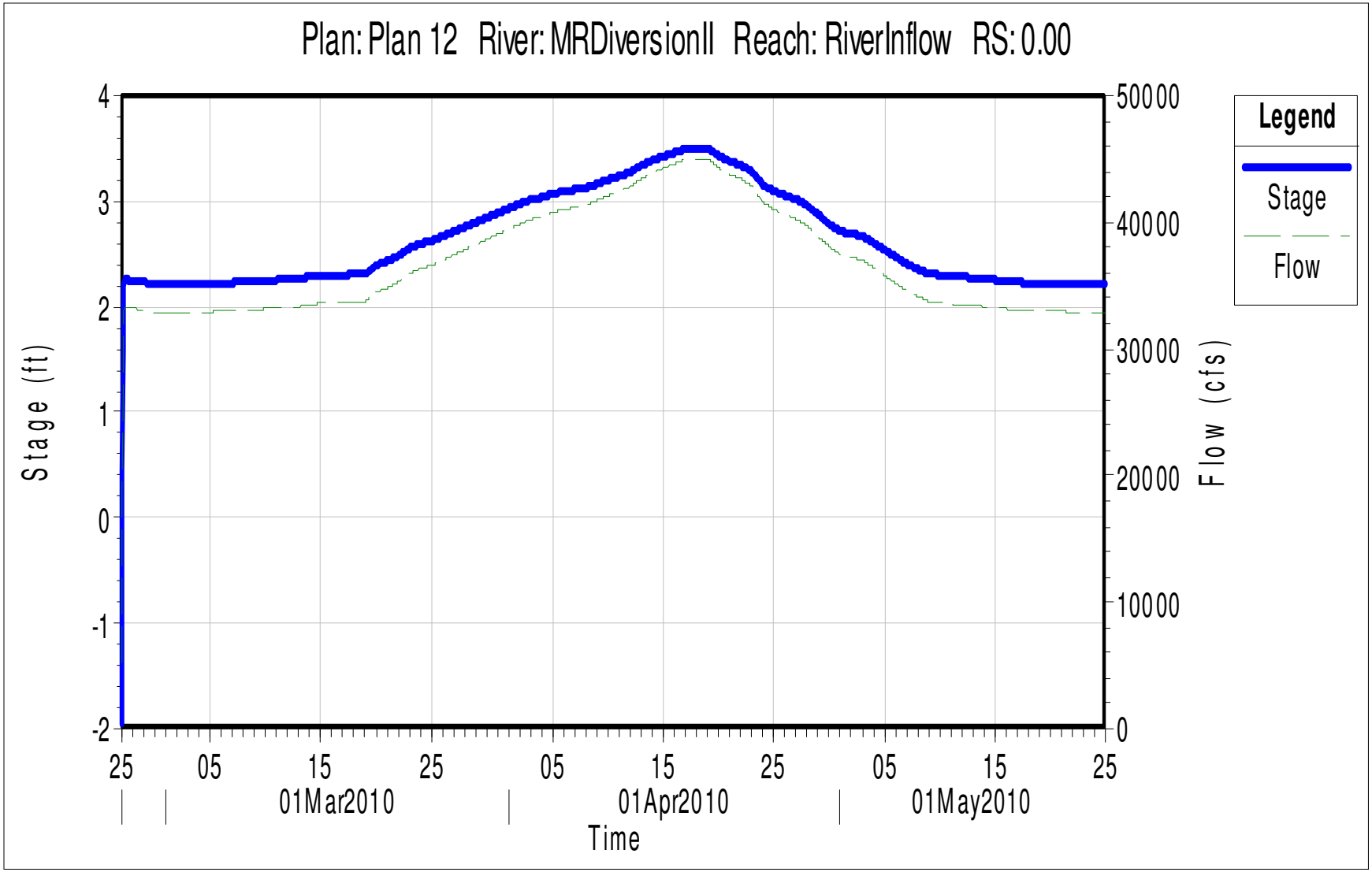
WATER SURFACE, BANK LINE, AND INVERT PROFILE FOR 45,000CFS PROJECT PLAN @ MYRTLE GROVE—RIVER BOUNDRY AT RIGHT AND BEGINNING OF TRANSITION @ 16,000 FT



STAGE AND FLOW HYDROGRAPHS AT THE UPSTREAM END OF MODEL IN MISSISSIPPI RIVER WITH PEAK INFLOW CONTROLLED BY GATE SETTINGS.



STAGE AND FLOW HYDROGRAPHS AT THE UPSTREAM SIDE OF SLUICE GATE IN MODEL



STAGE AND FLOW HYDROGRAPHS AT THE DOWNSTREAM END OF MODEL IN MARSH

HEC-RAS Plan: Plan 12 River: MRDiversionII Reach: RiverInflow Profile: 18APR2010 2100

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
RiverInflow	0.00	18APR2010 2100	44993.50	-4.00	3.50	-1.91	3.58	0.000206	2.29	19685.73	2650.00	0.15
RiverInflow	9.96000*	18APR2010 2100	44993.18	-4.00	3.69		3.77	0.000185	2.24	20104.09	2637.53	0.14
RiverInflow	19.92*	18APR2010 2100	44992.86	-4.00	3.86		3.95	0.000173	2.20	20480.54	2624.86	0.14
RiverInflow	29.88*	18APR2010 2100	44992.55	-4.00	4.05		4.12	0.000163	2.16	20821.97	2612.09	0.13
RiverInflow	39.84*	18APR2010 2100	44992.23	-4.00	4.21		4.28	0.000154	2.13	21133.60	2599.21	0.13
RiverInflow	49.8*	18APR2010 2100	44991.91	-4.00	4.36		4.43	0.000148	2.10	21418.90	2586.19	0.13
RiverInflow	59.76*	18APR2010 2100	44991.60	-4.00	4.50		4.57	0.000140	2.08	21681.02	2573.11	0.13
RiverInflow	69.72*	18APR2010 2100	44991.26	-4.00	4.64		4.70	0.000134	2.05	21922.87	2559.96	0.12
RiverInflow	79.68*	18APR2010 2100	44990.96	-4.00	4.77		4.83	0.000128	2.03	22148.18	2546.74	0.12
RiverInflow	89.64*	18APR2010 2100	44990.65	-4.00	4.90		4.96	0.000124	2.01	22353.11	2533.42	0.12
RiverInflow	99.60	18APR2010 2100	44990.33	-4.00	5.02		5.08	0.000119	2.00	22544.96	2520.10	0.12
RiverInflow	99.7*	18APR2010 2100	44990.33	-4.00	5.02		5.08	0.000113	1.94	23141.27	2590.18	0.11
RiverInflow	99.80	18APR2010 2100	44990.33	-4.00	5.03		5.08	0.000108	1.90	23739.65	2660.27	0.11
RiverInflow	102.983*	18APR2010 2100	44990.19	-4.73	5.07		5.12	0.000089	1.82	24718.54	2557.10	0.10
RiverInflow	106.127*	18APR2010 2100	44990.05	-5.45	5.11		5.16	0.000076	1.77	25485.29	2452.32	0.10
RiverInflow	109.290*	18APR2010 2100	44989.92	-6.18	5.14		5.19	0.000067	1.72	26096.75	2346.57	0.09
RiverInflow	112.454*	18APR2010 2100	44989.80	-6.91	5.17		5.22	0.000059	1.70	26531.49	2240.05	0.09
RiverInflow	115.618*	18APR2010 2100	44989.68	-7.64	5.20		5.24	0.000054	1.68	26792.41	2132.95	0.08
RiverInflow	118.781*	18APR2010 2100	44989.57	-8.36	5.22		5.26	0.000050	1.67	26882.55	2025.45	0.08
RiverInflow	121.945*	18APR2010 2100	44989.46	-9.09	5.24		5.28	0.000047	1.68	26783.89	1917.57	0.08
RiverInflow	125.109*	18APR2010 2100	44989.36	-9.82	5.26		5.30	0.000045	1.70	26537.19	1809.45	0.08
RiverInflow	128.272*	18APR2010 2100	44989.26	-10.55	5.27		5.32	0.000043	1.72	26123.16	1701.09	0.08
RiverInflow	131.436*	18APR2010 2100	44989.16	-11.27	5.29		5.34	0.000043	1.76	25526.83	1592.52	0.08
RiverInflow	134.6	18APR2010 2100	44989.06	-12.00	5.31		5.36	0.000043	1.82	24731.44	1483.84	0.08
RiverInflow	136.12*	18APR2010 2100	44989.05	-13.30	5.31		5.36	0.000041	1.84	24515.69	1379.92	0.08
RiverInflow	137.64*	18APR2010 2100	44989.03	-14.60	5.32		5.37	0.000039	1.88	23947.04	1275.77	0.08
RiverInflow	139.16*	18APR2010 2100	44989.00	-15.90	5.32		5.38	0.000040	1.95	23076.55	1171.49	0.08
RiverInflow	140.68*	18APR2010 2100	44988.98	-17.20	5.32		5.38	0.000042	2.05	21903.34	1067.04	0.08
RiverInflow	142.2*	18APR2010 2100	44988.96	-18.50	5.31		5.39	0.000046	2.20	20428.79	962.52	0.08
RiverInflow	143.72*	18APR2010 2100	44988.94	-19.80	5.31		5.40	0.000054	2.41	18650.52	857.84	0.09
RiverInflow	145.24*	18APR2010 2100	44988.93	-21.10	5.29		5.41	0.000067	2.72	16570.00	753.08	0.10
RiverInflow	146.76*	18APR2010 2100	44988.91	-22.40	5.26		5.42	0.000092	3.17	14182.28	648.09	0.12
RiverInflow	148.28*	18APR2010 2100	44988.90	-23.70	5.20		5.44	0.000147	3.82	11484.77	542.80	0.15
RiverInflow	149.80	18APR2010 2100	44988.89	-25.00	5.04		5.48	0.000307	5.32	8455.86	436.48	0.21
RiverInflow	150.00	18APR2010 2100	44988.69	-25.00	4.84		5.48	0.000227	6.43	7001.20	356.57	0.26
RiverInflow	157.385*	18APR2010 2100	44988.87	-25.00	4.99		5.63	0.000224	6.40	7030.70	357.05	0.25
RiverInflow	164.770*	18APR2010 2100	44988.84	-25.00	5.14		5.77	0.000222	6.37	7059.62	357.53	0.25
RiverInflow	172.156*	18APR2010 2100	44988.81	-25.00	5.29		5.91	0.000219	6.35	7086.46	357.95	0.25
RiverInflow	179.541*	18APR2010 2100	44988.78	-25.00	5.43		6.05	0.000217	6.32	7114.13	358.42	0.25
RiverInflow	186.927*	18APR2010 2100	44988.75	-25.00	5.58		6.19	0.000214	6.30	7141.20	358.88	0.25
RiverInflow	194.312*	18APR2010 2100	44988.71	-25.00	5.72		6.33	0.000212	6.28	7167.71	359.34	0.25
RiverInflow	201.698*	18APR2010 2100	44988.68	-25.00	5.86		6.47	0.000210	6.25	7193.68	359.80	0.25
RiverInflow	209.083*	18APR2010 2100	44988.65	-25.00	6.00		6.60	0.000208	6.23	7219.14	360.25	0.25
RiverInflow	216.469*	18APR2010 2100	44988.61	-25.00	6.14		6.74	0.000206	6.21	7242.73	360.65	0.24
RiverInflow	223.854*	18APR2010 2100	44988.57	-25.00	6.27		6.87	0.000204	6.19	7267.29	361.10	0.24
RiverInflow	231.24	18APR2010 2100	44988.54	-25.00	6.41		7.00	0.000202	6.17	7291.35	361.55	0.24
RiverInflow	231.44	18APR2010 2100	44988.53	-25.00	6.30		7.00	0.000127	6.72	6895.80	307.81	0.26
RiverInflow	239.727*	18APR2010 2100	44988.50	-25.00	6.33		7.08	0.000138	6.95	6469.82	300.49	0.26
RiverInflow	248.015*	18APR2010 2100	44988.47	-25.00	6.36		7.17	0.000150	7.21	6243.87	293.17	0.28
RiverInflow	256.302*	18APR2010 2100	44988.45	-25.00	6.39		7.26	0.000165	7.48	6017.92	285.87	0.29
RiverInflow	264.59	18APR2010 2100	44988.42	-25.00	6.43		7.37	0.000181	7.77	5791.98	278.57	0.30
RiverInflow	265.59*	18APR2010 2100	44988.41	-30.00	6.70		7.38	0.000109	6.63	6789.32	280.02	0.24
RiverInflow	266.59*	18APR2010 2100	44988.41	-35.00	6.86		7.39	0.000072	5.82	7734.92	279.55	0.19
RiverInflow	267.59	18APR2010 2100	44988.40	-40.00	6.97		7.39	0.000050	5.21	8640.15	277.89	0.16
RiverInflow	267.69*	18APR2010 2100	44988.40	-40.00	6.62		7.40	0.000075	7.11	6328.17	181.54	0.21
RiverInflow	267.79	18APR2010 2100	44988.40	-40.00	5.58		7.45	0.000191	10.97	4101.84	90.07	0.29
RiverInflow	268.14	18APR2010 2100	44988.40	-40.00	5.69		7.46	0.000191	10.97	4102.59	90.07	0.29
RiverInflow	268.19	18APR2010 2100	44988.40	-35.00	5.04		7.46	0.000271	12.49	3603.06	90.06	0.35
RiverInflow	268.29		Inl Struct									
RiverInflow	268.39	18APR2010 2100	44988.40	-35.00	7.01	-15.25	9.21	0.000238	11.90	3780.15	90.07	0.32
RiverInflow	268.44	18APR2010 2100	44988.40	-40.00	7.49		9.21	0.000172	10.53	4273.15	90.07	0.27
RiverInflow	268.89	18APR2010 2100	44988.40	-40.00	7.49		9.22	0.000171	10.53	4273.90	90.07	0.27
RiverInflow	269.09	18APR2010 2100	44988.40	-40.00	7.50		9.22	0.000171	10.53	4274.23	90.07	0.27
RiverInflow	276.73	18APR2010 2100	44988.36	-40.00	7.64		9.35	0.000170	10.49	4289.50	90.09	0.27
RiverInflow	276.93	18APR2010 2100	44988.36	-40.00	7.99		9.35	0.000280	9.35	4820.32	132.45	0.25
RiverInflow	277.13*	18APR2010 2100	44988.36	-40.00	8.17		9.36	0.000231	8.76	5145.27	148.53	0.24
RiverInflow	277.33*	18APR2010 2100	44988.36	-40.00	8.29		9.36	0.000200	8.30	5436.70	163.15	0.23
RiverInflow	277.53*	18APR2010 2100	44988.36	-40.00	8.39		9.36	0.000179	7.83	5687.61	175.70	0.23
RiverInflow	277.73	18APR2010 2100	44988.35	-40.00	8.46		9.37	0.000163	7.64	5908.04	183.53	0.22
RiverInflow	276.55	18APR2010 2100	44988.34	-40.00	8.42		9.38	0.000161	7.86	5727.61	173.02	0.23
RiverInflow	276.787*	18APR2010 2100	44988.34	-40.00	8.95		9.36	0.000080	5.09	8836.38	311.97	0.17
RiverInflow	279.025*	18APR2010 2100	44988.34	-40.00	9.11		9.36	0.000055	4.01	11218.74	443.30	0.14
RiverInflow	279.262*	18APR2010 2100	44988.33	-40.00	9.17		9.36	0.000044	3.48	12928.37	543.50	0.13
RiverInflow	279.50	18APR2010 2100	44988.32	-40.00	9.20		9.36	0.000034	3.17	14185.09	565.17	0.11
RiverInflow	280.225*	18APR2010 2100	44988.30	-40.00	9.25		9.36	0.000025	2.66	16889.38	702.78	0.10

HEC-RAS Plan: Plan 12 River: MRDiversionII Reach: RiverInflow Profile: 18APR2010 2100 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
RiverInflow	280.953*	18APR2010 2100	44988.27	-40.00	9.28		9.36	0.000016	2.25	19969.98	777.97	0.08
RiverInflow	281.68	18APR2010 2100	44988.23	-40.00	9.31		9.36	0.000011	1.93	23290.15	838.07	0.06
RiverInflow	281.987*	18APR2010 2100	44988.21	-40.00	9.31		9.36	0.000009	1.82	24724.99	877.30	0.06
RiverInflow	282.295*	18APR2010 2100	44988.20	-40.00	9.32		9.36	0.000008	1.72	26211.40	916.59	0.06
RiverInflow	282.602*	18APR2010 2100	44988.18	-40.00	9.32		9.37	0.000007	1.62	27745.19	955.00	0.05
RiverInflow	282.91*	18APR2010 2100	44988.16	-40.00	9.33		9.37	0.000006	1.53	29326.53	995.44	0.05
RiverInflow	283.217*	18APR2010 2100	44988.15	-40.00	9.33		9.37	0.000006	1.45	30958.71	1034.95	0.05
RiverInflow	283.525*	18APR2010 2100	44988.13	-40.00	9.34		9.37	0.000005	1.38	32639.61	1074.51	0.04
RiverInflow	283.832*	18APR2010 2100	44988.11	-40.00	9.34		9.37	0.000004	1.31	34366.64	1114.17	0.04
RiverInflow	284.14	18APR2010 2100	44988.09	-40.00	9.34		9.37	0.000004	1.24	36146.27	1153.86	0.04
RiverInflow	284.411*	18APR2010 2100	44988.07	-40.00	9.34		9.37	0.000004	1.22	36930.29	1170.19	0.04
RiverInflow	284.682*	18APR2010 2100	44988.05	-40.00	9.34		9.37	0.000003	1.19	37717.61	1186.57	0.04
RiverInflow	284.954*	18APR2010 2100	44988.03	-40.00	9.34		9.37	0.000003	1.17	38507.25	1202.90	0.04
RiverInflow	285.225*	18APR2010 2100	44988.02	-40.00	9.35		9.37	0.000003	1.14	39298.38	1219.27	0.04
RiverInflow	285.497*	18APR2010 2100	44988.00	-40.00	9.35		9.37	0.000003	1.12	40089.78	1235.58	0.03
RiverInflow	285.768*	18APR2010 2100	44987.97	-40.00	9.35		9.37	0.000003	1.10	40885.69	1251.95	0.03
RiverInflow	286.04	18APR2010 2100	44987.95	-40.00	9.35		9.37	0.000003	1.08	41682.44	1268.26	0.03
RiverInflow	286.897*	18APR2010 2100	44987.89	-40.00	9.35		9.37	0.000002	1.01	44429.28	1337.57	0.03
RiverInflow	287.755*	18APR2010 2100	44987.82	-40.00	9.35		9.37	0.000002	0.95	47359.45	1406.81	0.03
RiverInflow	288.612*	18APR2010 2100	44987.75	-40.00	9.35		9.37	0.000002	0.89	50471.03	1476.29	0.03
RiverInflow	289.47	18APR2010 2100	44987.68	-40.00	9.36		9.37	0.000001	0.84	53768.83	1545.75	0.02

APPENDIX E

TEXT AND COST ESTIMATE
Drawings 1E to 14E

MARCH 6, 2011

APPENDIX E

1. INTRODUCTION. It is the intent of the OCPRA that the variability of possible sizes of water and sediment diversions be analyzed within a reasonable set of limits between 15,000 cfs and 75,000 cfs, to bracket the likely benefits as well as the effects to the receiving side of the diversion. To achieve this OCPRA has requested that BCG produce drawings of the proposed structure and diversion channel dimensions, a one dimensional unsteady flow model of the diversion project, which extends from the Mississippi River to some three (3) miles into the receiving marsh, and a construction cost estimate for that project. Finally BCG has been requested to revise this draft report to incorporate this additional alternative. All of the hydrologic data included in the draft report is also applicable to this conceptual design; therefore none of that data is repeated in this Appendix E. The model hydraulic output for the 75,000 CFS alternative is presented in Appendix F. The control structure will have four (4) – 32' X 49' sluice gates to control outflow to marshes.

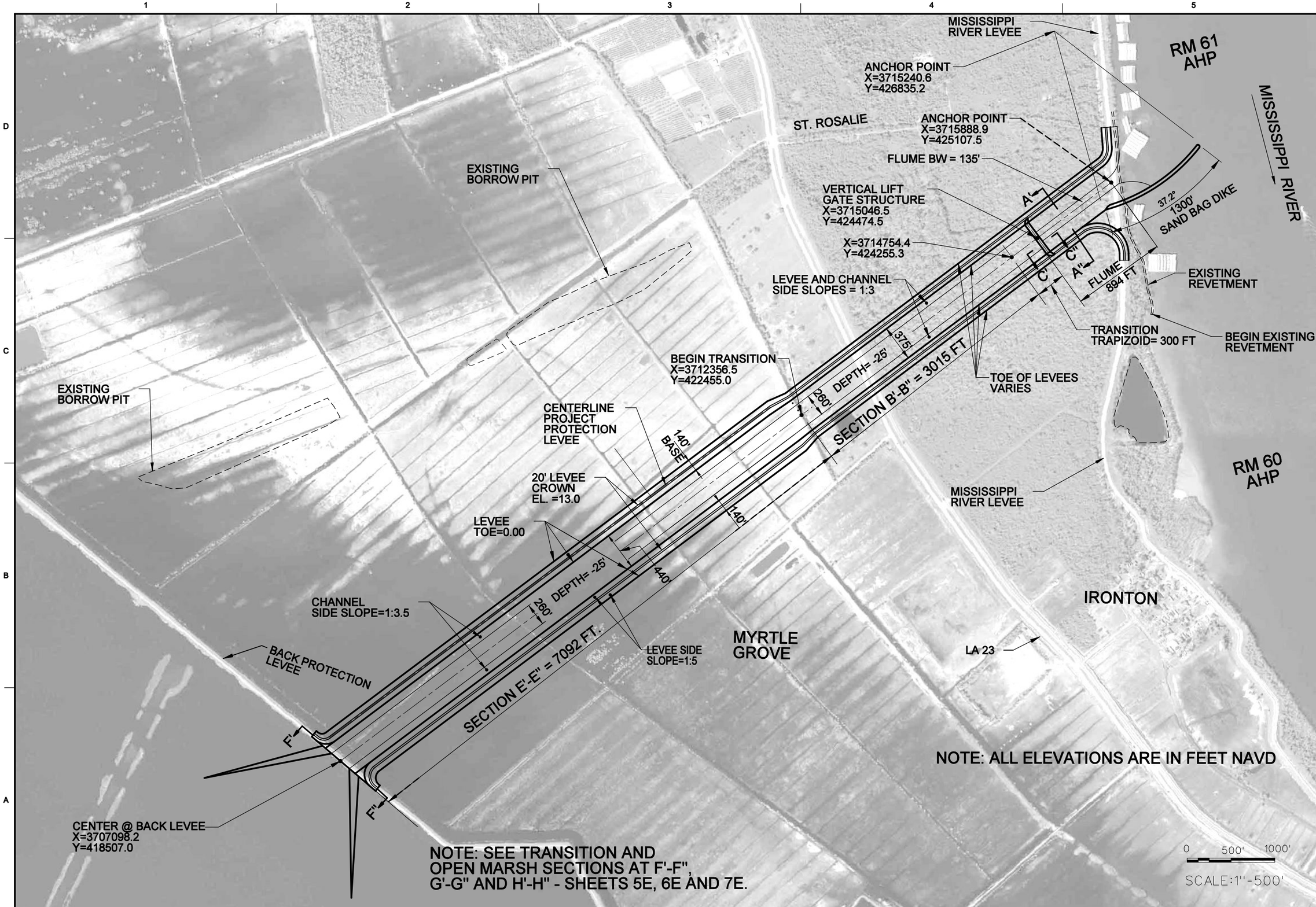
2. CONCLUSIONS. See the Drawing 1E for a Vicinity Map of the project area and the preferred alignment for this conceptual design. Drawings 2E through 12E show the dimensions of the gated structure, dikes, flume, and diversion channel sized to pass 75,000 cfs of water and sediment over the useful life of the project, which is expected to be 100 years. Drawings 13E and 14E show the required dimensions of a highway bridge on LA 23 that would be required at the crossing of the Diversion Channel. Finally Table E-1 is a construction cost estimate for this alternative, which is $450 million.

TABLE 1-E

BUDGETARY COST ESTIMATE
MYRTLE GROVE DIVERSION CHANNEL (75,000 CFS)

ITEM NO.	DESCRIPTION	UNIT	QUANITY	UNIT PRICE	AMOUNT	
1.	Mobilization	Lump			$3,750,000.00	
2.	Excavation	CY	3,927,419.00	$10.00	$39,274,910.00	
3.	Dredging	CY	6,190,512.00	$25.00	$154,762,800.00	
4.	Levees	CY	1,255,864.55	$14.00	$17,582,103.74	
5.	Rip-rap	Tons	468,342.00	$70.00	$32,783,940.00	
6.	Crushed Stones	CY	177,741.00	$55.00	$9,775,755.00	
7.	Geo-textile Fabric	SY	667,700.00	$10.00	$6,667,000.00	
8.	Dike (Sand-filled Geo-tubes)	CY	365,251.82	$20.00	$7,305,036.47	
8a.	Spread Excess Excavation On Site	CY	2,671,554.45	2.00	$5,343,108.89	
	Total Channel Construction					**$277,253,934.10**
9.	Reinforced Concrete					
	A. Flume:					
	1. Base Slab	CY	57,399.00	$500.00	$28,699,500.00	
	2. Walls	CY	10,022.00	$700.00	$7,015,400.00	
	B. T-walls:					
	1. Base Slab	CY	700.00	$500.00	$350,000.00	
	2. Walls	CY	570.00	$700.00	$399,000.00	
	C. Gate Structure:					
	1. Base Slab	CY	2,625.00	$500.00	$1,312,500.00	
	2. Piers	CY	930.00	$700.00	$651,000.00	
	D. Pile Caps for 60 inch Pipe	CY	26.00	$700.00	$18,200.00	
	Total Reinforced Concrete					**$38,445,600.00**
10.	Structural Steel Gates	Tons	108.00	$30,000.00	$3,240,000.00	
	Total Gates					**$3,240,000.00**
11.	Piling					
	HP 14 x 73	L.F.	468,800.00	$75.00	$35,160,000.00	
	12 inch dia. Steel Pipe Piles	L.F.	1,000.00	$75.00	$75,000.00	
12.	PZ-22 Cut-off	SF	12,380.00	$45.00	$557,100.00	
	Total Piling					**$35,792,100.00**
13.	60 inch dia. x 3/8 inch w.t.					
	AWWA C200 Pipe	L.F.	790.00	$1,200.00	$948,000.00	
	Total Steel Pipe					**$948,000.00**
14.	20 inch Dia. Sched 80 Shell Oil Overhead Pipe Crossing and Pipe Supports	L.F.	620.00	$450.00	$279,000.00	
	Total 20 inch Diameter Pipe					**$279,000.00**
15.	40 T Gantry Crane	Each	1.00	$500,000.00	$500,000.00	
	Total Gantry Crane					**$500,000.00**
16.	LA 23 Bridge	L.F.	1,230.00	$14,625.00	$17,988,750.00	
	Total LA 23 Bridge					**$17,988,750.00**
17.	Demobilization	Lump		$1,000,000.00	$1,000,000.00	
	Total Demobilization					**$1,000,000.00**
	Total W/O Contingencies					**$375,447,384.10**
18.	Contingencies (20%)	Lump			$75,089,476.82	
	TOTAL PROJECT COST					**$450,536,860.93**

NOTES:
1. All prices are in current dollars and do not include future inflation.
2. Quantities shown are based on preliminary design only and are not all inclusive.
3. Land acquisition costs not included.



MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Glemuch
PLOT SCALE: 1" = 500'
PLOT DATE: May 2010
SIZE: ANSI D
FILE NAME: FIG 2C.DGN

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc. Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

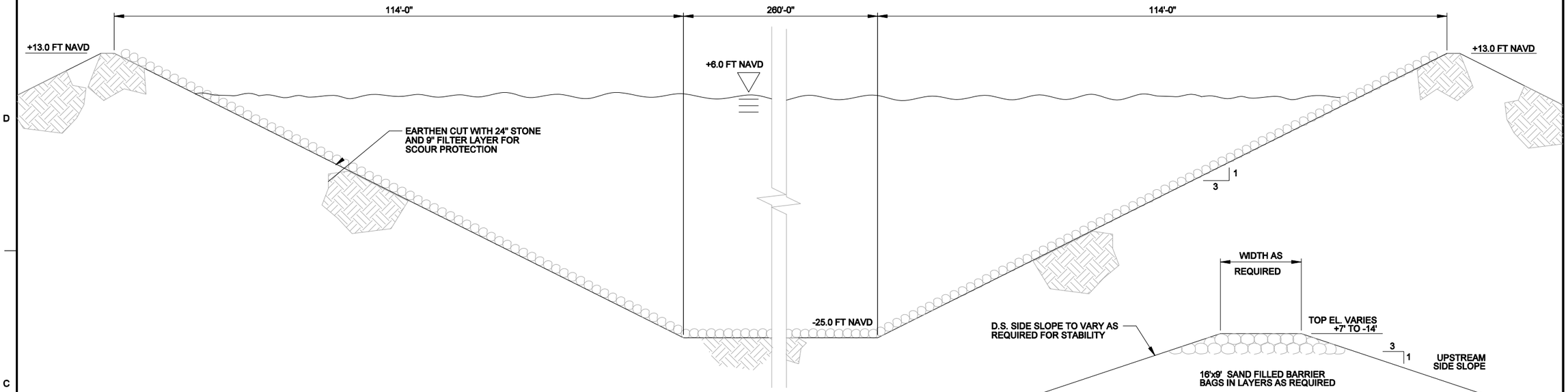
MYRTLE GROVE OPTION 5
75,000 CFS DIVERSION CHANNEL

CONCEPTUAL PLAN - VOLUME IV

SHEET IDENTIFICATION
1E

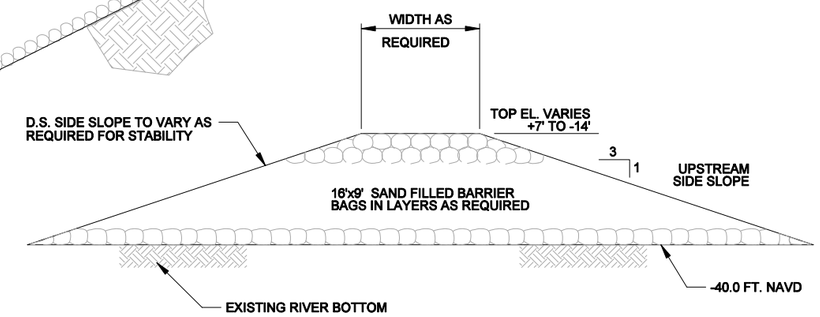
auto_time

...\VOLUME 4\FIG 1e PL OPT 5.dgn 27/10/2011 10:07:53 AM



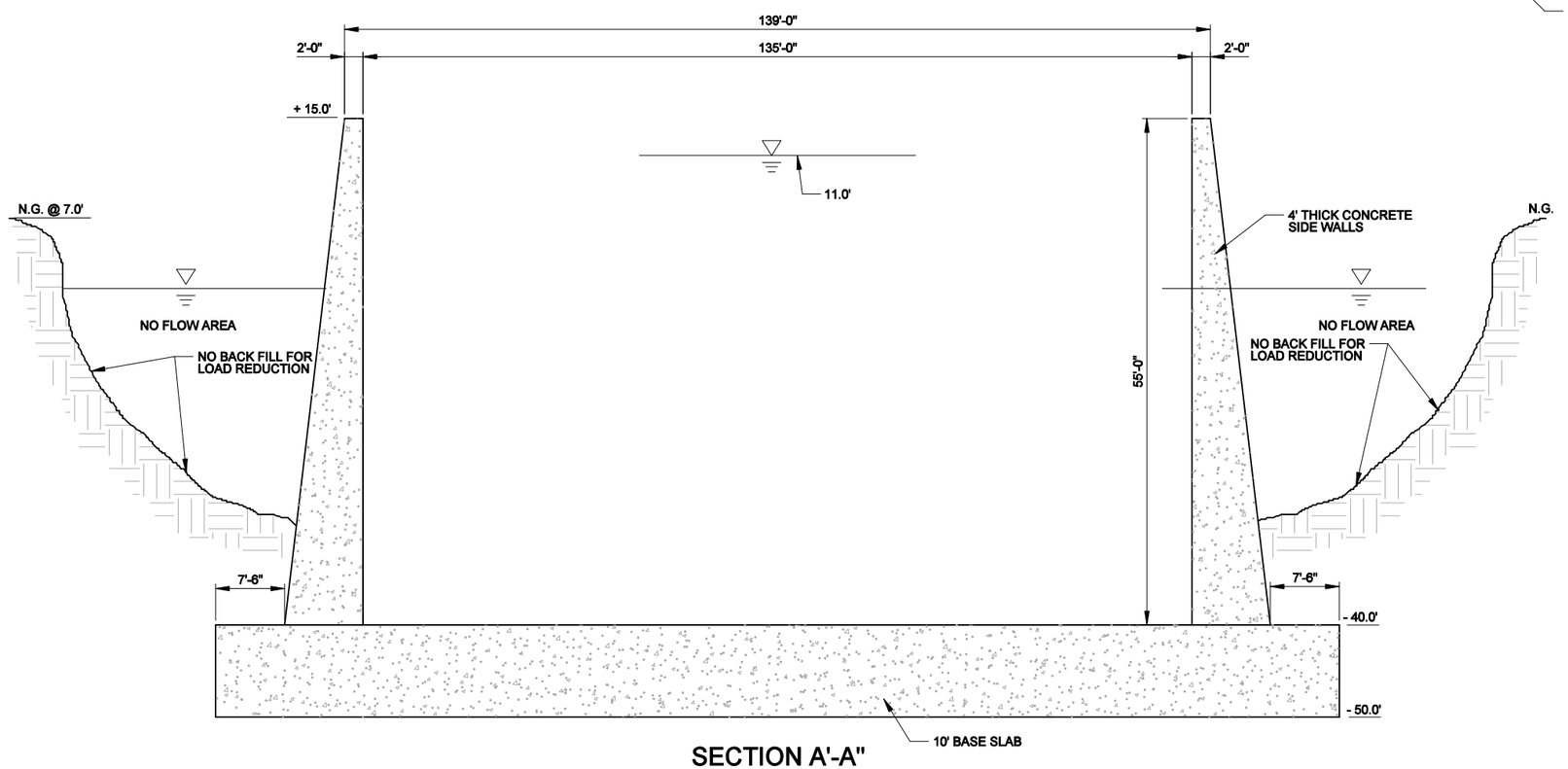
TYPICAL SECTION B'-B"

N.T.S.



TYPICAL SECTION D'-D"

N.T.S.



SECTION A'-A"

135' X 49' DIVERSION CANAL BETWEEN MISSISSIPPI RIVER AND CONTROL STRUCTURE

N.T.S.

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Glemoch
PLOT SCALE: 1:1 ANSI D
PLOT DATE: May 2010
SIZE: ANSI D
FILE NAME: FIG 3 40X45 DIVERSION CANAL

BCG Engineering & Consulting, Inc. Excellence in Engineering

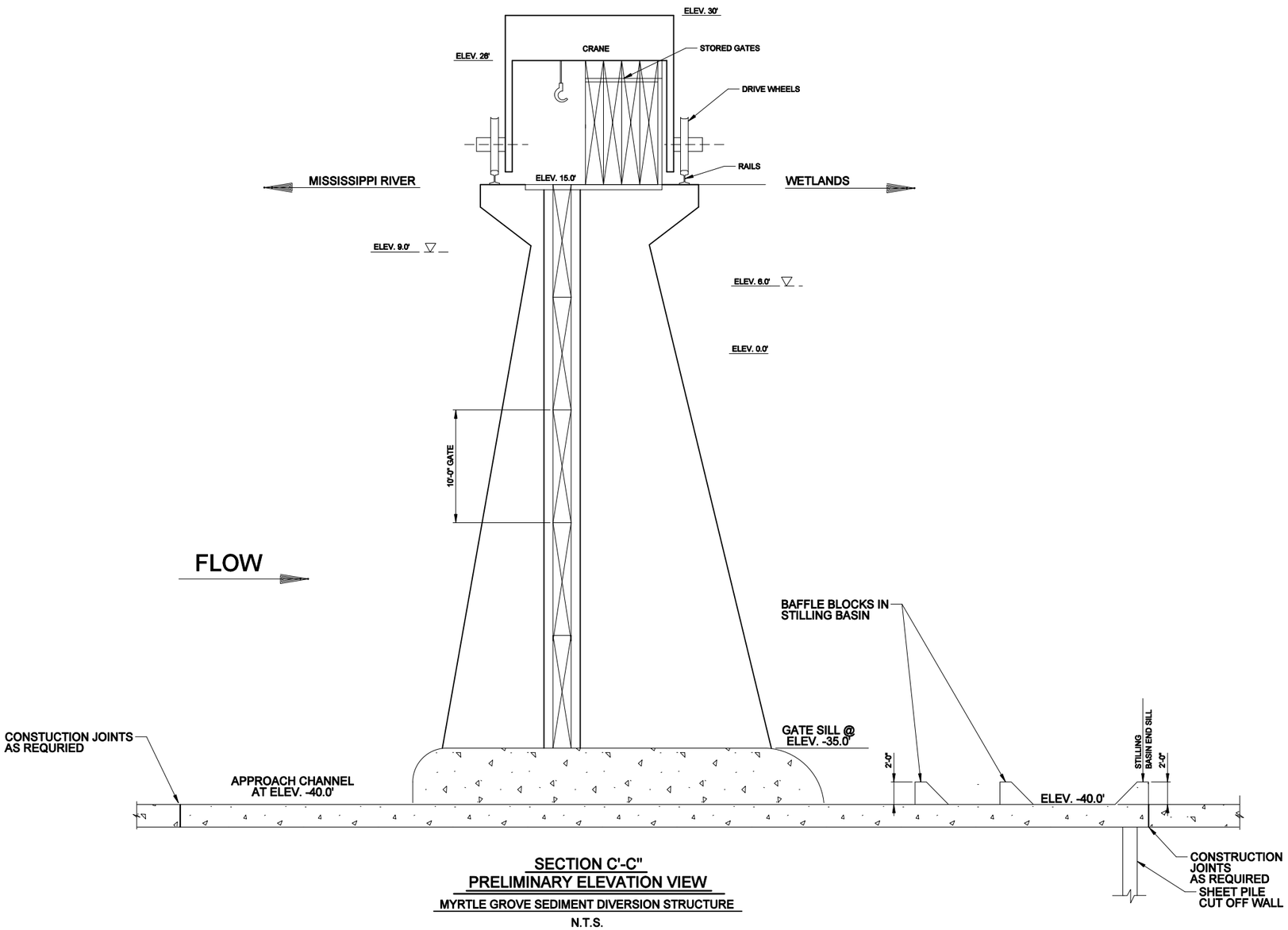
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

75,000 CFS DIVERSION
TYPICAL SECTION
VOLUME IV

SHEET IDENTIFICATION
2E

auto_time

...\FIG 2E_135 X 45 DIVERSION CANAL.dgn 27/10/2011 10:08:26 AM



SECTION C'-C"
PRELIMINARY ELEVATION VIEW
MYRTLE GROVE SEDIMENT DIVERSION STRUCTURE
N.T.S.

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DATE: AUGUST, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Germuth
PLOT SCALE: 1:1 for 11x17 or 1/2 for 22x34
PLOT DATE: August 2010
SIZE: ANSI D
FILE NAME: FIG 4.DGN

BCG Engineering & Consulting, Inc.
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MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

75,000 CFS DIVERSION
TYPICAL SECTION
CONCEPTUAL PLAN - VOLUME IV

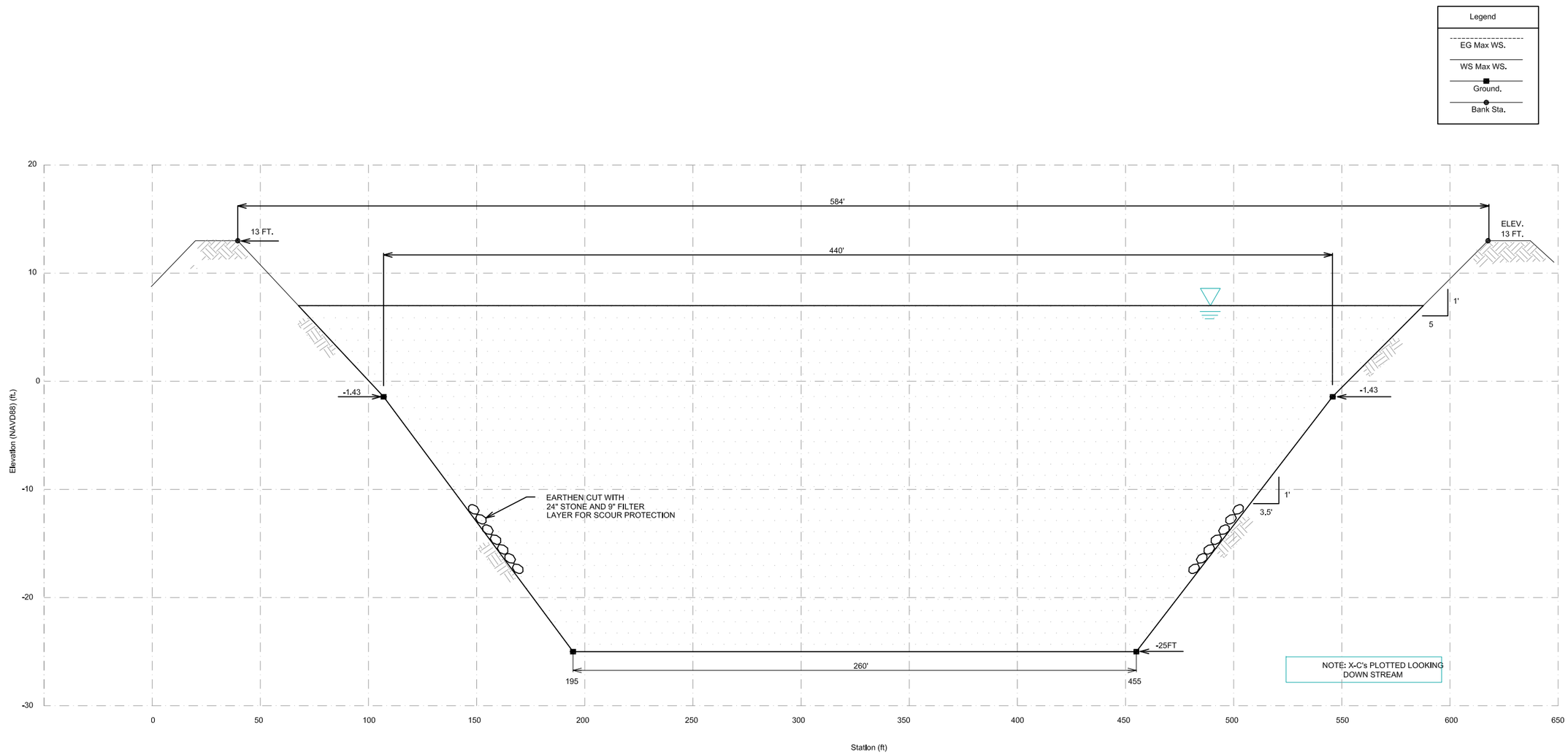
SHEET IDENTIFICATION
3E

auto_time

...\VOLUME 4\FIG 3E.dgn 27/10/2011 10:08:52 AM

RS=231+24

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



TYPICAL SECTION E'-E"

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: CWS
DATE: AUGUST 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Gannuch
PLOT SCALE:
PLOT DATE: August - 2010
SIZE: ANSI D
FILE NAME:

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

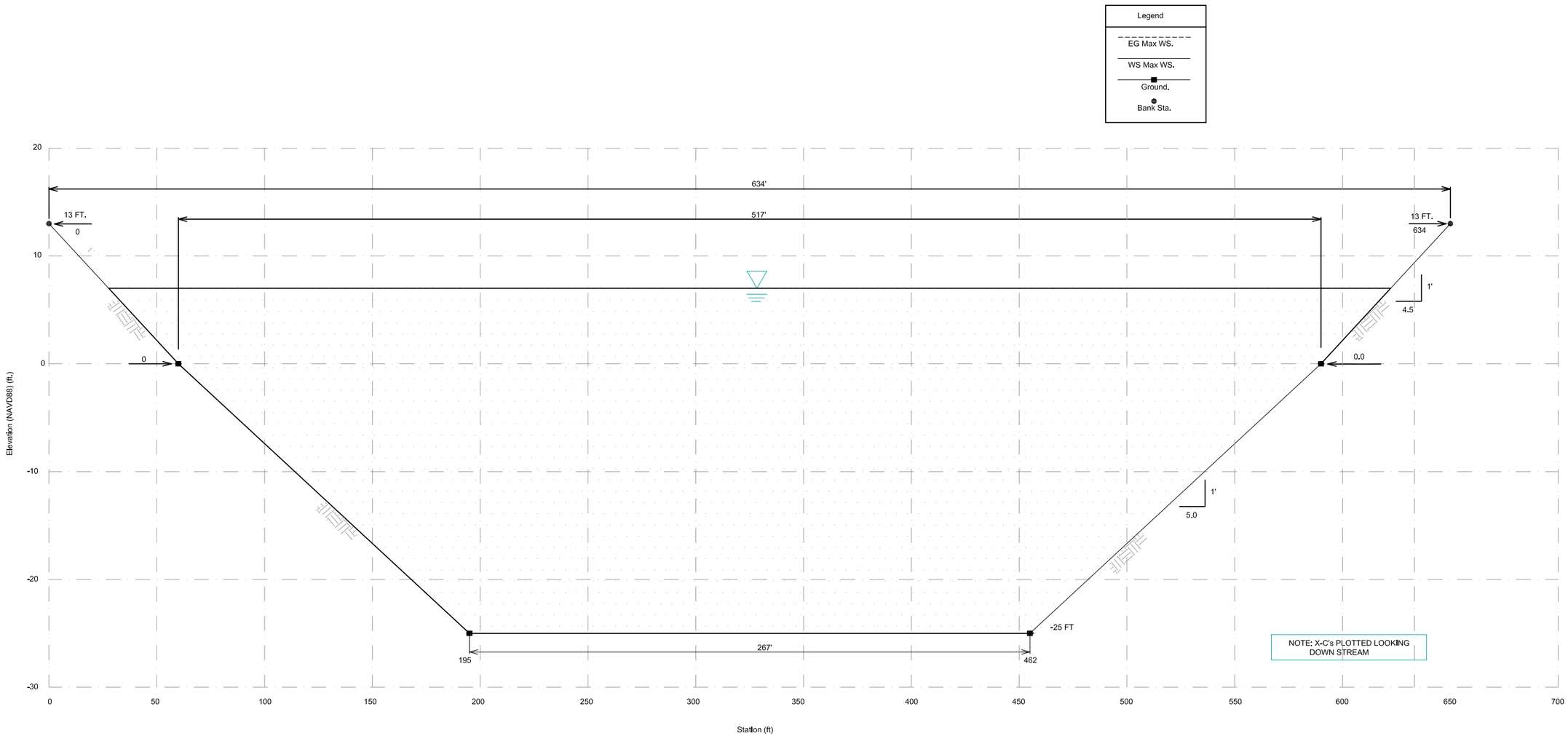
75,000 CFS DIVERSION
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION
CONCEPTUAL PLAN - VOLUME IV

SHEET IDENTIFICATION
4E

auto_time

RS=149.80 BEGIN TRANSITION TO OPEN MARSH

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



TYPICAL SECTION F'-F"

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

DESIGNED BY: CWS
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Gamuch
PLOT SCALE:
PLOT DATE: August 2010
SIZE: ANSI D
FILE NAME:
DATE: AUGUST 2010

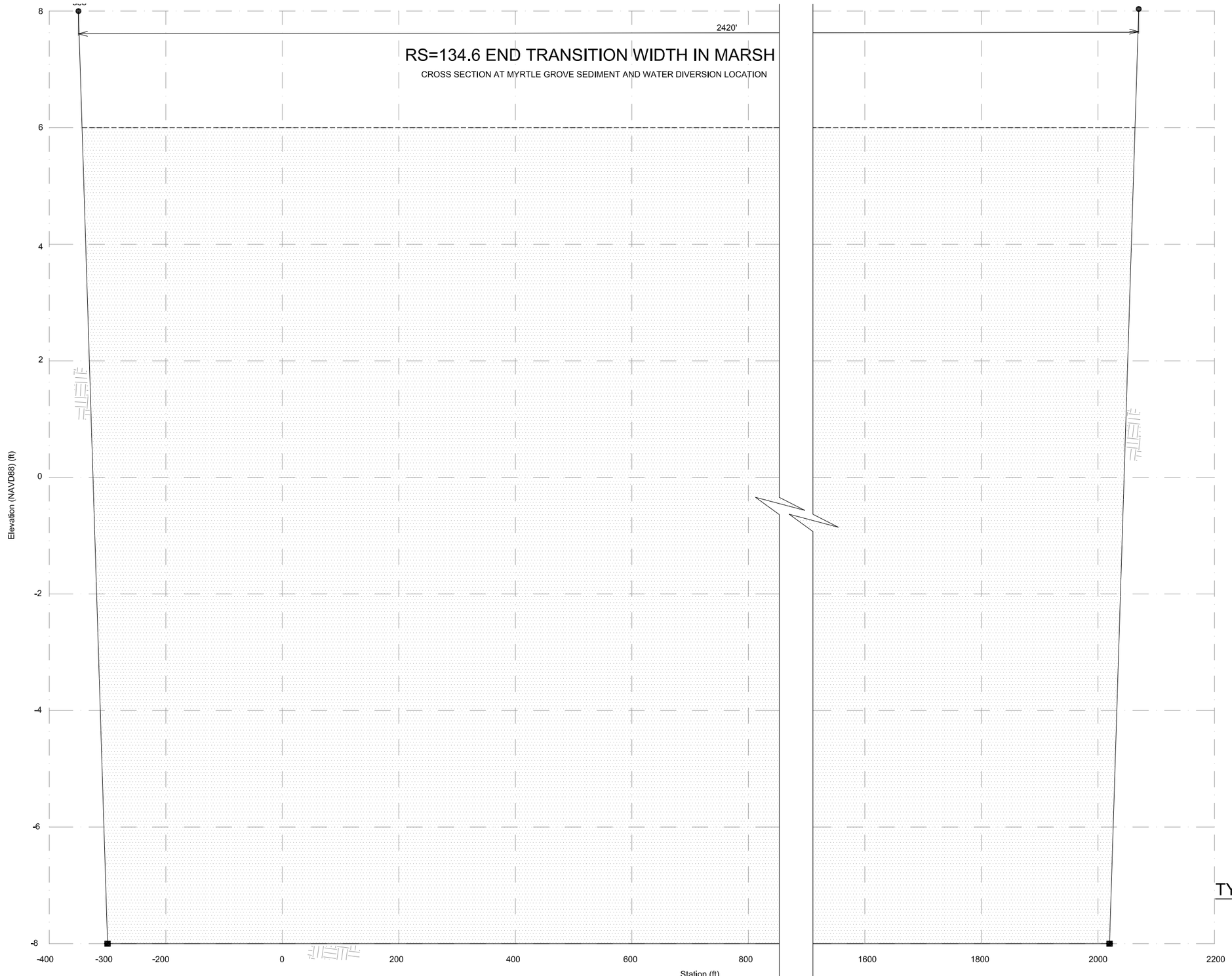
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=149.80 BEGIN TRANSITION TO OPEN MARSH

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION
VOLUME IV

SHEET IDENTIFICATION
5E

outo_time



Legend

EG Max WS.

WS Max WS.

Ground.

Bank Sta.

NOTE: X-C's PLOTTED LOOKING DOWN STREAM

TYPICAL SECTION G'-G'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: CWS
DATE: AUGUST 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Gannuch
PLOT SCALE:
PLOT DATE: August 4, 2010
SIZE: ANSI D
FILE NAME:

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

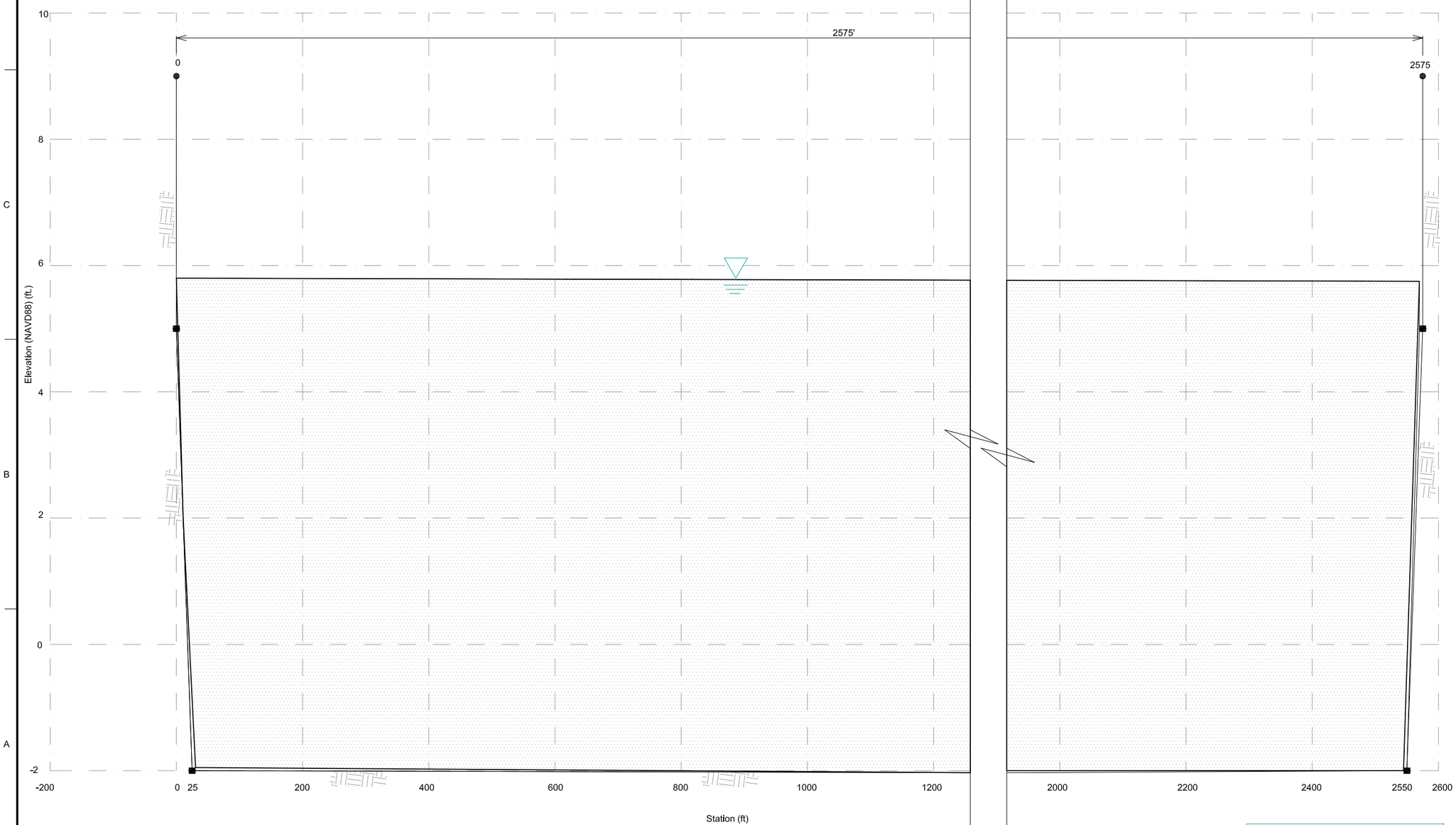
RS=134.6 END TRANSITION IN MARSH
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION
VOLUME IV

SHEET IDENTIFICATION
6E

auto_time

RS=99.80 END TRANSITION IN MARSH

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



TYPICAL SECTION H'-H"

NOTE: X-C's PLOTTED LOOKING DOWN STREAM

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: CWS
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Gamuch
DATE: AUGUST 2010
PLOT SCALE:
PLOT DATE: August 2010
SIZE: ANSI D
FILE NAME:

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

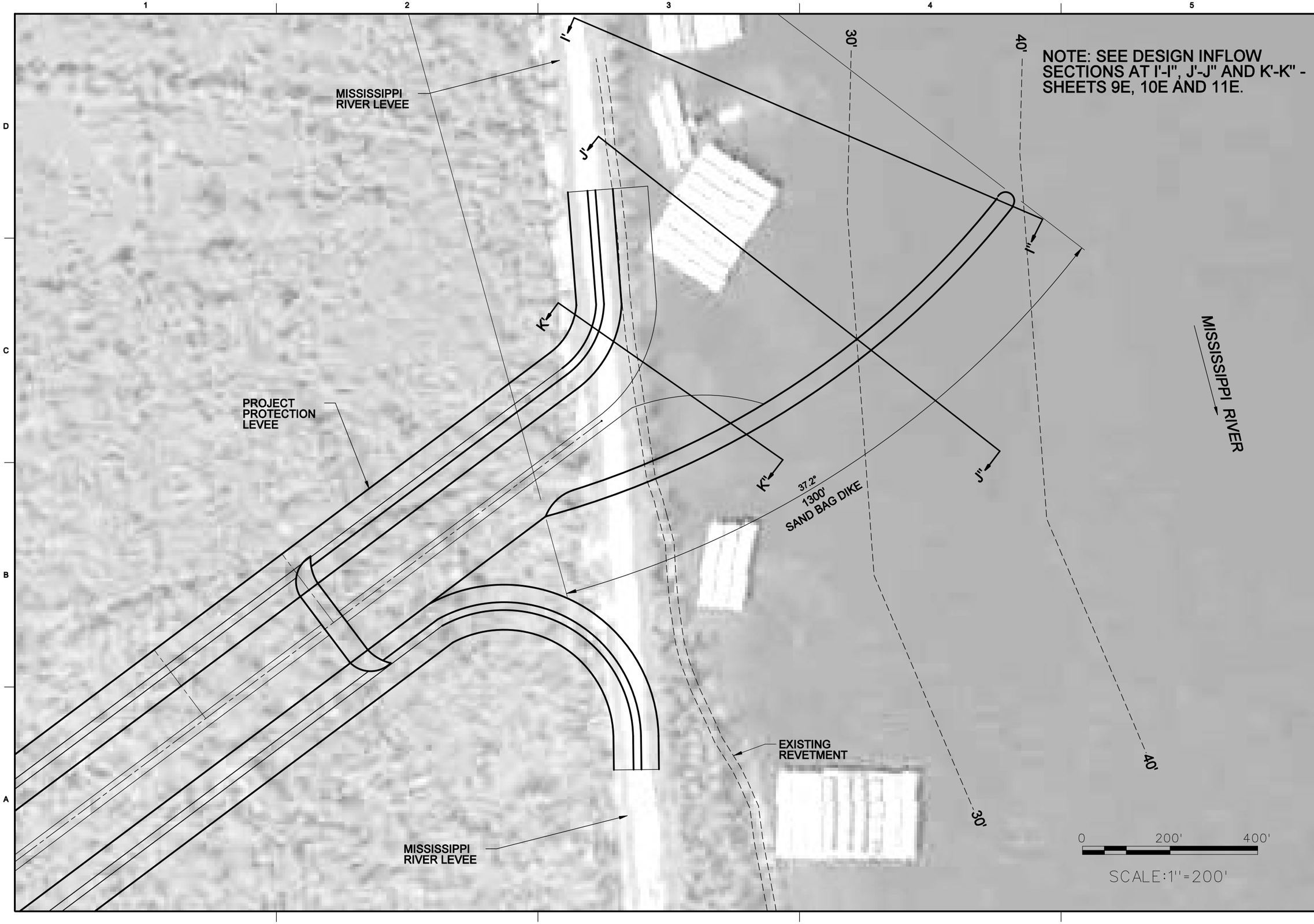
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=99.80 END TRANSITION IN MARSH

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION
VOLUME IV

SHEET IDENTIFICATION
7E

auto_time



MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc. Excellence in Engineering

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Glemuch
PLOT SCALE: 1" = 200'
PLOT DATE: May 2010
SIZE: ANSI D
FILE NAME: C-SB30cut.dgn

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

MYRTLE GROVE OPTION 3 - RIVER TIE-IN DIVERSION CHANNEL

CONCEPTUAL PLAN - VOLUME IV

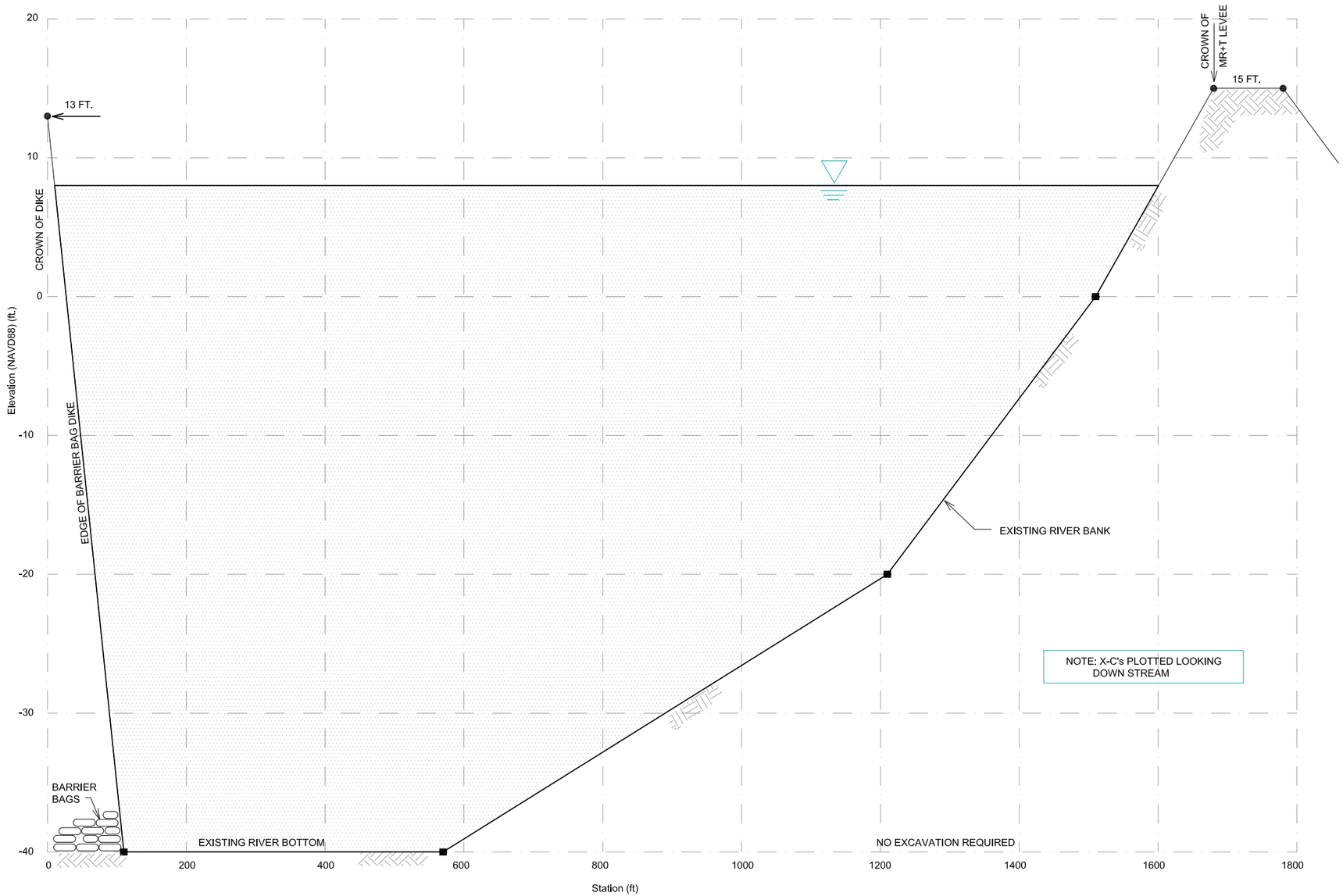
SHEET IDENTIFICATION
8E

auto_time

...\FIG 8E PL OPT 3 blowup.dgn 27/10/2011 10:10:04 AM

RS=289.47 MISSISSIPPI RIVER INFLOW

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



SECTION I'-I"

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: CWS
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Gernuch
PLOT SCALE:
PLOT DATE: August, 2010
SIZE: ANSI D
FILE NAME:
DATE: AUGUST 2010

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=289.47 MISSISSIPPI RIVER IN FLOW

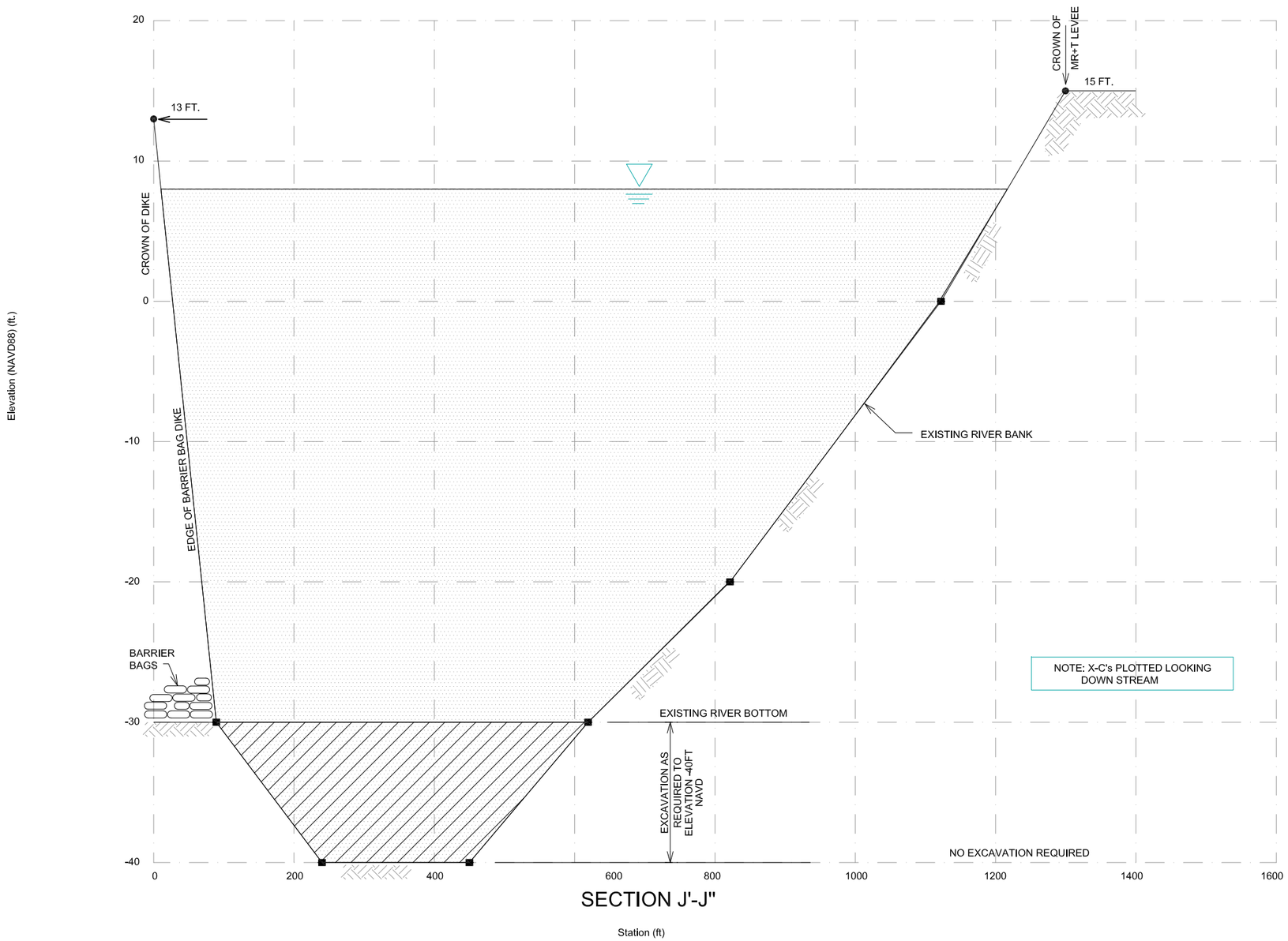
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION
VOLUME IV

SHEET IDENTIFICATION
9E

auto_time

RS=284.14 M.R. INFLOW TO FLUME

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



SECTION J'-J"

Legend

- EG Max WS.
- WS Max WS.
- Ground.
- Bank Sta.

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

DESIGNED BY: CWS
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Gamuch
DATE: AUGUST 2010
PLOT SCALE:
PLOT DATE: August 2010
SIZE: ANSI D
FILE NAME:

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=284.14 MISSISSIPPI RIVER INFLOW TO FLUME

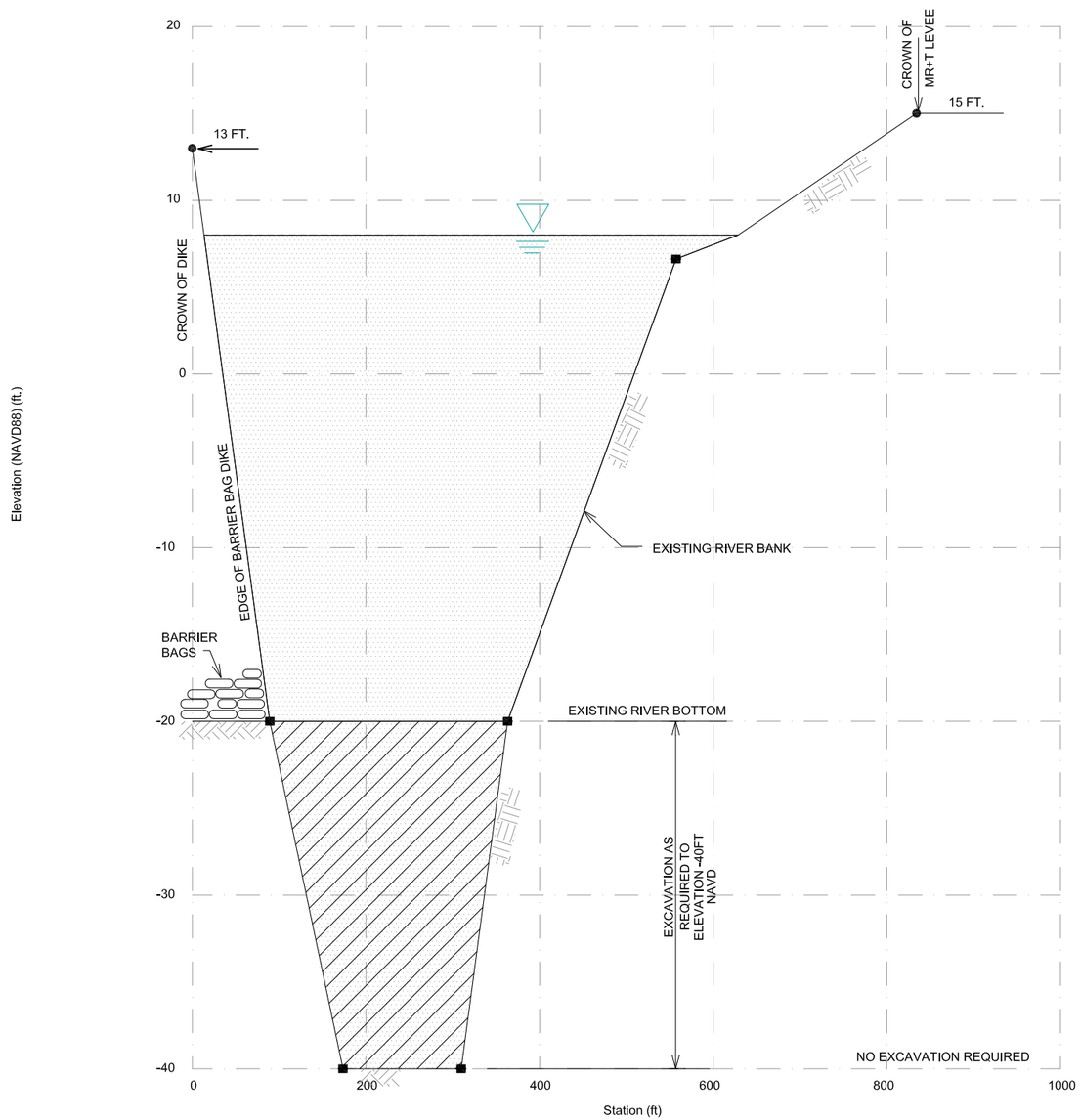
CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION - VOLUME IV

SHEET IDENTIFICATION
10E

auto_time

RS=279.50

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION



SECTION K'-K"

Legend
EG Max WS.
WS Max WS.
Ground.
Bank Sta.

NOTE: X-C's PLOTTED LOOKING DOWN STREAM

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

DESIGNED BY: CWS | DATE: AUGUST 2010
DWN BY: BCG | CKD BY: CWS
SUBMITTED BY: Rodney J. Gamuch
PLOT SCALE: | PLOT DATE: August 25, 2010
SIZE: ANSI D | FILE NAME:

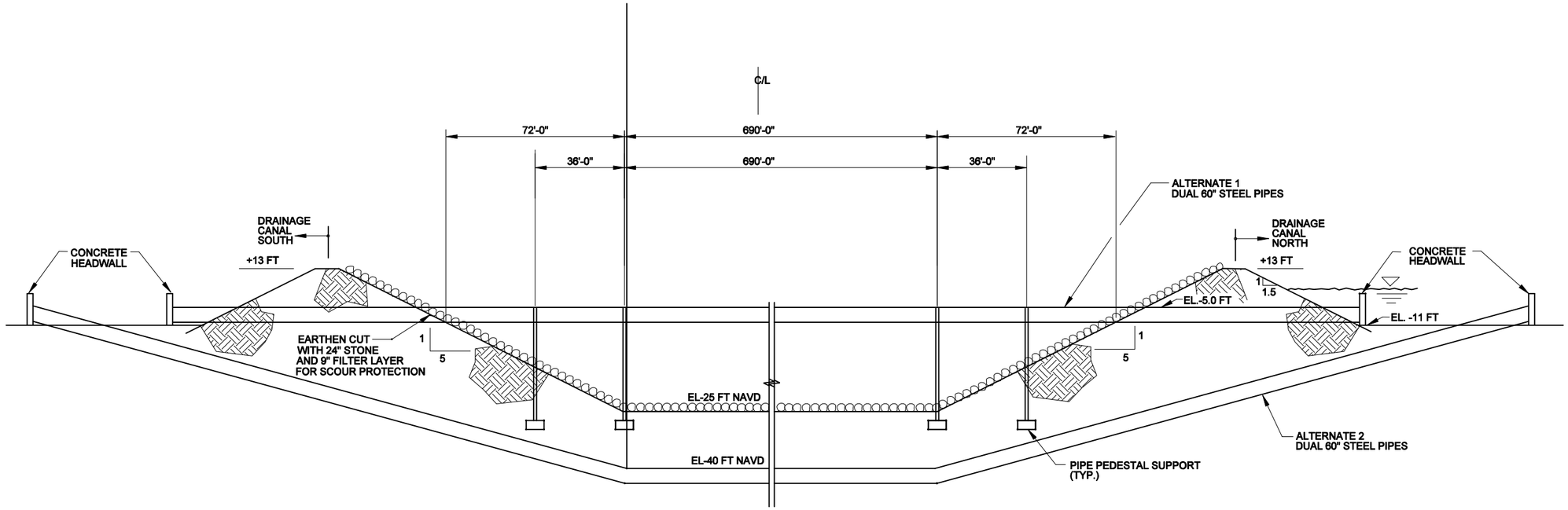
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

RS=279.50

CROSS SECTION AT MYRTLE GROVE SEDIMENT AND WATER DIVERSION LOCATION - VOLUME IV

SHEET IDENTIFICATION
11E

auto_time



TYPICAL INVERTED SIPHON ALTERNATIVES
DRAINAGE CANAL CROSSINGS
MYRTLE GROVE WATER AND SEDIMENT DIVERSION PROJECT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BC3
CKD BY: CWS
SUBMITTED BY: Rodney J. Samuch
PLOT SCALE: 1:1 on 24"x36"
PLOT DATE: Mar 2010
SIZE: ANSI D
FILE NAME: FIG 13_TYP INVERT SIPHON.DGN

BOG Engineering & Consulting, Inc.
Excellence in Engineering

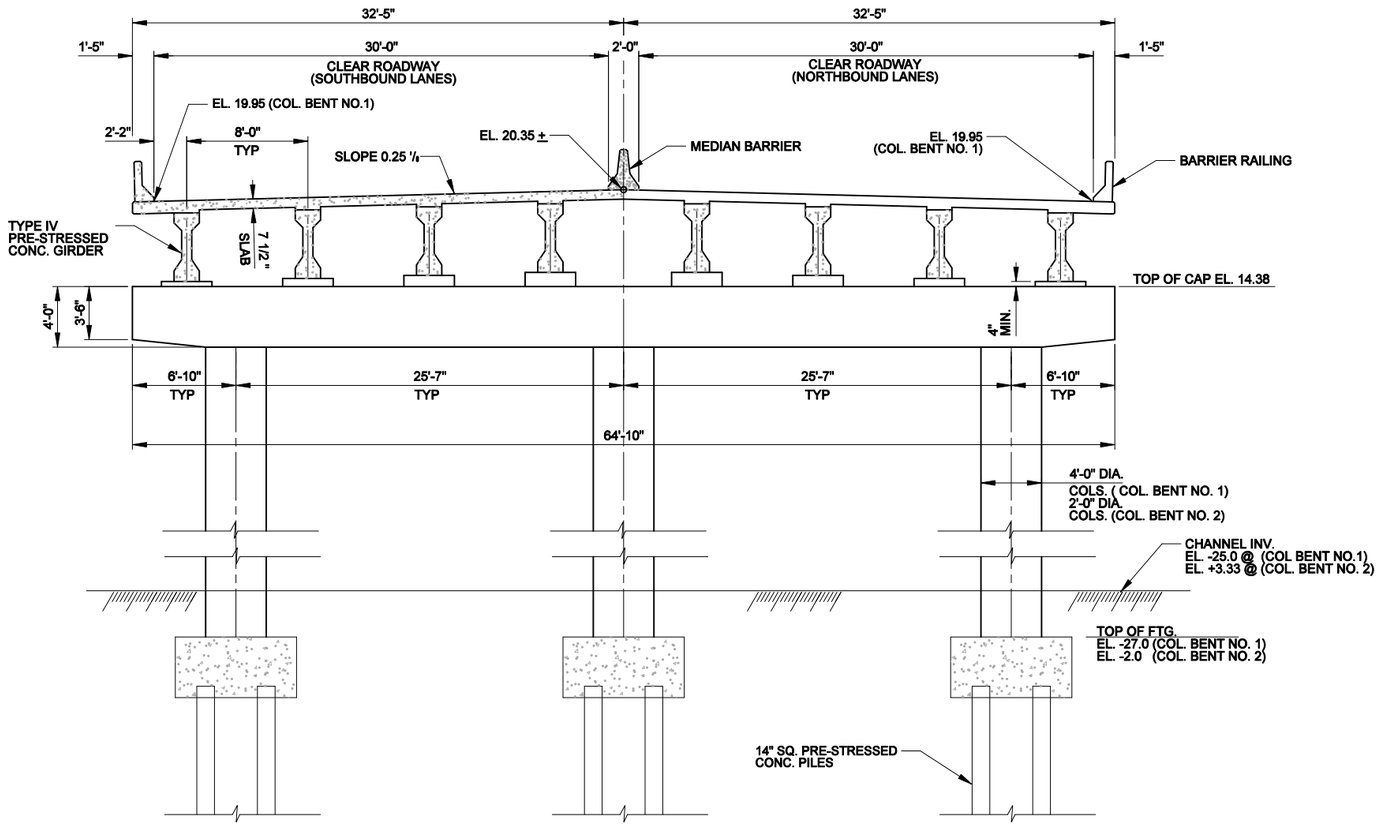
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

TYPICAL SECTIONS
CONCEPTUAL PLAN - VOLUME IV

SHEET IDENTIFICATION
12E

auto_time

...\FIG 12E_TYP. INVERT SIPHON.dgn 27/10/2011 10:10:34 AM

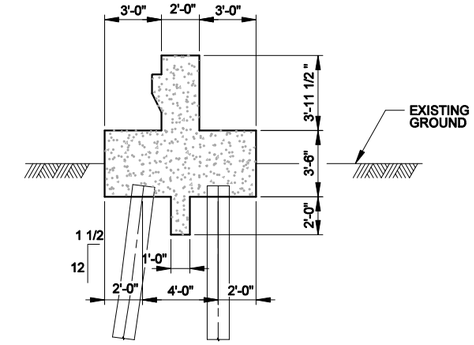


TYPICAL COLUMN BENT

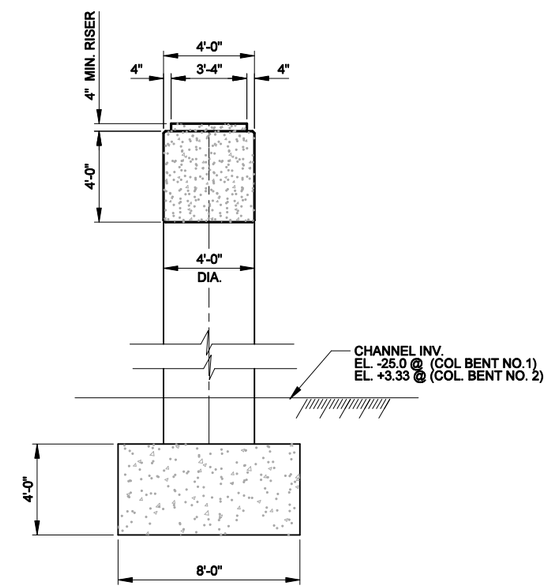
COL. BENT NO. 1, ONE(1) REQUIRED
COL. BENT NO. 2, TWO (2) REQUIRED

SCALE: 1" = 5'-0"

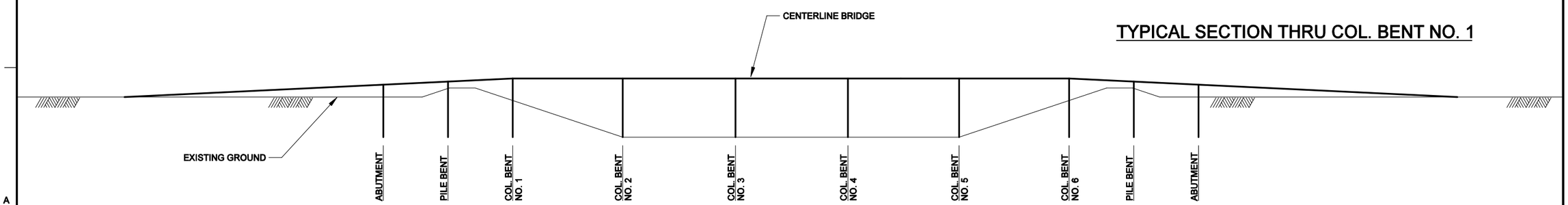
5' 0 5' 10'



TYPICAL ABUTMENT



TYPICAL SECTION THRU COL. BENT NO. 1



TYPICAL PROFILE ALONG C/L BRIDGE

SCALE: 1" = 40'-0"

40' 20' 0 40' 80'

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Samuch
PLOT SCALE: 1:1 = 100'
PLOT DATE: MAY 2010
SIZE: ANSI D
FILE NAME: C-SBDocc.dgn

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

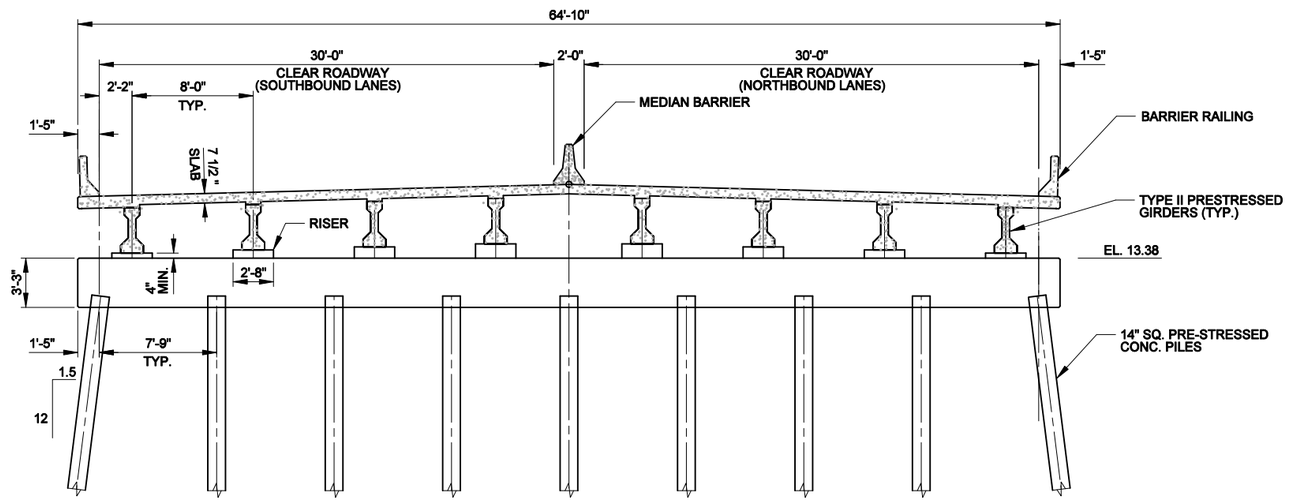
MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

75,000 CFS DIVERSION
CROSS SECTION AND PROFILE
LA. HWY 23 BRIDGE - VOLUME IV

SHEET IDENTIFICATION
13E

auto_time

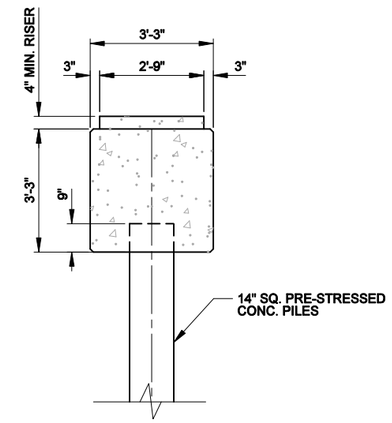
...\FIG 13E bridge section.dgn 27/10/2011 10:10:53 AM



TYPICAL PILE BENT

SCALE: 1" = 5'-0"

5' 0 5' 10'



TYPICAL SECTION THRU PILE BENT

SCALE: ½" = 1'- 0"

12" 0 2' 4' 6' 8'

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: Cecil Soileau
DATE: MAY, 2010
DWN BY: BCG
CKD BY: CWS
SUBMITTED BY: Rodney J. Giemuch
PLOT SCALE: 1" = 100'
PLOT DATE: Mar-2010
SIZE: ANSI D
FILE NAME: G-SB3bcoc.dgn

COASTAL PROTECTION AND RESTORATION AUTHORITY
OFFICE OF COASTAL PROTECTION AND RESTORATION

BCG Engineering & Consulting, Inc.
Excellence in Engineering

MYRTLE GROVE
MISSISSIPPI RIVER SEDIMENT DIVERSION
PLAQUEMINES PARISH, LOUISIANA

75,000 CFS DIVERSION
CROSS SECTION - PILE BENT
LA. HWY. 23 BRIDGE - VOLUME IV

SHEET IDENTIFICATION
14E

auto_time

...\VOLUME 4\FIG 14E PILE BENT.dgn 27/10/2011 10:11:10 AM

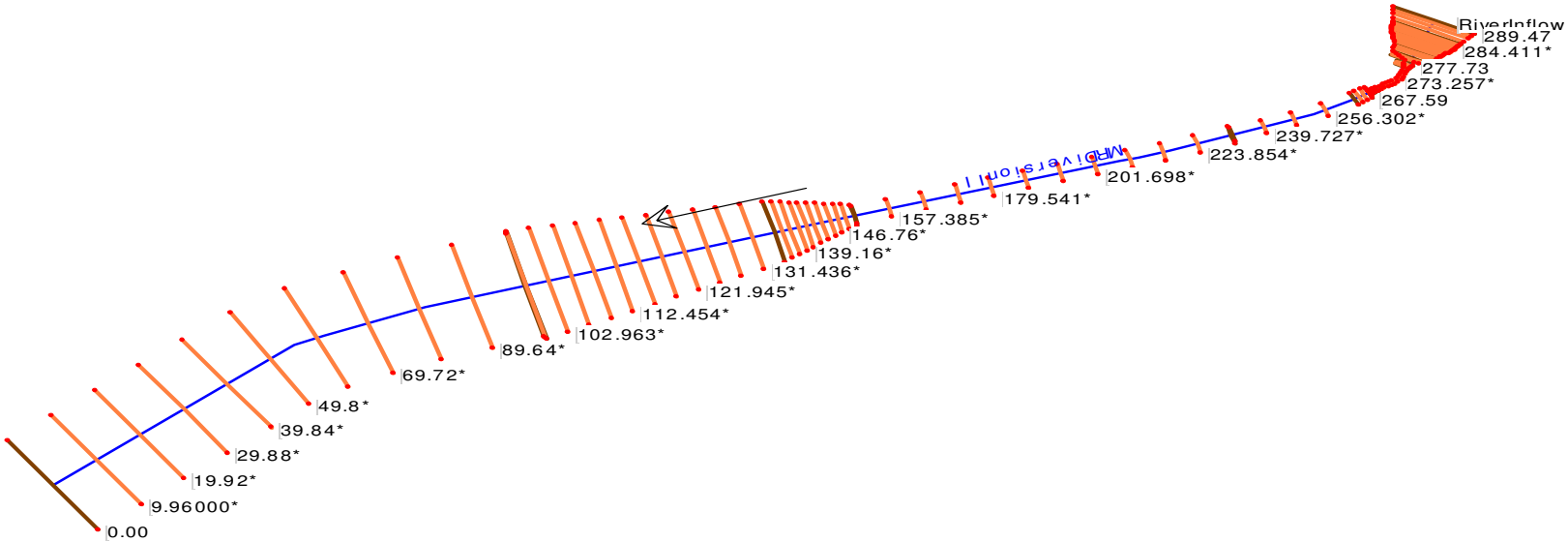
APPENDIX F

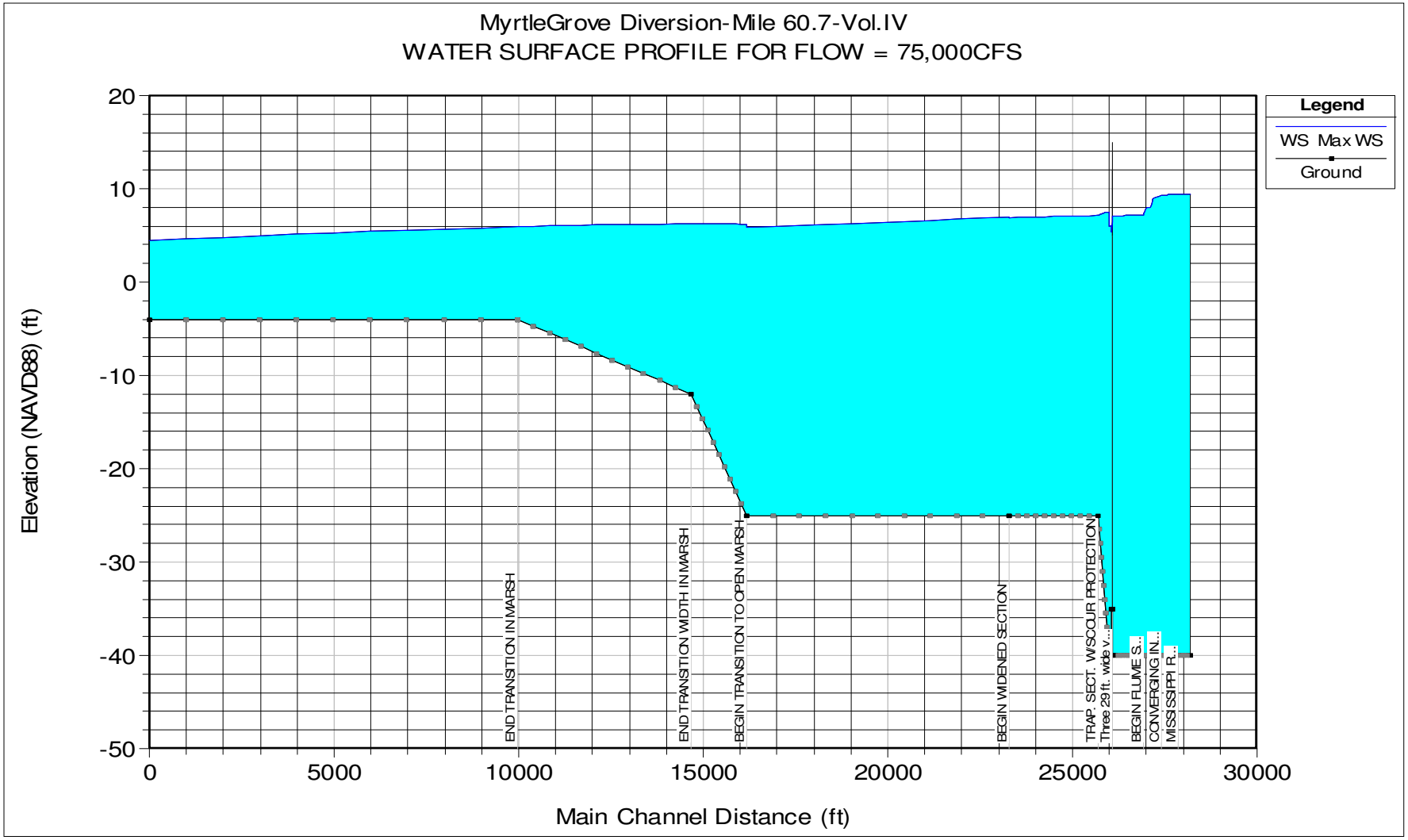
HYDRAULIC MODEL OUTPUT

FOR

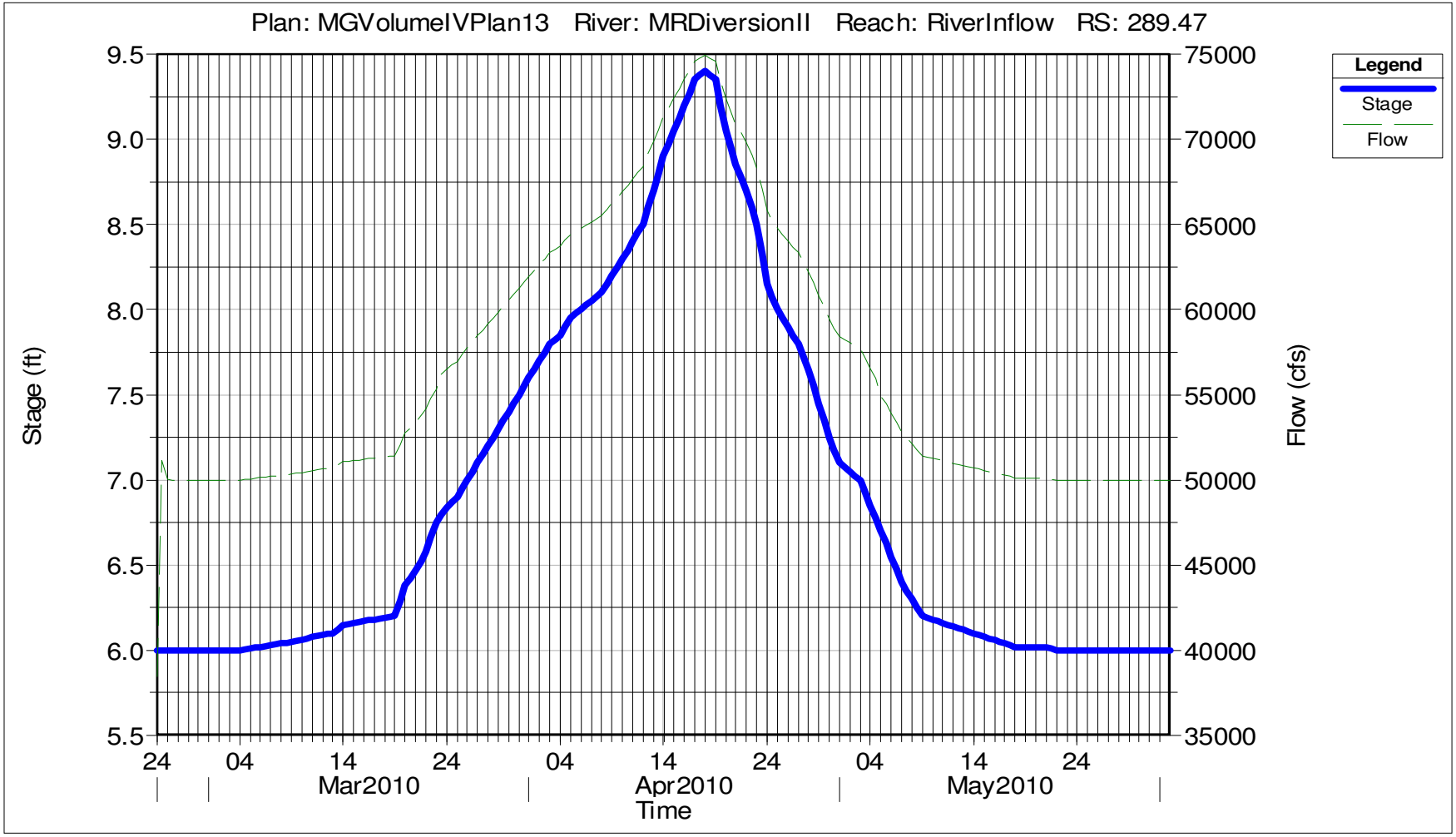
75,000 CFS ALTERNATIVE

AT RIVER MILE 60.7 NEAR MYRTLE GROVE

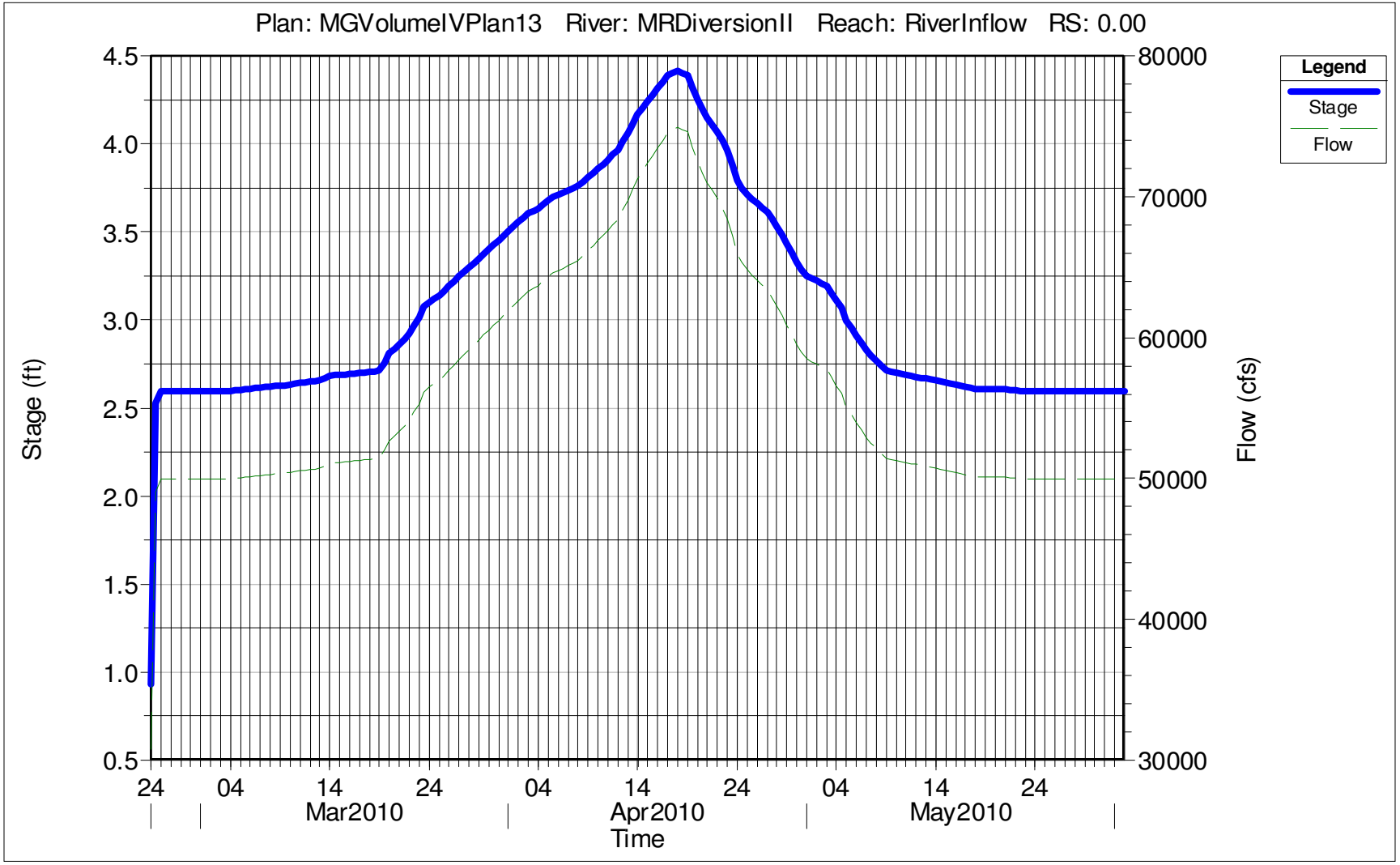

RiverInflow
289.47
284.411*
277.73
273.257*
267.59
256.302*
239.727*
223.854*
201.698*
179.541*
157.385*
146.76*
139.16*
131.436*
121.945*
112.454*
102.963*
89.64*
69.72*
49.8*
39.84*
29.88*
19.92*
9.96000*
0.00



WATER SURFACE AND INVERT PROFILE FOR 75,000CFS PR0JECT PLAN @ MYRTLE GROVE ---- RIVER BOUNDARY AT RIGHT AND BEGINNING OF TRANSITION @16,000 FT



STAGE AND FLOW HYDROGRAPHS AT THE UPSTREAM END OF MODEL IN MISSISSIPPI RIVER WITH PEAK INFLOW CONTROLLED BY GATE SETTINGS



STAGE AND FLOW HYDROGRAPHS AT THE DOWNSTREAM END OF MODEL IN MISSISSIPPI RIVER WITH PEAK INFLOW CONTROLLED BY GATE SETTINGS

TABLE 1 F -- HYDRAULIC DATA FOR 75,000CFS DIVERSION

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)
RiverInflow	289.47	Max WS	74931	-40	9.4	9.4	0.000004	1.4	53836.5	1546.3
RiverInflow	288.612*	Max WS	74931	-40	9.4	9.4	0.000005	1.5	50531.4	1476.8
RiverInflow	287.755*	Max WS	74931	-40	9.4	9.4	0.000006	1.6	47412.5	1407.4
RiverInflow	286.897*	Max WS	74931	-40	9.4	9.4	0.000006	1.7	44474.7	1338
RiverInflow	286.04	Max WS	74931	-40	9.4	9.4	0.000007	1.8	41720.2	1268.7
RiverInflow	285.768*	Max WS	74931	-40	9.4	9.4	0.000008	1.8	40921.2	1252.3
RiverInflow	285.497*	Max WS	74931	-40	9.4	9.4	0.000008	1.9	40123	1235.9
RiverInflow	285.225*	Max WS	74931	-40	9.4	9.4	0.000009	1.9	39327.3	1219.6
RiverInflow	284.954*	Max WS	74931	-40	9.4	9.4	0.000009	1.9	38535.7	1203.2
RiverInflow	284.682*	Max WS	74931	-40	9.4	9.4	0.000009	2	37743.6	1186.9
RiverInflow	284.411*	Max WS	74931	-40	9.4	9.4	0.00001	2	36953.8	1170.5
RiverInflow	284.14	Max WS	74931	-40	9.4	9.4	0.000011	2.1	36167.1	1154.1
RiverInflow	283.832*	Max WS	74931	-40	9.4	9.4	0.000012	2.2	34381.5	1114.4
RiverInflow	283.525*	Max WS	74931	-40	9.3	9.4	0.000013	2.3	32648.1	1074.6
RiverInflow	283.217*	Max WS	74931	-40	9.3	9.4	0.000015	2.4	30960.6	1035
RiverInflow	282.91*	Max WS	74931	-40	9.3	9.4	0.000017	2.6	29321.3	995.4
RiverInflow	282.602*	Max WS	74931	-40	9.3	9.4	0.00002	2.7	27732.5	955.8
RiverInflow	282.295*	Max WS	74931	-40	9.3	9.4	0.000023	2.9	26190.8	916.3
RiverInflow	281.987*	Max WS	74931	-40	9.3	9.4	0.000026	3	24695.9	876.9
RiverInflow	281.68	Max WS	74931	-40	9.3	9.4	0.00003	3.2	23252.1	837.5
RiverInflow	280.953*	Max WS	74931	-40	9.2	9.4	0.000046	3.8	19903	776.7
RiverInflow	280.226*	Max WS	74931	-40	9.1	9.4	0.00007	4.5	16787.6	699.9
RiverInflow	279.5	Max WS	74931	-40	9	9.4	0.000095	5.3	14053.7	556.8
RiverInflow	279.262*	Max WS	74931	-40	8.9	9.4	0.000125	5.9	12767.7	537.8
RiverInflow	279.025*	Max WS	74931	-40	8.7	9.4	0.000158	6.8	11034.4	438.3
RiverInflow	278.787*	Max WS	74931	-40	8.2	9.4	0.000236	8.7	8613.2	306.1
RiverInflow	278.55	Max WS	74931	-40	8	9.4	0.000218	9.4	7943.3	198.5
RiverInflow	278.386*	Max WS	74931	-40	8	9.4	0.000214	9.4	7967.5	205.8
RiverInflow	278.222*	Max WS	74931	-40	8	9.4	0.000212	9.4	7984.7	211.3
RiverInflow	278.058*	Max WS	74931	-40	8	9.4	0.00021	9.4	7994.4	215.1
RiverInflow	277.894*	Max WS	74931	-40	8	9.4	0.000211	9.4	7996.2	217.6
RiverInflow	277.73	Max WS	74931	-40	8	9.4	0.000212	9.4	7989.6	218.7
RiverInflow	277.53*	Max WS	74931	-40	7.9	9.4	0.000227	9.7	7766.8	211
RiverInflow	277.33*	Max WS	74931	-40	7.8	9.4	0.000245	10	7515.2	198.2
RiverInflow	277.13*	Max WS	74931	-40	7.7	9.4	0.000271	10.4	7226.6	183.4
RiverInflow	276.93	Max WS	74931	-40	7.5	9.4	0.00031	10.9	6898.5	167
RiverInflow	276.83*	Max WS	74931	-40	7.4	9.4	0.000239	11.3	6615.2	144.3
RiverInflow	276.73	Max WS	74931	-40	7.2	9.4	0.000167	11.8	6376.4	135.1
RiverInflow	276.035*	Max WS	74931	-40	7.2	9.4	0.000167	11.8	6374.4	135.1
RiverInflow	275.340*	Max WS	74931	-40	7.2	9.3	0.000167	11.8	6372.3	135.1
RiverInflow	274.646*	Max WS	74931	-40	7.2	9.3	0.000168	11.8	6370.6	135.1
RiverInflow	273.951*	Max WS	74931	-40	7.2	9.3	0.000168	11.8	6368.6	135.1
RiverInflow	273.257*	Max WS	74931	-40	7.2	9.3	0.000168	11.8	6366.6	135.1
RiverInflow	272.562*	Max WS	74931	-40	7.1	9.3	0.000168	11.8	6364.5	135.1

RiverInflow	271.868*	Max WS	74931	-40	7.1	9.3	0.000168	11.8	6362.5	135.1
RiverInflow	271.173*	Max WS	74931	-40	7.1	9.3	0.000168	11.8	6360.5	135.1
RiverInflow	270.479*	Max WS	74931	-40	7.1	9.3	0.000168	11.8	6358.7	135.1
RiverInflow	269.784*	Max WS	74931	-40	7.1	9.2	0.000169	11.8	6356.7	135.1
RiverInflow	269.09	Max WS	74931	-40	7.1	9.2	0.000169	11.8	6354.6	135.1
RiverInflow	268.89	Max WS	74931	-40	7.1	9.2	0.000169	11.8	6354.1	135.1
RiverInflow	268.44	Max WS	74931	-40	7.1	9.2	0.000169	11.8	6353	135.1
RiverInflow	268.39	Max WS	74931	-35	6.4	9.2	0.000241	13.4	5593.7	135.1
RiverInflow	268.29		Inl Struct							
RiverInflow	268.19	Max WS	74931	-35	5.3	8.3	0.00026	13.8	5444.8	135.1
RiverInflow	268.14	Max WS	74931	-40	6	8.3	0.00018	12.1	6211.1	135.1
RiverInflow	267.79	Max WS	74931	-40	6	8.3	0.00018	12.1	6210	135.1
RiverInflow	267.59	Max WS	74931	-40	7.5	7.8	0.000024	4.2	17871.3	471.1
RiverInflow	267.29*	Max WS	74931	-38.5	7.5	7.8	0.000027	4.3	17251.3	470.4
RiverInflow	266.99*	Max WS	74931	-37	7.5	7.8	0.00003	4.5	16632.1	469.6
RiverInflow	266.69*	Max WS	74931	-35.5	7.5	7.8	0.000034	4.7	16013.7	468.8
RiverInflow	266.39*	Max WS	74931	-34	7.4	7.8	0.000039	4.9	15395.8	467.9
RiverInflow	266.09*	Max WS	74931	-32.5	7.4	7.8	0.000044	5.1	14778.2	466.9
RiverInflow	265.79*	Max WS	74931	-31	7.4	7.8	0.00005	5.3	14160.6	465.8
RiverInflow	265.49*	Max WS	74931	-29.5	7.3	7.8	0.000058	5.5	13542.7	464.5
RiverInflow	265.19*	Max WS	74931	-28	7.3	7.8	0.000067	5.8	12924.1	463.2
RiverInflow	264.89*	Max WS	74931	-26.5	7.2	7.8	0.000079	6.1	12304.1	461.6
RiverInflow	264.59	Max WS	74931	-25	7.1	7.8	0.000093	6.4	11682.2	459.9
RiverInflow	261.275*	Max WS	74931	-25	7.1	7.8	0.000093	6.4	11671.4	459.7
RiverInflow	257.96*	Max WS	74931	-25	7.1	7.7	0.000093	6.4	11660.6	459.6
RiverInflow	254.645*	Max WS	74931	-25	7.1	7.7	0.000094	6.4	11649.8	459.4
RiverInflow	251.33*	Max WS	74930	-25	7.1	7.7	0.000094	6.4	11639	459.3
RiverInflow	248.015*	Max WS	74930	-25	7	7.7	0.000094	6.4	11628.1	459.2
RiverInflow	244.7*	Max WS	74930	-25	7	7.6	0.000094	6.4	11617.2	459
RiverInflow	241.385*	Max WS	74930	-25	7	7.6	0.000095	6.5	11606.3	458.9
RiverInflow	238.07*	Max WS	74930	-25	7	7.6	0.000095	6.5	11595.4	458.7
RiverInflow	234.755*	Max WS	74930	-25	6.9	7.6	0.000095	6.5	11584.4	458.6
RiverInflow	231.44	Max WS	74930	-25	6.9	7.6	0.000095	6.5	11573.5	458.4
RiverInflow	231.24	Max WS	74930	-25	6.9	7.6	0.000168	6.3	11968.7	507.4
RiverInflow	223.116*	Max WS	74930	-25	6.8	7.4	0.000167	6.2	12010	511.3
RiverInflow	214.992*	Max WS	74930	-25	6.7	7.3	0.000167	6.2	12052.9	515.1
RiverInflow	206.868*	Max WS	74930	-25	6.6	7.2	0.000166	6.2	12097.1	519
RiverInflow	198.744*	Max WS	74930	-25	6.5	7.1	0.000166	6.2	12143.1	522.8
RiverInflow	190.62*	Max WS	74930	-25	6.4	7	0.000165	6.1	12190.6	526.7
RiverInflow	182.496*	Max WS	74930	-25	6.3	6.8	0.000164	6.1	12239.7	530.5
RiverInflow	174.372*	Max WS	74929	-25	6.2	6.7	0.000164	6.1	12290.5	534.3
RiverInflow	166.248*	Max WS	74929	-25	6	6.6	0.000163	6.1	12343.7	538.1
RiverInflow	158.124*	Max WS	74929	-25	5.9	6.5	0.000162	6	12397.5	541.9
RiverInflow	150	Max WS	74929	-25	5.8	6.4	0.000161	6	12453.4	545.7
RiverInflow	149.8	Max WS	74929	-25	6.1	6.4	0.000114	3.8	19530.3	786.7
RiverInflow	148.28*	Max WS	74929	-23.7	6.2	6.4	0.000079	3.2	23302	928.1
RiverInflow	146.76*	Max WS	74929	-22.4	6.2	6.3	0.000061	2.8	26663.3	1069.2

RiverInflow	145.24*	Max WS	74929	-21.1	6.2	6.3	0.00005	2.5	29622.5	1210.2
RiverInflow	143.72*	Max WS	74929	-19.8	6.2	6.3	0.000044	2.3	32179.3	1350.9
RiverInflow	142.2*	Max WS	74929	-18.5	6.2	6.3	0.000041	2.2	34336.9	1491.5
RiverInflow	140.68*	Max WS	74929	-17.2	6.2	6.3	0.000039	2.1	36093.4	1631.9
RiverInflow	139.16*	Max WS	74929	-15.9	6.2	6.3	0.000038	2	37451.3	1772.2
RiverInflow	137.64*	Max WS	74929	-14.6	6.2	6.3	0.000039	2	38408.2	1912.2
RiverInflow	136.12*	Max WS	74929	-13.3	6.2	6.3	0.000041	1.9	38965.3	2051.9
RiverInflow	134.6	Max WS	74928	-12	6.2	6.3	0.000044	1.9	39120.8	2191.2
RiverInflow	131.436*	Max WS	74928	-11.3	6.2	6.3	0.000045	1.9	39834.2	2321.5
RiverInflow	128.272*	Max WS	74928	-10.6	6.2	6.3	0.000046	1.9	40371.4	2451.8
RiverInflow	125.109*	Max WS	74928	-9.8	6.2	6.2	0.000048	1.8	40684.3	2582.1
RiverInflow	121.945*	Max WS	74927	-9.1	6.2	6.2	0.000051	1.8	40794.5	2712.3
RiverInflow	118.781*	Max WS	74927	-8.4	6.1	6.2	0.000054	1.8	40700.3	2842.4
RiverInflow	115.618*	Max WS	74927	-7.6	6.1	6.2	0.000059	1.9	40429.7	2972.4
RiverInflow	112.454*	Max WS	74926	-6.9	6.1	6.1	0.000065	1.9	39921.7	3102.3
RiverInflow	109.290*	Max WS	74926	-6.2	6.1	6.1	0.000073	1.9	39201.1	3231.6
RiverInflow	106.127*	Max WS	74925	-5.5	6	6.1	0.000083	2	38263	3360.9
RiverInflow	102.963*	Max WS	74925	-4.7	6	6	0.000097	2	37135.9	3490.2
RiverInflow	99.8	Max WS	74925	-4	5.9	6	0.000115	2.1	35741.1	3619.5
RiverInflow	99.75*	Max WS	74925	-4	5.9	6	0.000115	2.1	35739	3619.5
RiverInflow	99.7*	Max WS	74925	-4	5.9	6	0.000115	2.1	35736.9	3619.5
RiverInflow	99.65*	Max WS	74925	-4	5.9	6	0.000115	2.1	35734.7	3619.5
RiverInflow	99.6	Max WS	74925	-4	5.9	6	0.000115	2.1	35732.6	3619.5
RiverInflow	89.64*	Max WS	74924	-4	5.8	5.9	0.00012	2.1	35302.4	3619.4
RiverInflow	79.68*	Max WS	74923	-4	5.7	5.8	0.000125	2.1	34853.9	3619.3
RiverInflow	69.72*	Max WS	74922	-4	5.5	5.6	0.000131	2.2	34385	3619.3
RiverInflow	59.76*	Max WS	74921	-4	5.4	5.5	0.000138	2.2	33893.2	3619.2
RiverInflow	49.8*	Max WS	74920	-4	5.3	5.3	0.000145	2.2	33376	3619.1
RiverInflow	39.84*	Max WS	74919	-4	5.1	5.2	0.000153	2.3	32830.6	3619.1
RiverInflow	29.88*	Max WS	74918	-4	5	5	0.000162	2.3	32252.7	3618.8
RiverInflow	19.92*	Max WS	74917	-4	4.8	4.9	0.000173	2.4	31637.1	3618.1
RiverInflow	9.96000*	Max WS	74916	-4	4.6	4.7	0.000185	2.4	30978.3	3617.3
RiverInflow	0	Max WS	74915	-4	4.4	4.5	0.0002	2.5	30268.9	3616.5

APPENDIX D

Water and Sediment Surveys of the Mississippi River Channel Conducted at Myrtle Grove and Magnolia in Support of Numerical Modeling (October 2008-May 2010)

A report to the State of Louisiana
Office of Coastal Protection and Restoration
as subcontracted to C.H. Fenstermaker and Associates,

by

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1.0 DESIGN OF SURVEYS

Five main field surveys were carried out to capture a range of Mississippi River water discharges—October 2008, April 2009, May 2009, April 2010 and May 2010. In addition, a Laser In-Situ Scattering and Transmissometry (LISST) instrument was placed on the USGS Belle Chasse platform in August 2009 and operated through January 2010. The five boat surveys were carried out on the University of Texas Institute of Geophysics (UTIG) 22' vessel R/V *Lake Itasca* (Fig. 1): more information on the *Itasca* is available at http://www.ig.utexas.edu/jsg/mgg/courses/geof391/itasca.pdf. The vessel operated out of Empire, LA during each survey.

Figure 1. R/V Lake Itasca conducting a river survey. The multibeam bathymetric pole is deployed at midships, and the ADCP on the bow.

Data were collected only above the bend at Myrtle Grove in October 2008, and at Myrtle Grove and Magnolia sites in 2009-2010. Data were collected under the following river discharge conditions:

Table 1. Survey Date/Methods and Discharge at Tarbert Landing, MS

Cruise	Date	Site**	Tarbert Discharge (m^3/sec)*	Activity[a]
October 2008	10th	MGup	9,968	Basemap, P-63, bedload, adcp
October 2008	11th	MGup	9,854	Basemap, CTD/LISST, grab, bedload
April 2009	4th	MAG	19,567	Bedload, adcp, CTD/LISST
April 2009	5th	MAG,MGdown	19,737	Bedload, P-63, adcp, basemap
April 2009	6th	No collection	20,020	Due to weather

April 2009	7th	MGdown, MGup	20,331	Bedload, CTD/LISST, adcp, P-63, basemap
April 2009	8th	MAG, MGdown, MGbend	20,643	Grab, bedload, CTD/LISST
April 2009	9th	MGbend	20,954	P-63, adcp
May 2009	1st	MAG,MGdown,MGbend	22,512	Adcp, grab
May 2009	2nd	MGup, MGdown,MAG	22,370	Bedload, adcp, CTD/LISST
May 2009	3rd	MGup, MGdown, MAG	22,200	Bedload, CTD/LISST
May 2009	4th	MAG, MGup, MGbend	22,200	P-63, CTD/LISST
May 2009	5th	No collection	22,342	Boat engine failure
May 2009	6th	MGup, MGbend, MGdown	22,710	Grab, P-63
May 2009	7th	MGdown, MGbend	23,220	Grab, adcp
April 2010	12th	MAG	25,598	Basemap
April 2010	13th	MAG, MGup	25,485	ADCP, basemap, CTD
April 2010	14th	MAG, MGup, MGbend, MGdown	24,098	ADCP, bedload, CTD/LISST
April 2010	15th	MAG, MGup, MGdown	23,956	P-6, bedload, basemap
April 2010	16th	MGup, MGbend, MGdown	23,616	ADCP, P-6, bedload
May 2010	11th	MGup, MAG	20,671	ADCP, P-6, bedload, CTD/LISST
May 2010	12th	MAG, MGup	21,096	ADCP, basemap, bedload, grab
May 2010	13th	MGup, MAG	21,606	Basemap, bedload, ADCP, grab
May 2010	14th	MGup, MAG	22,115	Bedload, basemap, grab, ADCP
May 2010	15th	MAG, MGup	22,596	P-6, ADCP, bedload, CTD/LISST

* Water discharge daily averages at Tarbert Landing, MS were obtained at http://www.mvn.usace.army.mil/cgi-bin/wcmanual.pl?01100

**MGup = Myrtle Grove survey area above the bend, MGbend = Myrtle Grove survey area within the bend, MGdown = Myrtle Grove survey area below (downriver) of the bend, MAG = Magnolia

[a]Basemap = multibeam basemap mapping, P-63 = isokinetic water sampling with P-63 point integrative sampler, bedload = multibeam repeat bathymetric mapping for bedload transport measurements, grab = bottom grab sampling, adcp = acoustic Doppler current profiler cross-sections of water discharge and backscatter (suspended sediment)

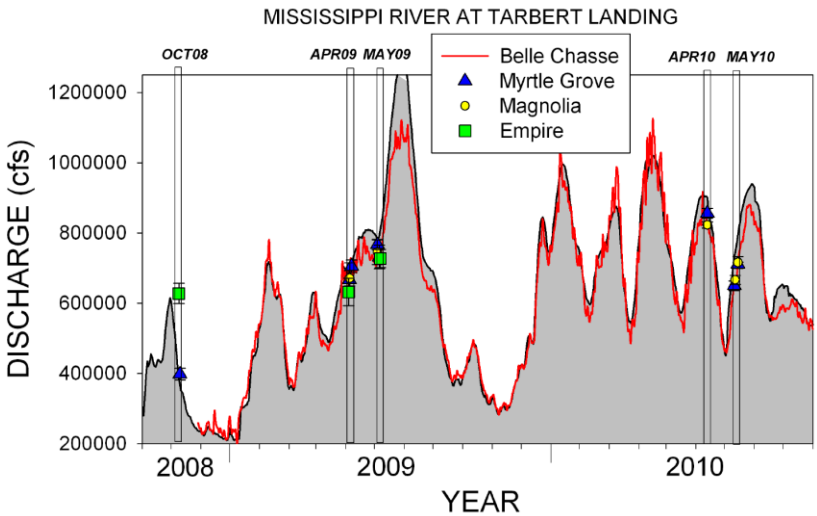

Figure 2. River discharge at Tarbert Landing, MS (black line with gray fill) with the times of the five Myrtle Grove-Magnolia surveys in 2008-10 superimposed. Also plotted is the discharge at the USGS Belle Chasse gage (H-ADCP; red line) and data collected at Empire on the 2008-9 surveys.

2.0 MULTIBEAM DATA COLLECTION AND POST-PROCESSING

Multibeam bathymetric surveys were collected for two purposes 1) to build a bathymetric basemap of the channel floor in the Myrtle Grove and Magnolia regions to provide a detail grid for 3-D numerical modeling and 2) to calculate bedload transport rates at several sites at near Myrtle Grove and at Magnolia. Details of the bedload survey methods and calculations are described in section 3.0 below. Basemap data at Myrtle Grove was mainly collected during the October 2008 surveys, but was added to in April 2009 and May 2010. The Magnolia basemap was collected in April 2010. A pole-mounted (see Fig. 1) Reson Seabat 8101 swath bathymetry profiler (101 transducers) with side scan projector and Reson 6042 software were used for multibeam data collection on the R/V *Lake Itasca*. The data in 2010 was collected with the system newly upgraded to a Reson 7101 processing computer that expanded to 512 beams and equal angle projection (equal beam density in the across-track direction). Attitude data (heave, pitch, roll, yaw) were collected with a gyroscope inertial guidance system (Applanix, Inc.), mounted inside the vessel hull. Dual antenna differential GPS provided position, heading, and velocity data, which were integrated with measurements of ship attitude using Applanix POS/MV hardware. Within the system software, error limits were manually set, and notification (warning) lights appeared if attitude, heading, position, and velocity exceeded the input threshold (0.05°, 0.1°, 2 m, 0.5 m/sec, respectively). Data were not collected if these limits were exceeded. Typical values, however, were generally much lower than the input thresholds (common survey values: ~0.025°, 0.018°, 0.3 m, 0.04 m/sec). Position and attitude data were collected at 1 ms intervals, and tagged to bathymetry measurements and stored in Universal Transverse Mercator (UTM) zone 16N horizontal datum. Vertical resolution for the multibeam instrument, as estimated by the manufacturer, is ~1.5 cm.

After data collection, raw files (generated in proprietary .db Reson format) were converted to industry standard XTF format and then imported into CARIS© HIPS 6.1 software for post-processing. This post-processing includes removal of multiples and other noise and navigation drop-outs. Data were then corrected for elevation of the water surface (combined river stage and tidal phase, using hourly USACE water level gage data, referenced to NGVD sea level at the Alliance gage at river mile 62.5 (see

http://www2.mvr.usace.army.mil/WaterControl/shefdata2.cfm?sid=01390&d=7&dt=S) for Myrtle Grove data, and the West Pointe a la Hache gage at river mile 48.3 (see http://www2.mvr.usace.army.mil/WaterControl/stationinfo2.cfm?sid=01400&fid=&dt=S) for the Magnolia data. Given that these datasets were collected above the salt wedge, sound velocity correction was applied at a standard 1500 m/sec for freshwater. Motion sensor and navigation data were then merged and channel bathymetry grids of 1 x 1 m cell size and 5 x 5 m cell size were created in HIPS for final export. These data are provided as electronic file supplements to this report and the extent of the Myrtle Grove basemap is shown in Figure 3 and Magnolia basemap in Figure 4.

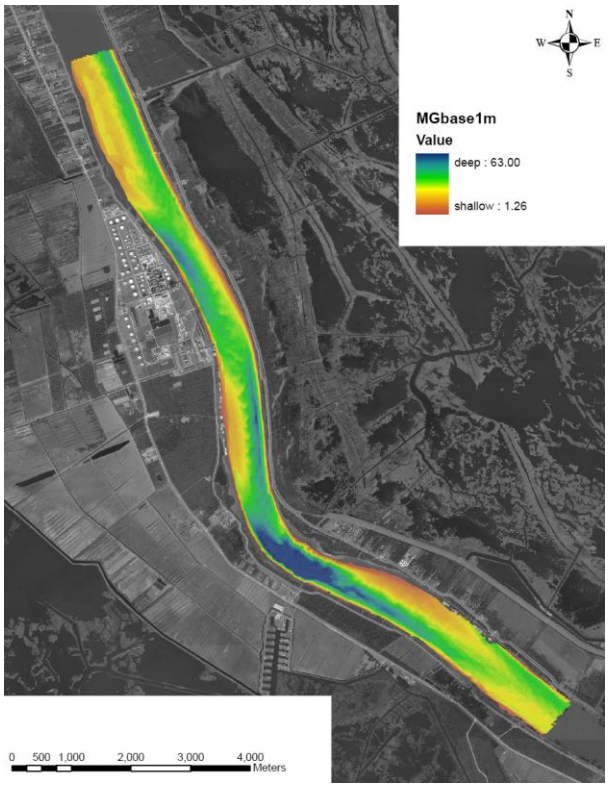

Figure 3. Limits of the multibeam bathymetric basemap collected at Myrtle Grove in 2008-10.

3.0 BEDLOAD (BEDFORM) SEDIMENT TRANSPORT RATES

Bedload surveys were conducted using a modified version of the survey and post-processing methods developed in Nittrouer et al. (2008). Surveys were collected at MGup (above the Myrtle Grove bend) in October 2008, April and May 2009, and April and May 2010, at MGdown (below the bend) in April and May 2009 and April 2010, and at Magnolia in April and May 2009 and April 2010. The data collected at Magnolia in 2010 was adjusted downstream about 500m to place it upon the center of the bar (basemap done in April 2010) and centered over the ADCP/cast cross-sectional line: it is referred to in Table 2 as MAGb to distinguish its different location. Bedload data were not collected at MGbend (in the bend itself) because this area does not contain bedforms due to the high energy that transitions sand to suspension. Each survey consisted of two bathymetry data sets of a ~500 m long river reach (see Fig. 5), collected within 24 h of one another. The exception was May 2010 at MGup, where repeat surveys were collected on five consecutive days (grids A-E in Table 2). Along-channel, bank-parallel transects were run between the concrete revetments that protect the base of the artificial levee (5 m water depth). Bathymetry transects were conducted with at least 30% swath overlap (swath width is proportional to water depth by a ratio of 7.5:1), and were oriented in the same ship travel direction to minimize bathymetry error from variable ship squat (angle of the vessel's keel relative to the water line) and heave (that varies with wave direction). The repeat surveys followed similar line order and direction as the initial.

Initial post-processing and export of grids followed the methods outlined in section 2.0 above. Following this step, data were exported to ArcView 9.2© software, where data manipulations for calculating bedload transport from downstream migration of bedforms were made. A raster grid

was constructed using an inverse distance weighting interpolation function (1 m cell size). Repeat surveys were then compared; visual identification of crest locations confirmed that dunes had migrated less than one wavelength between surveys (a prerequisite of the survey). Raster gridding of the multibeam data revealed bathymetry-matching irregularities, visible as sharp changes in apparent bathymetry situated along the outer beams where swaths overlapped. These inconsistencies result from fewer (scattered) data in the far range, which make interpolation of a consistent grid difficult. Unlike earlier methods applied in Nittrouer et al. (2008),

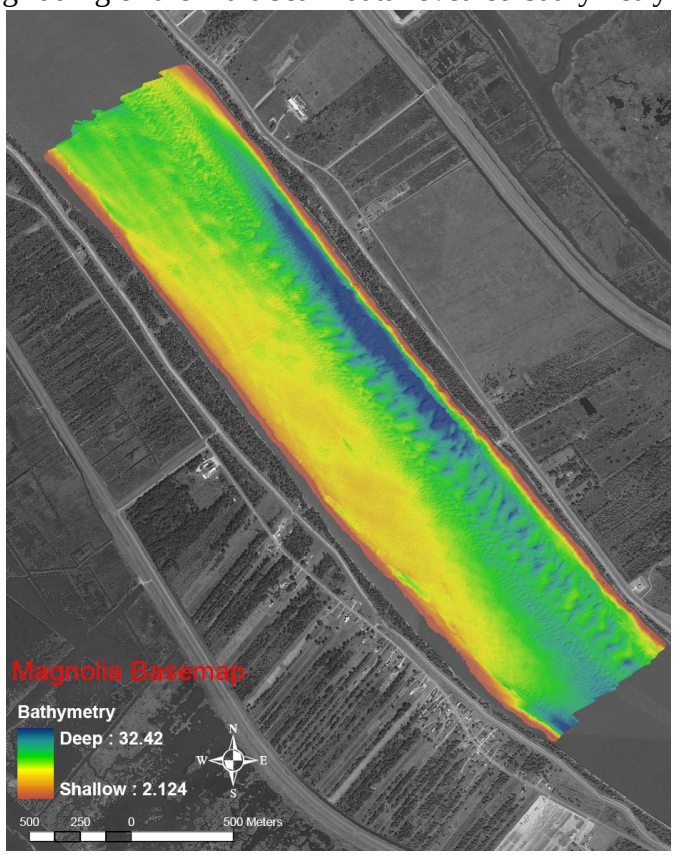

Figure 4. Limits of the multibeam bathymetric basemap collected at Myrtle Grove in 2010.

the effect of these swath-matching anomalies was minimized by focusing upon a reduced area of the mapped bedform field downriver (see Fig. 5). A set of bedform field polygons was outlined, corresponding to areas of similar bedform wavelength (inherent in the final calculation. These varied in downriver width from between survey sites and cruises due to the effect mentioned above, and in river width as the bedform field changed dimensions. As can be seen in a comparison in Figure 5 at MGdown, the bedform field expanded in width between the April and May 2009 and the number of individual bedform wavelength polygon areas changed from 2 to 3. In the three cruises, typical number of cross-sectional polygons of the bedform field width of the river ranged from 2-5. Areas of no bedforms were assumed to have a bedload transport rate of 0 (see Nittrouer et al., 2008).

Volume of sand deposited for each polygon were calculated assuming a sediment porosity of 35% and a grain density of 2,650 kg/m^3. These values were converted to final bed material flux measurements (q_s in m^2/sec) for each cross-sectional polygon, and were then summed cross-channel and converted using sediment grain density to derive a final bedform translational flux (Q_s in mass per unit time) for that river cross-section. See Nittrouer et al. (2008) for a more thorough explanation of the equations utilized.

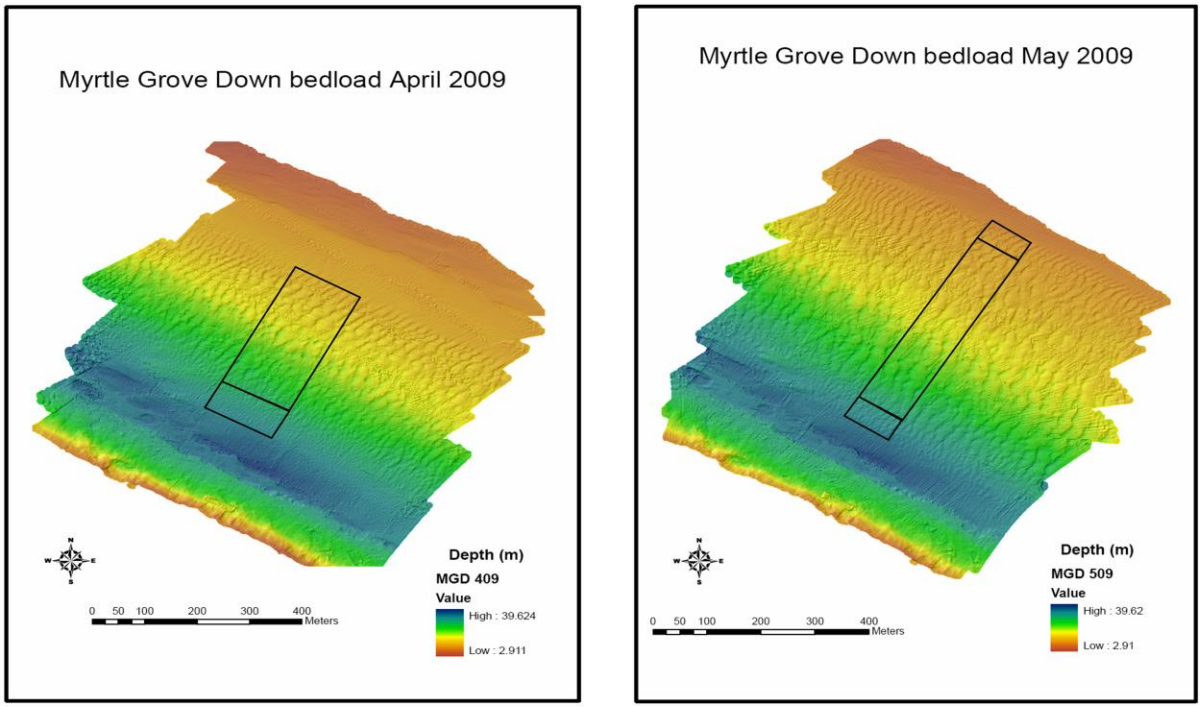

Figure 5. Bedload survey grids at the Myrtle Grove site below the bend. This is one (of two) surveys each from the April and May 2009 cruises showing the limits of the final sub-grids of individual areas of similar bedform wavelength used to make the bedform flux calculations.

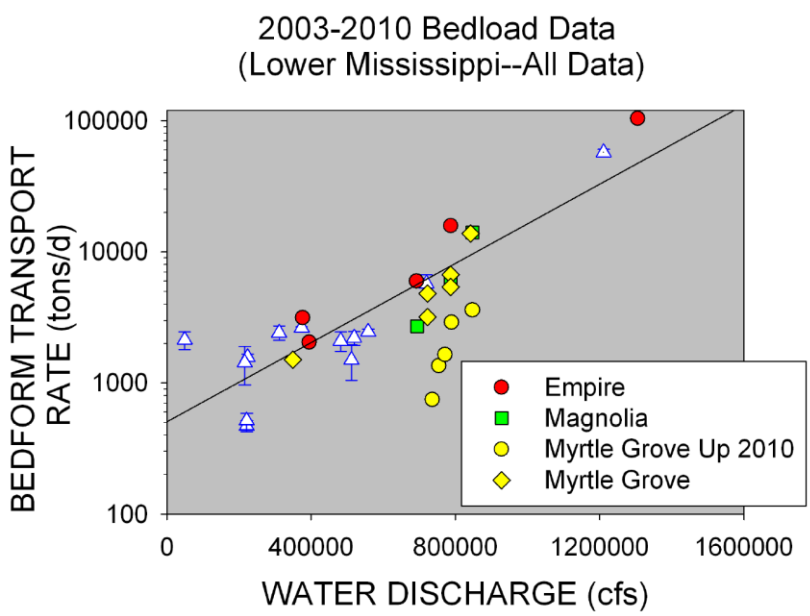

Figure 6. Bedform transport rates listed in Table 2 for the Myrtle Grove and Magnolia data relative to data published for 2003-2006 (blue triangles) in the lower river in Nittrouer et al. (2008). Also plotted are data collected at Empire (River mile 23.3) in 2008 and 2009 for another project (CWPPRA Scofield Island Restoration) by the author. Myrtle Grove data are separated into two categories: the "Myrtle Grove Up 2010" category is only those data collected in the bar above the Myrtle Grove bend in April-May 2010 that were potentially affected by the bar dredging upriver at Alliance. The "Myrtle Grove" category is the data collected at Myrtle Grove Up and Down in 2008-9 and the Myrtle Grove Down data in April 2010. The best fit regression line (r2=0.78) excludes the data from the "Myrtle Grove Up 2010" surveys.

Table 2. Bedload (bedform) mass flux rates at Myrtle Grove and Magnolia in 2008-10

Survey Grid*	Study Dates/hours**	River Mile***	Polygon# and Channel Width (m)	Polygon flux per unit width (q_s in m^2/sec)	Channel mass flux (Q_s) in tons/day[a]
MGup (2008)	2103-2154 Oct. 10 2042-2130 Oct. 11	61.6	P1(157) P2(291)	2.89E-05 7.10E-06	1511.4
MAG (2009)	1453-1628 April 4 1511-1711 April 5	46.6	P1(210) P2(285)	4.27E-05 1.03E-05	2718.2^
MGdown (2009)	1558-1715 April 7 1617-1743 April 8	58.0	P1(54) P2(322)	9.97E-06 4.14E-05	3190.7
MGup (2009)	1957-2053 April 7 1935-2041 April 8	61.6	P1(48) P2(65) P3(171) P4(137)	2.02E-05 2.80E-05 9.76E-05 1.15E-05	4822.4
MGup (2009)	1514-1619 May 2 1451-1544 May 3	61.6	P1(181) P2(118) P3(130)	1.17E-04 5.66E-05 1.04E-05	6683.5
MGdown (2009)	1730-1832 May 2 1602-1701 May 3	58.0	P1(159) P2(272) P3(64)	2.31E-05 7.16E-05 8.08E-06	5419.3
MAG (2009)	1911-2107 May 2 1802-1958 May 3	46.6	P1(228) P2(102) P3(137)	7.69E-05 4.00E-05 3.17E-05	5942.8
MGup (2010)	1845-1950 April 14 1903-2005 April 15	61.6	P1(117) P2(52) P3(58) P4(141)	1.23E-04 1.25E-05 6.37E-06 2.17E-06	3606.6
MAGb (2010)	2231-2338 April 14 2319-0000 April 15	46.6	P1(176) P2(190) P3(218)	3.13E-04 2.47E-05 8.02E-06	14101.8
MGdown (2010)	2041-2141 April 15 2015-2101 April 16	58.0	P1(181) P2(174) P3(79)	2.44E-04 6.89E-05 5.12E-05	13795.4
MGupAB (2010)	1957-2024 May 11 1852-1954 May 12	61.6	P1(180) P2(72) P3(106) P4(81)	1.32E-05 6.92E-06 6.29E-07 3.92E-06	747.7
MGupBC (2010)	1852-1954 May 12 1748-1856 May 13	61.6	P1(187) P2(65) P3(116) P4(71)	2.57E-05 6.43E-06 4.53E-06 2.23E-06	1353.5
MGupCD (2010)	1748-1856 May 13 1657-1743 May 14	61.6	P1(231) P2(95) P3(69) P4(38)	2.27E-05 8.83E-06 2.80E-06 2.47E-05	1652.6

MGupDE (2010)	1657-1743 May 14 1609-1658 May 15	61.6	P1(210) P2(66) P3(132) P4(39)	4.91E-05 1.03E-05 1.27E-05 2.18E-05	2898.6

*MGup = Myrtle Grove survey area above the bend, MGbend = Myrtle Grove survey area within the bend, MGdown = Myrtle Grove survey area below (downriver) of the bend, MAG = Magnolia

**** Times for these surveys are in Greenwich Mean (GMT)

*** River mile of the center of the survey grid reach

[a] Figures are in metric tons (1 MT – 1,000 kg)

[^]See text for corrected value for water discharge when Myrtle Grove data was collected (April 7-8th).

Bedload (bedform) flux rate results for each site and cruise are shown in Table 2. Given that discharge varied over the dates of the surveys for each of the three grid areas (MGup, MGdown, MAG/MAGb), intercomparison of the data presented in Table 2 requires a correction for discharge. For example, the Magnolia data in April 2009 was collected on the 4-5th, while the Myrtle Grove data (both areas) was collected on April 7-8th. Utilizing 1) the water discharge at Tarbert Landing for those days (average of the two survey days) and 2) the log relationship of bedload transport mass flux with linear water discharge shown in Nittrouer et al. (2008) and Figure 6, the Magnolia data is revised upward to 3804.2 tons/day (based on 694,000 cfs on April 4-5 and 723,500 cfs on April 7-8th). Tarbert discharges were not corrected for the time lag between the site and Myrtle Grove given the variable mean downstream flow velocity with discharge—this error is thought to fall within the measurement error.

4.0 WATER DISCHARGE

Cross-sectional water discharge was collected in 2008-2009 along a series of transects (primarily to aid in calculation of suspended sediment load along those cross-sections) that included all four study grids (MGup, MGbend, MGdown, MAG). In addition in 2009, transects were collected as well as 1) two additional cross-sections (2 crossings), 2) one thalweg downriver line at Myrtle Grove, and 3) stationary data collected at two sediment sampling stations in MGbend. Data was collected at the 4 main transect sites except MGbend using an RD Instruments 120kHz Workhorse Monitor acoustic doppler current profiler (ADCP). This ADCP was bow-mounted on the R/V *Lake Itasca*. Data was collected and cross-sectional discharge was integrated using RD Instruments WinRiver© software package. Data were collected following US Geological Survey standard methods outlined in Edwards and Glysson (1988). Four replicate cross-sections were done at the four main transects within a 30-60 min period and results averaged to yield a mean discharge at the cross-section. Vertical data was binned at 0.5 m depth cells and position was obtained from a differential GPS, which is necessary in moving bed situations where the bottom tracking method is inaccurate.

The WinRiver software integrates by extrapolating for the four areas of the channel cross-section not measured; e.g., surface water above the sensor, the near bottom, right and left banks too shallow to survey. The depth of sensor is measured and the distance to the banks are input for each survey by the operator.

Depending on water temperature, the maximum depth of the 1200 kHz unit is ~40 m, hence the bottom could not be reached in the MGbend area where water depths reach 60 m (Fig. 3). For these areas, a 600 kHz RD Instruments Workhorse Monitor was borrowed from the Louisiana USGS Water Sciences Center. The instrument was operated identically to the 1200 kHz unit except vertical data was binned at 1 m depth cells. It should also be noted that discharges (and standard deviations) calculated from the MGbend and MGbelowbend transects was in a zone where the discharge numbers were complicated by the upriver-flowing eddy along the east bank of the river. The latitude/longitude of the starting and stopping points of all transects shown in Table 3 is also presented in supplemental digital file. This file also contains locations of all

A different water discharge and suspended sediment sampling strategy was followed in April and May 2010. At Magnolia, the newly created basemap in April was used to setup a grid of 4 cross-sectional and four river longitudinal transects (Fig. 7a). The 2009 transect location in Table 3, was repeated on 4/13/10 but none of the other new cross-sections were surveyed in April. On 4/14 the four longitudinal lines were surveyed with ADCP. At Myrtle Grove Up, a similar grid of 5 cross-sectional and four river longitudinal transects (Fig. 7a) was created in April 2010. The longitudinal lines crossed the MGup transect (second furthest upriver of the 5 cross-sections in Fig. 7b). The pre-existing MGup transect in Table 3 was resurveyed on 4/13/10, and the four longitudinal lines on 4/13/10 and repeated on 4/14. In addition, a single survey of 14 cross-sections was conducted on 4/16/10 to examine the east bank eddy structure at Myrtle Grove (Fig. 8). The survey was done from downriver to upriver with transect one at MGdown, transect 6 at MGbend, and transect 12 at MGup: the five cross-sections shown in Fig. 7a are transects 9-13 on this survey. Table 3 reports only the discharges from the pre-existing MGup, MGbend and MGdown stations from this survey. All ADCP data collected in April 2010 was collected with the 1200kHz unit on 4/13-4/14 and the 600kHz unit on 4/15-4/16.

In May 2010 at Myrtle Grove Up, ADCP cross-sectional data was collected at the five transects shown in Fig. 7a on 5/11, and again on 5/13 and 5/15. Cross-sectional data was only collected at MGup (transect 4) on 5/12 and 5/14. Only the MGup data is reported in Table 3. The four Myrtle Grove Up longitudinal transects in Fig. 7a were surveyed on 5/11/10, and again on 5/13 and 5/15. In May 2010 at Magnolia, ADCP cross-sectional data was collected at the four transects shown in Fig. 7a on 5/11, and again on 5/13 and 5/15. Cross-sectional data was only collected at MAG (second most upriver) on 5/12 and 5/14. Cross-sectional data was only collected at MAG on 5/12 and 5/14. Only the MGup data is reported in Table 3. The four Magnolia longitudinal transects in Fig. 7a were surveyed on 5/12/10, and again on 5/14 and 5/15. All ADCP data collected in May 2010 was collected with the 1200kHz unit.

Table 3. Cross-sectional Water Discharges at Myrtle Grove and Magnolia in 2008-10

Survey Grid*	Study Dates/hours**	River Mile***	Discharge at Tarbert Landing, MS (m^3/sec)	Mean Measured Discharge (m^3/sec)	Standard Deviation of Discharge (m^3/sec)
MGup	10/10/08 2257-2331	61.6	9,968	11,253	491
MGup	04/07/09 1823-1855	61.6	20,331	19,964	512
MGabovebend	04/09/09 2030-2045	59.9	20,954	20,980[a]	423
MGbend	04/09/09 1748-1820	59.3	20,954	18,458[a]	1,402
MGbelowbend	04/09/09 1916-1930	58.9	20,954	18,682 [a]	962
MGdown	04/05/09 1916-1950	58.0	19,737	18,886	1,349
MGdown	04/09/09 2054-2110	58.0	20,954	20,862[a]	1,081
MAG	04/04/09 1653-1732	46.6	19,567	18,987	609
MGup	05/02/09 1729-1802	61.6	22,370	21,705	432
MGabovebend	05/07/09 1759-1805	59.9	23,220	19,981[a]	-
MGbend	05/07/09 1723-1753	59.3	23,220	20,214[a]	1,325
MGbelowbend	05/07/09 1813-1820	58.9	23,220	21,078[a]	-
MGdown	05/01/09 2119-2155	58.0	22,512	20,697	956
MAG	05/01/09 1937-2020	46.6	22,512	21,092	996
MGup	04/13/10 1714-1737	61.6	25,200	24,767	914
MAG	04/13/10 1212-1237	46.6	25,200	23,861	842
MGup	04/16/10 1327-1338	61.6	23,616	23,797[a]	206
MGbend	04/16/10 1153-1205	59.3	23,616	22,901[a]	1444
MGdown	04/16/10 1029-1048	58.0	23,616	22,819[a]	1699
MGup	05/11/10 1321-1342	61.6	20,440	18,770	459
MAG	05/11/10 1749-1755	46.4	20,440	19,343	1233
MGup	05/12/10 1507-1540	61.6	20,860	19,548	533
MAG	05/12/10 1742-1748	46.4	20,860	19,891	1326
MGup	05/13/10 1540-1546	61.6	21,364	19,885	409
MAG	05/13/10 1837-1844	46.4	21,364	20,327	1,416
MGup	05/14/10 1417-1425	61.6	21,868	22,332	619
MAG	05/14/10 1617-1638	46.4	21,868	20,443	941
MGup	05/15/10 1045-1051	61.6	22,344	20,572	858
MAG	05/15/10 1443-1451	46.4	22,344	20,701	1241

*MGup = Myrtle Grove survey area above the bend, MGbend = Myrtle Grove survey area within the bend, MGdown = Myrtle Grove survey area below (downriver) of the bend, MAG = Magnolia, MGabovebend = Myrtle Grove survey area between MGup and MGbend, MGbelowbend = Myrtle Grove survey area between MGbend and MGdown

**** Times for these surveys are in Greenwich Mean (GMT)

*** River mile of the center of the survey grid reach

[a] Data was collected with the 600 kHz ADCP unit on loan from the USGS Baton Rouge.

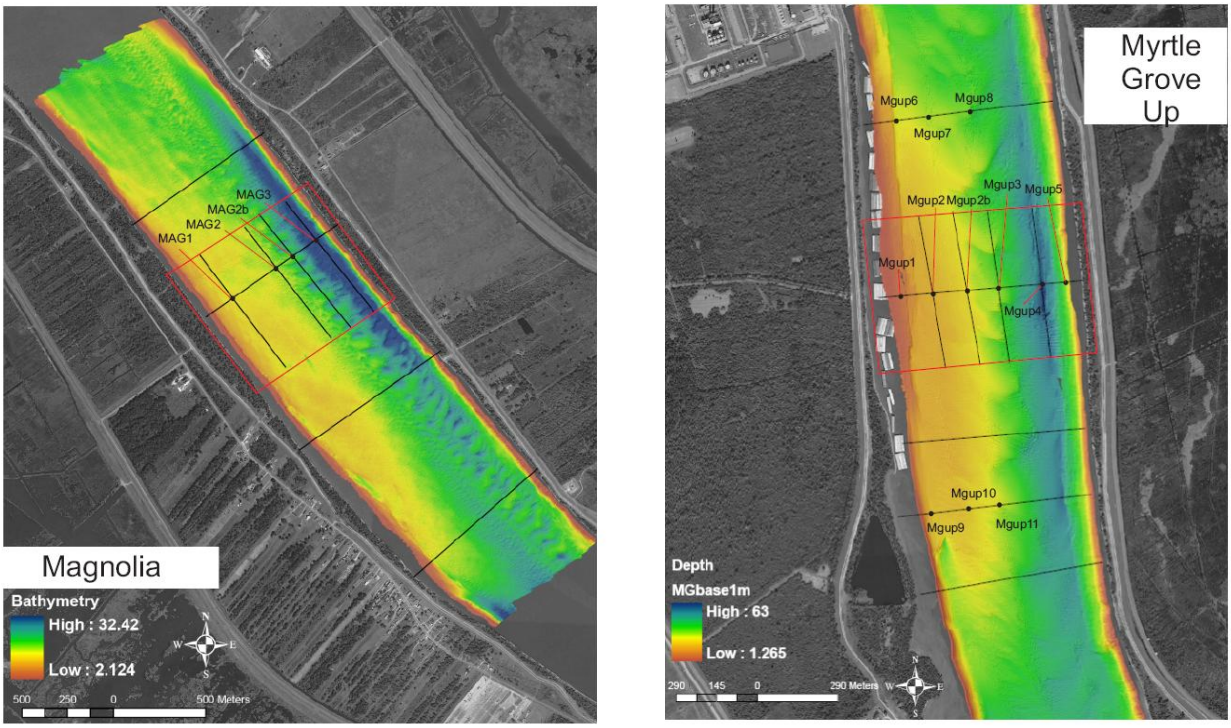

Figure 7a. Sampling stations and ADCP transects utilized in 2008-2010 at Magnolia (left) and Myrtle Grove Up (right). Limits of the bedload grid areas are shown in red.

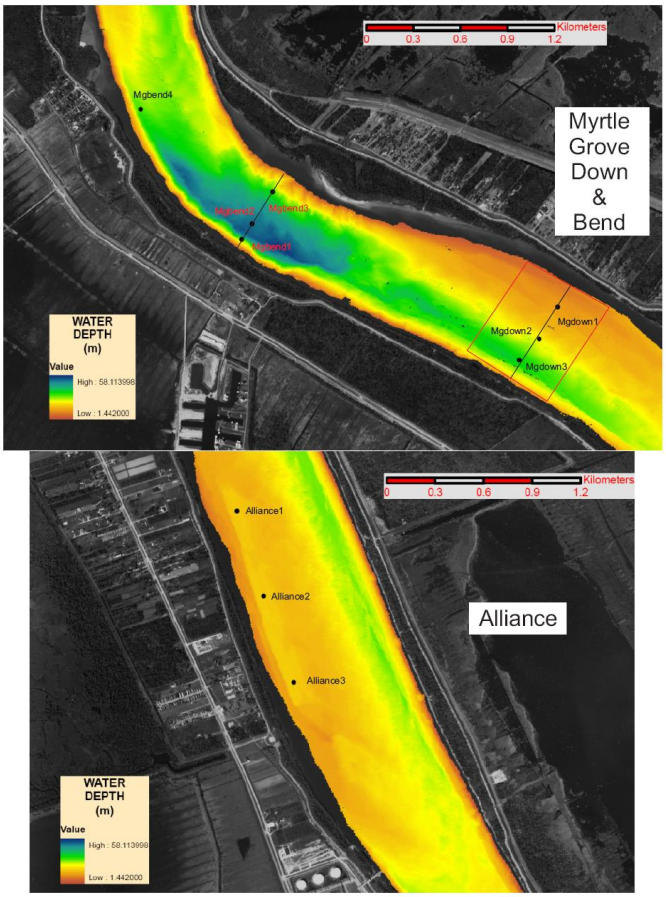

Figure 7b. Sampling stations and ADCP transects utilized in 2008-2010 at Myrtle Grove Bend and Down (upper) and Alliance (lower). Limits of the bedload grid areas are shown in red.

the stationary (MGbend1, MGbend2, MGbend3 on April 7) and thalweg lines (Myrtle Grove thalweg upriver on April 7) not utilized for water discharge calculations. WinRiver format and ascii format digital files are also provided for each ADCP transect/station.

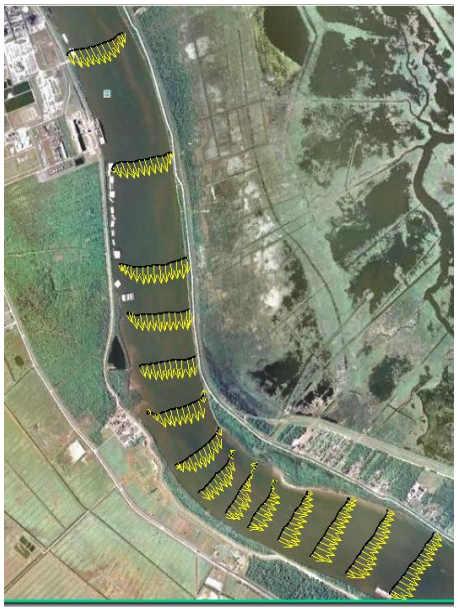

Figure 8. ADCP 14 cross-section survey conducted at Myrtle Grove on 4/16/10 showing line locations and mean, depth averaged, velocity vector (length of line indicates velocity magnitude).

5.0 ISOKINETIC SUSPENDED SEDIMENT SAMPLES

Suspended sediment point samples in 2008-2009 were collected with a P-63 isokinetic sampler obtained from the USGS Hydrographic Instrumentation Facility (HIF) in Stennis, MS and calibrated by the Federal Interagency Sedimentation Project (FISP) in Vicksburg, MS. The P-63 is a 208 lb sampler that utilizes quart-sized bottles for samples collected in less than 120' of water, and pint-sized bottles for deeper sampling. The winch wire on the R/V *Itasca* is 150 ft in length, limiting sampling in the deepest thalweg reaches in Myrtle Grove (which can reach 200 ft depth). In 2010 an improved sampler (P-6), built by Wayne O'Neill, former FISP director, was utilized—it is of the same weight and geometry as the older sampler. Generally three stations were occupied in each cruise along each of the cross-sectional transects where ADCP , bottom grabs, CTD/LISST casts and bedload transport rates were also being conducted. Latitude/longitude data for these station locations can be found in the table in Section 7. At the Myrtle Grove stations in 2009, P-63 sampling was done following standard USGS methods of samples at 0.1,0.30.5, 0.7 and 0.9 water depth. In the deepest stations in the bend (e.g., MGbend2 and MGbend3) data was not collected from these intervals where the depth exceeded winch wire length. At Myrtle Grove in 2009, sampling was done at the surface (0), 0.5 and 0.9 total water depth. Data collection points are shown in Table 4. In 2010, the April survey was conducted at the 2009 stations at both Myrtle Grove and Magnolia, but all stations used the surface in lieu of a 0.1 water bottle, and Magnolia was sampled at 5 depths instead of the 3 used in 2009. In May 2010, the MGup and Magnolia stations were sampled twice—on 5/11 and 5/15—but at only 0, 0.5 and 0.9 depths.

Water samples from the P-63 were vacuum-filtered through pre-weighed 0.2 micron polycarbonate filters to concentrate sediment. At the time when the samples were poured into

the vacuum chamber, they were filtered through a 63 micron (4 phi) seive to remove sand. Sand fractions were dried in an oven at 60°C and weighed to determine total sand weight. Mud fractions (clay and silt finer than 63 microns) on filters were then dried in an oven at 60°C and re-weighed to determine total mud weight. A graduated cylinder was utilized to determine water sample size (ml) of the filtered sample, and total suspended concentration (in mg/l) determined from this value and the summation of mud and sand concentrations. Table 4 also shows the results of these suspended sediment concentration determinations.

Table 4. Isokinetic Point Sampler (P-63) Suspended Sediment Concentration Values at Myrtle Grove and Magnolia in 2008-10

Survey Grid and Station*	Station Date/Time **	Water Depth (m)	Concentration Sand (mg/l)***	Concentration Mud (mg/l) [a]
MGup2 0.1	10/10/08 1845-2050	1.49	0.00	39.78
0.3		4.48	0.00	45.91
0.5		7.50	0.00	88.47
0.7		10.52	0.00	95.41
0.9		13.50	0.00	115.15
MGup3 0.1	10/10/08 1845-2050	2.29	0.00	77.91
0.3		6.89	0.00	106.87
0.5		11.61	0.00	125.67
0.7		16.09	0.00	158.73
0.9		20.70	5.33	234.67
MGup4 0.1	10/10/08 1845-2050	4.82	0.00	35.06
0.3		12.19	0.00	129.35
0.5		20.39	4.88	126.18
0.7		28.59	2.86	168.86
0.9		36.70	0.00	164.49
MGup2 0.1	04/11/09 2111-2127	1.83	4.58	121.20
0.3		5.18	13.58	144.78
0.5		8.53	17.72	134.39
0.7		11.58	33.39	150.68
0.9		14.94	63.67	138.17
MGup3 0.1	04/11/09 2133-2214	2.44	22.70	143.33
0.3		7.32	26.85	137.59
0.5		12.80	58.98	145.10
0.7		18.29	94.11	156.43
0.9		21.95	252.79	163.72

MGup4 0.1 0.3 0.5 0.7 0.9	04/11/09 2149-2209	3.96 11.28 19.20 26.52 34.44	36.21 37.10 43.79 38.29 67.05	152.73 148.99 158.97 160.17 172.95
MGbend1 0.1 0.3 0.5 0.7 0.9	04/09/09 1608-1628	3.35 10.36 17.37 24.69 31.39	17.66 15.86 18.10 14.79 22.98	120.47 113.71 121.90 115.63 119.36
MGbend2 0.1 0.3 0.5 0.7	04/09/09 1640-1830	4.88 14.63 24.38 34.14	30.97 48.91 54.89 45.38	131.29 131.82 133.11 146.92
MGbend3 0.1 0.3 0.5 0.7 0.9	04/09/09 1840-1846	3.05 9.14 15.24 21.34 27.43	26.48 75.21 51.72 66.13 132.22	145.07 149.58 154.14 136.53 169.63
MGdown1 0.1 0.3 0.5 0.7 0.9	04/05/09 2008-2020	1.40 4.27 7.01 9.75 12.50	29.60 32.10 39.41 62.63 69.56	169.40 171.77 174.51 198.60 195.58
MGdown2 0.1 0.3 0.5 0.7 0.9	04/05/09 2025-2150	2.44 7.32 12.19 17.07 21.95	30.21 32.44 74.41 77.09 146.17	163.19 170.55 185.38 170.60 179.57
MGdown3 0.1 0.3 0.5 0.7 0.9	04/05/09 2025-2150	3.66 10.97 18.29 25.60 32.92	15.90 27.64 34.46 38.64 56.14	140.96 162.36 163.93 174.32 167.86

MAG1 0 0.5 0.9	04/05/09 1739-1743	0.00 10.06 17.68	3.66 34.39 55.58	101.97 168.60 176.46
MAG2 0 0.5 0.9	04/05/09 1751-1754	0.00 11.28 20.42	14.41 67.26 204.03	154.55 178.41 185.88
MAG3 0 0.5 0.9	04/05/09 1759-1802	0.00 14.02 25.30	6.67 27.39 72.26	120.41 156.76 205.38
MGup2 0.1 0.3 0.5 0.7 0.9	05/04/09 1817-1946	1.49 4.48 7.50 10.52 13.56	8.85 15.52 18.75 15.00 8.13	61.95 91.38 93.75 95.00 79.67
MGup3 0.1 0.3 0.5 0.7 0.9	05/04/09 1950-2200	2.44 7.32 12.80 18.29 23.16	21.67 15.63 12.50 19.64 18.60	96.67 96.88 93.75 105.36 111.63
MGup4 0.1 0.3 0.5 0.7 0.9	05/04/09 1950-2200	3.66 10.97 18.29 25.60 32.92	10.00 11.63 10.53 16.67 22.54	76.67 97.67 98.25 103.03 107.04
MGbend1 0.1 0.3 0.5 0.7 0.9	05/06/09	3.35 10.36 17.07 24.08 31.09	8.85 8.51 8.77 12.03 8.45	90.27 95.74 96.49 96.24 92.96

MGbend2 0.1 0.3 0.5 0.7	05/06/09	4.88 14.63 24.38 34.14	7.48 12.80 20.31 27.78	65.42 112.00 115.63 112.50
MGbend3 0.1 0.3 0.5 0.7 0.9	05/06/09	3.35 9.75 16.46 23.16 29.87	27.27 42.35 36.04 40.32 67.16	101.82 105.88 93.69 106.45 111.94
MGdown1 0.1 0.3 0.5 0.7 0.9	05/06/09	1.22 3.66 6.10 8.53 10.67	6.96 9.09 18.46 20.69 31.11	90.43 113.64 107.69 108.05 108.89
MGdown2 0.1 0.3 0.5 0.7 0.9	05/06/09	2.44 7.01 11.58 16.15 20.73	3.45 12.50 23.91 38.18 27.12	70.69 96.88 95.65 107.27 94.92
MGdown3 0.1 0.3 0.5 0.7 0.9	05/06/09	3.35 10.36 17.37 24.38 31.39	14.55 4.76 7.14 17.19 26.09	87.27 83.33 92.86 98.44 101.45
MAG1 0 0.5 0.9	05/04/09 1525-1532	0.00 10.06 17.68	3.28 29.69 100.00	62.30 109.38 110.87
MAG2 0 0.5 0.9	05/04/09 1543-1550	0.00 11.28 20.42	3.39 73.33 174.65	61.02 115.00 123.94

MAG3 0 0.5 0.9	05/04/09 1608-1611	0.00 14.02 25.30	3.48 34.37 82.35	67.83 109.38 113.73
MGup2 0 0.3 0.5 0.7 0.9	04/15/10 2204-2212	0 5.19 8.85 12.51 15.86	0.00 10.00 19.61 13.59 8.00	58.97 77.5 90.20 73.79 90.00
MGup3 0 0.3 0.5 0.7 0.9	04/15/10 2226-2236	0 8.23 14.03 19.22 24.71	3.61 33.01 20.76 35.00 84.00	50.60 81.55 81.13 98.33 44.00
MGup4 0 0.3 0.5 0.7 0.9	04/15/10 2244 04/16/10 1944-1951	0 11.90 13.73 19.51 24.71	0.00 4.51 27.72 28.89 34.01	87.10 90.23 81.19 155.56 68.03
MAG1 0 0.3 0.5 0.7 0.9	4/15/10 1751-1814	0 5.80 9.46 13.73 17.08	0.00 14.93 8.00 3.08 67.37	29.81 104.48 92.00 61.54 96.84
MAG2 0 0.3 0.5 0.7 0.9	4/15/10 1738-1742	0 6.41 10.07 13.73 17.69	4.29 12.35 39.53 44.04 82.35	50.00 93.83 100.00 73.39 105.88

MAG3 0 0.3 0.5 0.7 0.9	4/15/10 1635-1732	0 9.15 15.56 21.96 28.37	0.00 0.00 25.93 15.38 28.57	60.67 72.88 83.33 86.71 81.63
MGup2 0 0.5 0.9	5/11/10 1915-1923	0 9.15 16.17	0.00 7.32 27.91	147.10 351.22 327.91
MGup3 0 0.5 0.9	5/11/10 1929-1935	0 14.03 25.01	0.00 12.80 193.75	272.26 388.80 393.75
MGup4 0 0.5 0.9	5/11/10 1941-1947	0 19.83 35.38	0.00 22.22 9.70	153.33 351.39 378.18
MAG1 0 0.5 0.9	5/11/10 0024-0028	0 9.76 17.39	0.00 16.67 14.95	105.36 363.89 370.09
MAG2 0 0.5 0.9	5/11/10 0016-0020	0 10.37 18.00	0.00 7.81 68.00	107.14 350.00 340.00
MAG3 0 0.5 0.9	5/11/10 0007-0011	0 15.86 28.67	0.00 7.45 6.72	139.87 351.06 375.00
MGup2 0 0.5 0.9	5/15/10 1520-1523	0 9.15 16.17	0.00 23.08 19.61	103.23 169.23 168.63

MGup3 0 0.5 0.	5/15/10 1513-1515	0 14.03 25.01	0.00 25.24 166.97	110.84 192.23 190.83
MGup4 0 0.5 0.9	5/15/10 1502-1508	0 19.83 35.38	0.00 9.93 12.79	73.60 194.33 183.72
MAG1 0 0.5 0.9	5/15/10 1358-1401	0 9.76 17.39	0.00 11.49 24.79	83.53 193.10 204.96
MAG2 0 0.5 0.9	5/15/10 1405-1409	0 10.37 18.00	0.00 36.54 83.72	119.75 269.23 212.40
MAG3 0 0.5 0.9	5/15/10 1412-1416	0 15.86 28.67	0.00 15.75 29.41	106.83 193.70 226.47

*MGup = Myrtle Grove survey area above the bend, MGbend = Myrtle Grove survey area within the bend, MGdown = Myrtle Grove survey area below (downriver) of the bend, MAG = Magnolia, MGabovebend = Myrtle Grove survey area between MGup and MGbend, MGbelowbend = Myrtle Grove survey area between MGbend and MGdown. Station numbers are identical to those whose positions are shown in Section 7. Fractional water depths are also shown for each sample at the station.

**** Times for these surveys are in Greenwich Mean (GMT)
*** Refers to the fraction larger than 63 microns collected on a seive
[a] Refers to the fraction smaller than 63 microns that passed through the sieve and was collected onto 0.2 micron filters

6.0 SUSPENDED SEDIMENT LOADS

Suspended sediment loads were calculated from the results presented in Table 3 (water discharges) and Table 4 (isokinetic point sampler data) utilizing the USGS methods outlined in Edwards and Glysson (1988). Each river cross-section is divided into subsections that each have a vertical sampler station at the mid-point of the subsection. In each cross-section, the mean concentration of sediment was calculated as an average of the P-63 isokinetic point samples shown in Table 4. The WinRiver ADCP software was then utilized to calculate the portion of the total cross-sectional water discharge contained within that sub-section from surface to

bottom. This was done for each of the four repeat cross-sections and the average utilized for the suspended load calculation. The resulting suspended sediment loads are shown in Table 5 (uncorrected sediment load columns for sand and mud). All sediment loads are expressed in metric tons/day.

There are several sources of error in the calculations. Slight variations in water discharge may result from diurnal tidal modulation of flow. These tend to be magnified as you approach the Gulf and at lower river discharge phases. Given that all sections are above river mile 46, we anticipate that this source of error is relatively minor. Another source of error relates to time differences between when the ADCP water discharge and P-63 suspended sediment concentration data was collected: same day of collection was accomplished in many of the sections, but the time difference was as great as 5 days (MGdown in May 2009). A third source of error is found when intercomparing sections collected on the same cruise on different days when river discharge—the individual sections were collected over a maximum six day period (May cruise). This is also clear in May 2010 when both MGup and MAGb were sampled on 5/11 and 5/15 during a rising discharge (see Fig. 2). The ADCP versus P-63 time difference was reconciled in Table 5 (corrected sediment load columns for sand and mud) by using the day-by-day water discharge calculated at Tarbert Landing, MS above the reach of tides. The P-63 data collection date was utilized as the baseline and a ratio multiplied to the uncorrected data based on the water discharge difference at Tarbert on the two dates. The latter source of error is not accounted for in Table 5.

A final source of variability, but not strictly error, was observed in the MGbend and MGdown cross-sections where the eddy on the inside of the Myrtle Grove bend resulted in upriver flows along the east bank of the river. These yield negative discharges for those subsections, but vertical cast stations were not optimized for isolating these upriver-flowing areas in individual subsections. In general, suspended sediment load calculations are not made by most groups in river bend regions for this region, hence, these results must be interpreted with caution.

Table 5. Suspended sediment loads at Myrtle Grove and Magnolia in 2008-10

Survey Grid*	Date (P-63) Date (ADCP)	Ratio†	Uncorr Sand*** (tons/d)	Uncorr Mud[a] (tons/d)	Corr Sand (tons/d)	Corr Mud (tons/d)
MGup	10/10/08 10/10/08	1.000	1,006	115,410	1,006	115,410
MGup	04/07/09 04/07/09	1.000	97,118	259,344	97,118	259,344
MGbend	04/09/09 04/09/09	1.000	71,335	224,535	71,335	224,535
MGdown	04/05/09 04/05/09	1.000	81,592	275,776	81,592	275,776
MAG	04/05/09 04/04/09	1.009	101,304	265,010	100,400	262,646
MGup	05/04/09 05/02/09	0.992	28,052	175,320	28,278	176,734

MGbend	05/06/09 05/07/09	0.978	36,442	168,573	37,262	172,362
MGdown	05/06/09 05/01/09	1.009	30,777	173,097	30,502	171,553
MAG	05/04/09 05/01/09	0.986	108,077	172,290	109,612	174,736
MAG	04/15/10 04/13/10	0.940	47,754	164,217	44,890	154,364
MGup	04/15/10 04/13/10	0.940	47,444	177,501	44,597	166,851
MAG	05/11/10 05/11/10	1.000	19,209	468,051	19,209	468,051
Mgup	05/11/10 05/11/10	1.000	49,502	500,229	49,502	500,229
MAG	05/15/10 05/15/10	1.000	41,558	321,127	41,558	321,127
MGup	05/15/10 05/15/10	1.000	49,531	274,062	49,531	274,062

*MGup = Myrtle Grove survey area above the bend, MGbend = Myrtle Grove survey area within the bend, MGdown = Myrtle Grove survey area below (downriver) of the bend, MAG = Magnolia in 2009/April 2010, MGabovebend = Myrtle Grove survey area between MGup and MGbend, MGbelowbend = Myrtle Grove survey area between MGbend and MGdown.

[†]Refers to the ratio of water discharge at Tarbert Landing, MS on the P-63 data collection day versus the ADCP data collection day. This is the ratio utilized in the corrected sediment load columns.

*** Refers to the fraction larger than 63 microns collected on a seive.

[a] Refers to the fraction smaller than 63 microns that passed through the sieve and was collected onto 0.2 micron filters

7.0 SEDIMENT BOTTOM GRABS

Sediment bottom sampling was conducted at each vertical cast station in October, April and May 2009 cruises where suspended sediment concentration data was collected except those whose depth exceeded the 150' wire length on the R/V *Itasca* winch. Samples were collected with a Shipek®, spring-fired bucket grab, which samples to a depth of about 5 cm. Samples were bagged and returned to the laboratory for grain size analyses. Station locations are shown in Table 6. Grab sampling was not conducted in April 2010, but was done at selected stations in the May 2010 cruise. This includes the three bar (non-thalweg) stations at MGup and MAG, additional samples on the bars above and below MGup (MGup 5-11) and MAG (MAG5-9) transects , and three grabs from the area near Alliance where dredging was conducted and the basemap was extended in April 2010 (Alliance 1-3).

Grab samples from each station and date were wet seived through stacked 2mm (-1 phi) and 63 micron (4 phi) seives to concentrate the gravel and sand fractions, respectively. The filtrate (containing particulates finer than 63 microns) was washed into a 1 liter graduated cylinder with a 0.5% concentration solution of sodium metaphosphate (a flocculation inhibitor). This water-sediment mixture was then raised to 1 liter using the the same metaphosphate solution, stirred and a 25 ml aliquot was taken. This aliquot was tried in an oven at 60°C to determine weight of silt/clay (<63 micron fraction) after multiplying to account for the 1/40th aliquot of the total mud-water mixture. Sand and gravel fractions were as dried in an oven at 60°C and weighed to determine total weights. Fractional (%) weights of these fractions are reported in Table 6. All gravel fraction materials were observed to be shell material.

Table 6. Grain size of bottom grabs collected at Myrtle Grove and Magnolia in 2008-10

Survey Grid and Station	Collection Dates	Water Depth (m)	latitude	longitude	% gravel	%sand	%mud
MGup1	10/10/08	9.1	29 39.817	89 57.711	0.00	13.95	86.05
	05/06/09				0.00	26.02	73.98
MGup2	10/10/08				0.00	99.13	0.87
	04/07/09	15.6			1.56	98.34	0.10
	05/06/09		29 39.826	89 57.640	0.00	99.94	0.06
	05/12/10				0.06	99.93	0.00
MGup2b	05/12/10	19.2	29 39.835	89 57.564	0.05	99.95	0.00
MGup3	10/10/08				0.00	99.14	0.86
	04/07/09	23.8			0.00	99.83	0.07
	05/06/09		29 39.843	89 57.489	0.00	99.92	0.08
	05/12/10				0.00	99.99	0.01
MGup4	10/10/08				0.00	100.00	0.00
	04/07/09	38.7	29 39.853	89 57.397	1.90	97.96	0.13
	05/06/09				0.00	100.00	0.00
MGup5	10/10/08	25.9	29 39.860	89 57.347	18.84	29.83	51.33
	05/06/09				0.00	100.00	100.00
MGup6	05/12/10	22.6	29 40.175	89 57.728	0.27	99.73	0.00
MGup7	05/12/10	21.9	29 40.182	89 57.659	1.33	98.66	0.01
MGup8	05/12/10	27.5	29 40.192	89 57.568	0.00	99.99	0.01
MGup9	05/12/10	17.7	29 39.400	89 57.635	26.75	72.78	0.47
MGup10	05/12/10	19.8	29 39.413	89 57.522	0.00	100.00	0.00
MGup11	05/12/10	28.1	29 39.423	29 57.429	0.00	100.00	0.00
MGbend1	04/08/09	36.8	29 38.466	89 57.011	93.70	0.00	6.30
	05/07/09				0.00	100.00	0.00
MGbend2	Too deep	51.6	29 38.521	89 56.971	-	-	-
MGbend3	04/08/09	32.4	29 38.632	89 56.893	2.72	89.20	8.08
	05/01/09				0.00	100.00	0.00
MGbend4	CTD site only	31.7	29 38.906	89 57.423	-	-	-

MGdown1	04/05/09 05/01/09	14.0	29 38.262	89 55.754	0.00 0.00	99.96 99.96	0.04 0.04
MGdown2	04/05/09 05/01/09	24.4	29 38.151	89 55.825	0.00 0.00	100.00 100.00	0.00 0.00
MGdown3	04/05/09 05/07/09	36.6	29 38.076	89 55.902	0.81 0.00	99.19 100.00	0.00 0.00
MAG1	04/08/09 05/06/09 05/13/10	19.8	29 33.212	89 46.441	0.00 0.00 0.00	100.00 99.95 100.00	0.00 0.05 0.00
MAG2	04/08/09 05/06/09 05/13/10	24.1	29 33.283	89 46.331	0.00 7.69 0.11	100.00 91.20 99.89	0.00 1.12 0.00
MAG2b	05/13/10	19.9	29 33.101	89 46.079	10.31	89.68	0.00
MAG3	04/08/09 05/06/09	28.0	29 33.364	89 46.193	0.00 0.00	99.96 100.00	0.04 0.00
MAG4	05/13/10	17.8	29 33.249	89 46.509	0.04	99.96	0.00
MAG5	05/13/10	18.4	29 33.303	89 46.431	0.27	99.73	0.00
MAG6	05/13/10	22.3	29 33.357	89 46.342	12.46	87.53	0.01
MAG7	05/13/10	17.5	29 32.238	89 45.555	0.08	99.88	0.05
MAG8	05/13/10	20.1	29 32.313	89 45.459	0.00	100.00	0.00
MAG9	05/13/10	22.4	29 32.418	89 45.336	0.00	99.99	0.01
Alliance1	05/14/10	17.8	29 42.815	89 59.235	0.40	99.59	0.01
Alliance2	05/14/10	17.5	29 42.532	89 59.124	0.00	99.97	0.03
Alliance3	05/14/10	19.0	29 42.247	89 59.000	0.00	99.98	0.02

Sand fractions from bottom grabs that were separated and weighted for the aforementioned %gravel:sand:mud analysis were further analyzed for detailed size fractions using a Femto automated settling column. The column generates frequency percentages at 1/10 phi size fraction intervals. This data is provided separately in digital files. Table 7 shows statistical results for mean, standard deviation (sorting), median and 10% and 90% fraction and skewness/kurtosis from the settling column analysis.

Table 7. Sand grain size statistics of bottom grabs at Myrtle Grove and Magnolia in 2008-10

Survey Grid and Station	Collection Dates and Times	Mean (µm)*	Std Dev (µm)	Skewness	Kurtosis	D_{10} (µm)[a]	D_{50} (µm)	D_{90} (µm)
MGup1	10/10/08 05/06/09	108.3 152.9	44.98 57.42	2.229 6.422	10.88 80.25	66.7 92.5	98.6 157.2	167.3 193.3
MGup2	10/10/08 04/07/09 05/06/08 05/12/10	349.0 270.5 304.5 268.0	166.4 80.27 111.8 69.67	3.851 0.822 5.227 0.248	22.19 5.197 51.57 3.940	233.9 189.6 214.5 174.6	320.4 261.5 288.5 271.4	465.7 371.9 405.2 349.2
MGup2b	05/12/10	260.1	66.94	5.424	94.54	199.9	256.6	325.3

MGup3	10/10/08	230.8	95.93	5.424	55.07	172.3	207.5	318.5
	04/07/09	189.1	28.07	-0.239	5.646	157.9	189.4	220.4
	05/06/09	199.2	45.13	8.516	164.2	162.4	194.7	238.3
	05/12/10	181.0	29.94	0.631	5.475	149.9	178.1	218.3
MGup4	10/10/08	281.9	63.50	2.184	26.61	226.1	273.5	352.9
	04/07/09	338.9	144.0	6.859	67.37	233.9	334.0	413.8
	05/06/09	213.7	43.32	0.769	4.512	166.2	208.0	270.4
MGup5	10/10/08	189.5	103.4	2.593	16.80	93.6	172.3	304.0
	05/06/09	221.7	135.0	2.248	7.560	131.7	176.9	437.1
MGup6	05/12/10	280.0	89.53	4.829	64.34	181.1	284.1	359.4
MGup7	05/12/10	360.3	122.7	3.997	32.18	248.7	350.4	455.0
MGup8	05/12/10	194.6	41.37	1.892	9.701	158.1	187.5	237.9
MGup9	05/12/10	162.0	70.03	7.283	100.1	107.6	154.4	208.6
MGup10	05/12/10	287.2	69.57	0.291	3.920	197.3	286.3	368.6
MGup11	05/12/10	211.9	46.46	1.114	5.208	163.3	203.6	271.8
MGbend 1	04/08/09	Shelly	-	-	-	-	-	-
	05/07/09	204.8	94.02	5.013	35.34	151.9	189.2	251.0
MGbend 2	Too deep	-	-	-	-	-	-	-
MGbend 3	04/08/09	167.4	67.60	7.766	105.6	110.9	165.3	205.2
	05/01/09	180.3	35.50	1.479	8.379	142.2	177.7	214.7
MGbend 4	CTD site only	-	-	-	-	-	-	-
MGdown 1	04/05/09	190.4	89.88	9.022	115.3	149.7	179.5	209.4
	05/01/09	192.6	31.69	0.380	5.220	159.5	189.7	232.4
MGdown 2	04/05/09	210.5	34.58	0.481	5.641	172.9	207.9	252.3
	05/01/09	207.6	36.55	1.078	6.788	170.3	203.0	252.0
MGdown 3	04/05/09	349.7	66.85	-0.479	5.661	263.2	357.4	418.3
	05/07/09	311.3	82.35	4.961	58.76	238.6	308.0	364.9
MAG1	04/08/09	274.1	59.37	3.092	88.62	207.0	272.6	343.3
	05/06/09	282.5	87.48	6.774	91.18	207.6	271.0	362.2
	05/13/10	271.7	44.69	-0.590	4.398	216.0	274.2	325.0
MAG2	04/08/09	204.2	39.51	1.034	6.158	165.9	196.7	255.1
	05/06/09	223.1	91.05	8.156	102.8	169.7	210.0	274.2
	05/13/10	234.6	51.92	0.752	4.494	175.1	226.6	302.9
MAG2b	05/13/10	2.034	56.48	13.46	363.9	162.0	193.4	258.2
MAG3	04/08/09	163.1	44.11	22.15	775.46	131.6	163.2	188.7
	05/06/09	181.1	51.43	23.22	78.9	156.7	178.1	202.1
MAG4	05/13/10	276.2	51.04	-0.072	4.327	213.0	277.6	336.6
MAG5	05/13/10	257.2	72.26	5.776	129.2	181.1	257.8	333.6

MAG6	05/13/10	188.2	41.99	14.81	571.5	154.4	182.2	228.4
MAG7	05/13/10	234.2	45.28	3.669	74.99	188.7	231.2	282.6
MAG8	05/13/10	257.3	160.3	7.745	67.33	184.2	241.7	302.5
MAG9	05/13/10	192.3	164.4	7.877	65.22	143.9	171.5	206.8
Alliance1	05/14/10	267.5	86.01	5.323	82.63	182.2	267.7	353.5
Alliance2	05/14/10	262.5	76.70	-0.106	2.157	163.1	270.0	359.8
Alliance3	05/14/10	209.4	49.61	0.268	2.983	152.6	203.6	276.7

* Mean, standard deviation, skewness and kurtosis were calculated using the arithmetic method of moments.

[a]D_{10} is the 10th percentile of cumulative size frequency, D_{50} is the 50th percentile (median), and D_{90} is the 90th percentile.

8.0 CTD/LISST CAST DATA

Casts were conducted at each station and date shown in Table 6 using a Seabird SBE-11 CTD (conductivity/temperature/depth) water column profiler. The unit records time-synchronized data at a 1 Hz frequency for the following instruments built into the profiler:

a) Microcat CT sensor (records water conductivity and temperature)
b) Paroscientific DigiQuartz pressure sensor
c) Pump flow-through system with pH and dissolved oxygen sensors
d) 5cm path length transmissometer (optical turbidity)
e) D&A Asssociates (now Campbell Scientific) OBS-3 optical backscatterance turbidity sensor

In addition to these sensors, a Sequoia Scientific Laser In Situ Scattering and Transmissometry (LISST) sensor was attached to the CTD profiler frame and operated during each cast. This LISST 100-X model, type C, measures particle size (disaggregated and flocculated combined) over a size range of 2.5 to 500 microns. This unit was operated with an 80% path reduction module to compensate for the relatively high turbidities in the Mississippi River (without the module, the unit saturates at concentrations above about 100 mg/l). The unit was operated by magnetic switch to begin sampling at a 1 Hz rate at the onset of the cast. Each 1 Hz sample is an average of 10 samples. Depth is provided by an internal pressure sensor separate from that of the CTD profiler. Data files are too large to provide in tabular form in this report, but are explained for each unit in the results sections below and provided as digital files for each station.

8.1 CTD PROFILES

Profiles from the CTD are provided in digital file form with filename and date matching the station designations in Table 6. Data has been edited to remove the initial part of the cast, where the profiler is held at the water surface to allow the pump to flush water through the system, and to remove the upcast after the sampler reaches maximum depth. This leaves only the downcast. Data is presented in tabular form with the following columns:

Col 1 Depth (m)
Col 2 OBS-3 Backscatter (nephelometric turbidity units--NTU)
Col 3 pH
Col 4 Oxygen (ml/L)
Col 5 Beam Transmission (fractional % with 100% = 1)
Col 6 Conductivity (uS/cm)
Col 7 Salinity (PSU)
Col 8 Temperature (degrees C)

Figure 8 is an example of the turbidity data (transmissometer and OBS) from a single station (MGup3) at each of the three cruise dates. Note the reversed response of the two optical systems: increasing suspended sediment concentration reduces transmission but increases backscatter. Note also the differences in average turbidity and water column structure with depth between the three cruises.

8.2 LISST SUSPENDED SEDIMENT GRAIN SIZE

LISST suspended sediment grain size data was post-processed using Matlab© scripts provided by the manufacturer (Sequoia Scientific) and described at:
http://sequoiasci.com/Articles/ArticlePage.aspx?pageId=128
It is necessary to calibrate the unit using a background value (zscat) for a particle-free sample of the water mass being studied. In the case of the Mississippi River surveys, those samples were collected in the field at the Myrtle Grove site, filtered through 0.2 micron polycarbonate filters and the water used to measure in triplicate values for each of the 32 size rings. This zscat file was then applied in the matlab script.

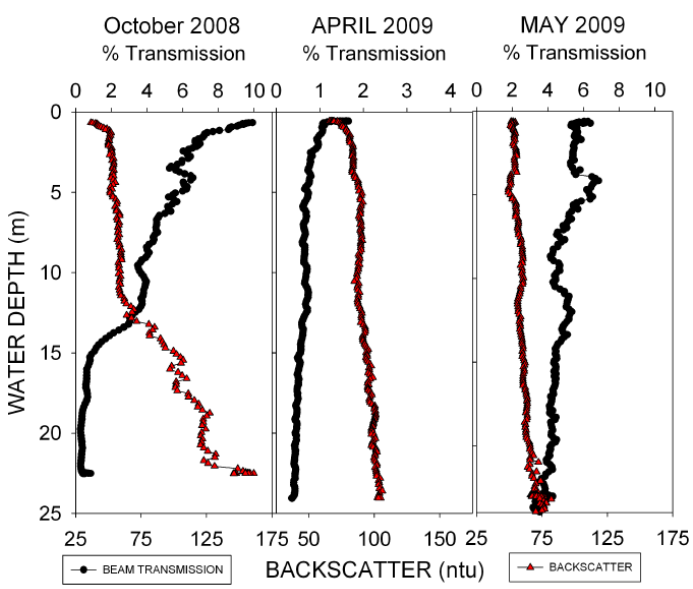

Figure 9. Example of optical turbidity data recovered from the CTD profiler at station MGup3 in October, April and May cruises. Data for both the optical backscatterance sensor and transmissometer are plotted.

The LISST processing yields a 42 column data table that is stored in ascii digital files provided with this report for each station and date listed in Table 6. The first 32 columns of these are outputs of 32 ring detectors, each of which measures scattering of laser light from particles into a

specific narrow sub-range of angles (e.g., particle sizes). These values are expressed in volume concentration (microliters/liter) for each size class represented by a ring. The other variables that are stored are:

Col(33) pressure (water depth in meters)
Col(34) julian day number*100+hr
Col(35) minutes*100+seconds
Col(36) computed beam-c in units of 1/m
Col(37) D50 (median grain size microns)
Col(38) AVG (mean grain size in microns)
Col(39) STD (standard deviation of grain size in microns)
Col(40) total volume concentration of particles in microliters/liter
Col(41) water temperature (degrees C)
Col(42) tau (transmission ratio of laser power through sample versus background value of clean water, which is also known as the zscat)

The median particle sizes for each ring (the upper size of each ring is 1.18 times the lower) is as follows:

Ring 1 (col 1)	2.72 microns
Ring 2	3.20
Ring 3	3.78
Ring4	4.46
Ring5	5.27
Ring6	6.21
Ring7	7.33 microns
Ring8	8.65
Ring9	10.2
Ring10	12.1
Ring11	14.2
Ring12	16.8
Ring13	19.8
Ring14	23.4
Ring15	27.6
Ring16	32.5
Ring17	38.4
Ring18	45.3
Ring19	53.5
Ring20	63.1
Ring21	74.5
Ring22	87.9
Ring23	104
Ring24	122
Ring25	144
Ring26	170
Ring27	201
Ring28	237

Ring29	280
Ring30	331
Ring31	390
Ring32	460 microns

Data can be reduced in several ways. Figure 10 shows depth profiles from the same station that turbidity data is shown in Figure 9 (MGup3) for the three dates based upon grouping ring data in size classes (2-32 micron clay and fine silt, 32-62 micron coarse silt, 62-125 micron very fine sand and >125 micron fine to coarse sand). These designations do not reflect the fact that some (or all) sand-sized particles are actually flocculated muds. Figure 11 shows a plot of the total range of particle sizes for three depths at the same station in the April 2009 cast. Each point in Figure 10 can be examined in this detailed size range way.

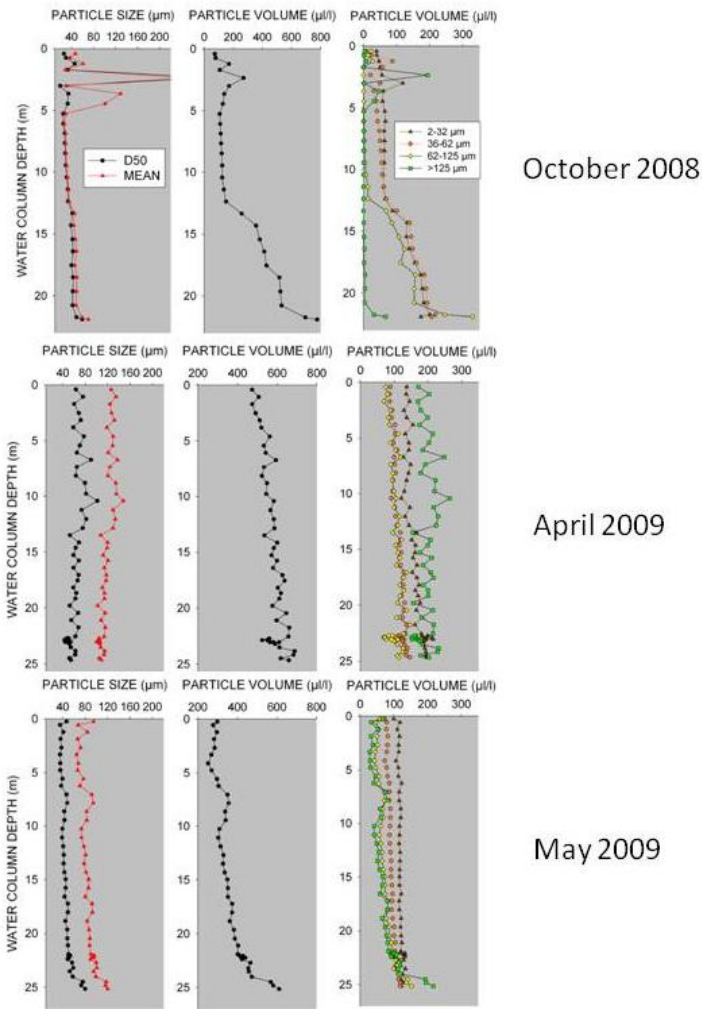

Figure 10. LISST water column mean and median (D50) particle size (left plot), total particle volume (middle plot) and particle volumes for size ranges (right plot) for station MGup3 during the three cruises. These size classes reflect a combination of discrete and aggregate particles.

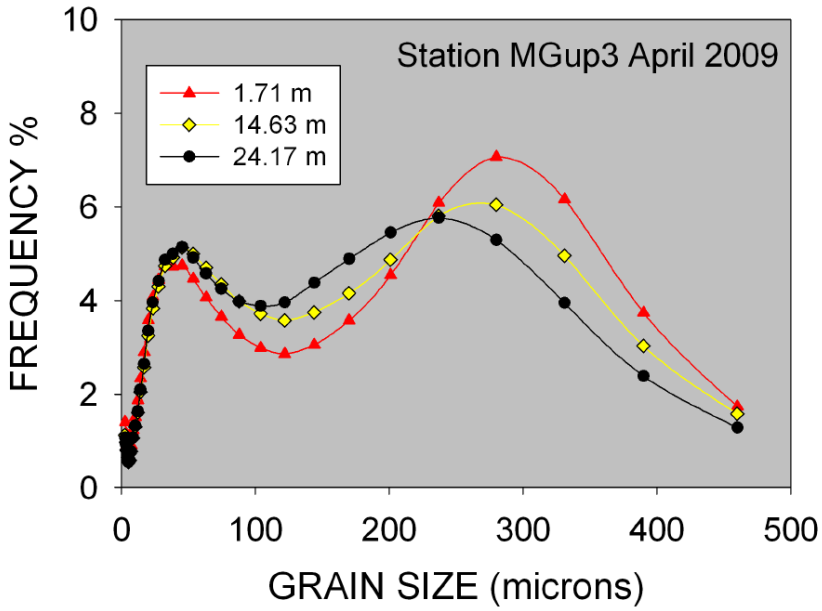

Figure 11. LISST grain size frequency plots for three depths at station MGup3 in April 2009.

9.0 REFERENCES CITED

Edwards, T. K., and G. D. Glysson, 1988. Field methods *U.S. Geological Survey Open-File Report* 86-531.

Nittrouer, J.A., Allison, M.A., and Campanella, R., 2008. Evaluation of bedload transport in the lower Mississippi River: implications for sand transport to the Gulf of Mexico. *J. Geophysical Research-Earth Surface Processes* 113, F03004, doi:10.1029/ 2007JF000795.

APPENDIX E

Myrtle Grove Delta Building Diversion: Numerical Modeling of Hydrodynamics and Sediment Transport in Lower Mississippi near Myrtle Grove River Bend

A report to the State of Louisiana
Office of Coastal Protection and Restoration (OCPR)

By

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October 2011

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1. Introduction

The focus of this study is to setup an appropriate computer flow model to optimize and analyze the design of a sediment diversion at Myrtle Grove, Louisiana. This diversion is authorized through the Water Resources Development Act (WRDA) of 2007 – Section 7006 (c) (1) (E). The formal title of this diversion is "Medium Diversion at Myrtle Grove with Dedicated Dredging" (MDMG) (Project Fact Sheet, 2010).

The project is located in the vicinity of Myrtle Grove, Plaquemines Parish, Louisiana. The proposed diversion is on the right descending bank of the Mississippi River (west bank). The intent of the project is to provide nutrients and sediment to the Barataria Basin. The project is to build land in the near-term through the use of dedicated dredged material. This land building capacity will be sustained in the long-run by diverting sediment from the Mississippi River.

2. Field Data

The Field data effort included two main components, setting up real-time monitoring stations, and gathering boat-based observations. The real-time data collection effort is described below. The boat-based field observations are fully described in "Interpretative Report on Water and Sediment Surveys of the Mississippi River Channel Conducted at Myrtle Grove and Magnolia in Support of Numerical Modeling (October 2008-May 2010)" report by Dr. Mead Allison.

The real-time field observations are intended to supplement the boat-based observations and provide an overall understanding of the dynamics of the Lower Mississippi River near the proposed Myrtle Grove Delta Building Diversion Project. The area of interest here is in Plaquemines Parish, Louisiana (Mississippi River Mile 55 to 65 Above Head of Passes). C.H. Fenstermaker and Associates established monitoring stations at three locations (Table 2-1 and Figure 2-1). These locations were selected to monitor stage, temperature, turbidity, and conductivity upstream and downstream of the Myrtle Grove Delta Building Diversion Project area.

Table 2-1: Myrtle Grove Diversion Study Area

Station ID	Location	Mississippi River Mile
1	Belle Chasse, LA	Mile 72.8
2	ConocoPhillips Alliance Refinery	Mile 63.2
3	Buras, LA	Mile 24.2

A water level, conductivity, and temperature Sonde (manufactured by YSI Inc - model number 600 OMS) was installed at each of the three locations shown in Figure 2-1 as yellow dots. Knowing the water temperature, salinities were calculated from the conductivity measurements. Table 2-2 lists specifications for the YSI Sondes.

Table 2-2: YSI Sondes Specifications

Type	Conductivity	Depth	Temperature	Turbidity
YSI Data Sonde	0 to 100 mS/cm	200 ft	-5 to 50°C	0 to 1,000 NTU

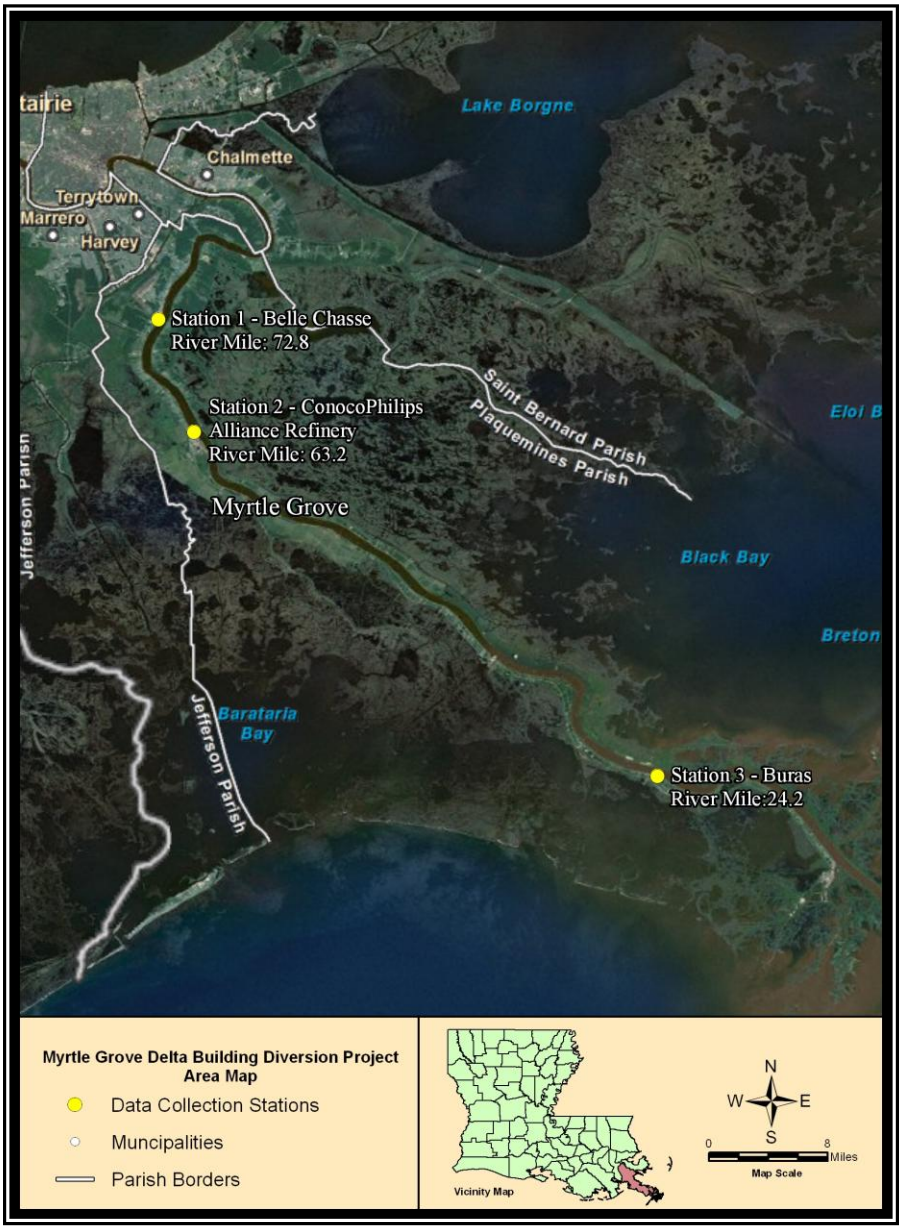

Figure 2-1: Myrtle Grove Diversion Study Area

Figure 2-2 shows stage data for all three stations along the Mississippi River. The data was quality reviewed prior to usage, as it should be for any modeling effort. Figure 2-3 shows the salinity record at the three monitoring stations.

Figure 2-4 displays raw turbidity data for the three stations. This unprocessed turbidity data required filtering due to large fluctuations observed in the raw data. A least square cubic spline function was implemented to process the raw turbidity records into a usable format (Figure 2-5). This filtering technique places a line of best fit through the data by minimizing the squared difference between the line and data points. This line has a set of spline breakpoints which represent the number of inflection (change in slope on straight line) points in the line of best fit.

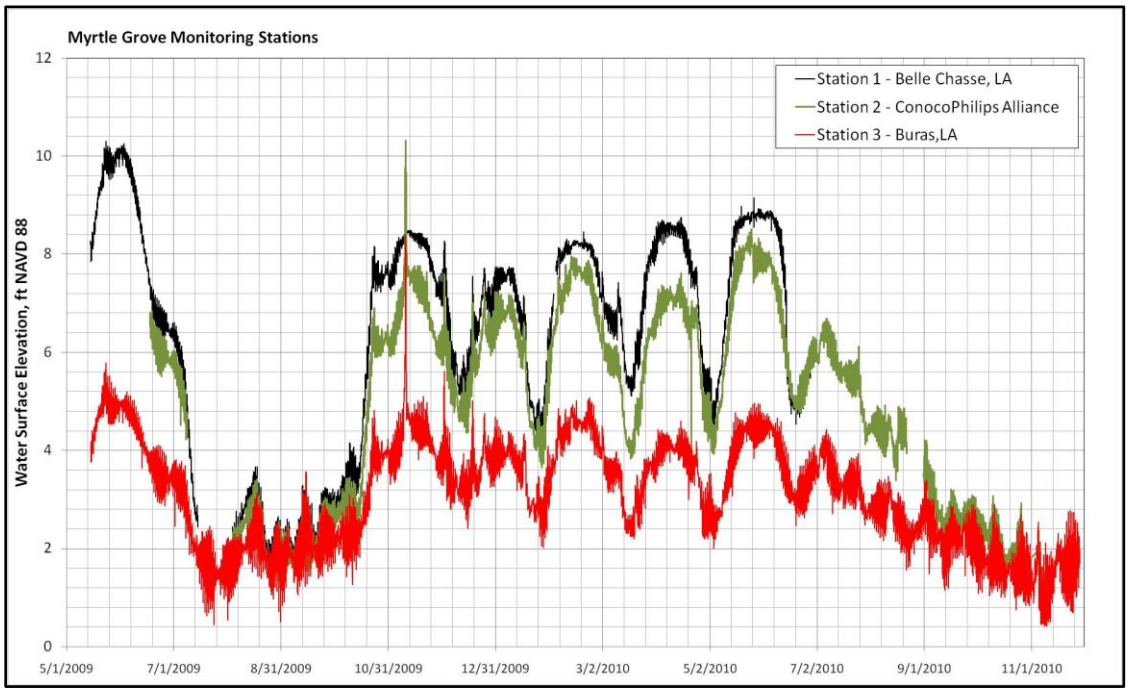

Figure 2-2: Myrtle Grove Diversion Stage Data

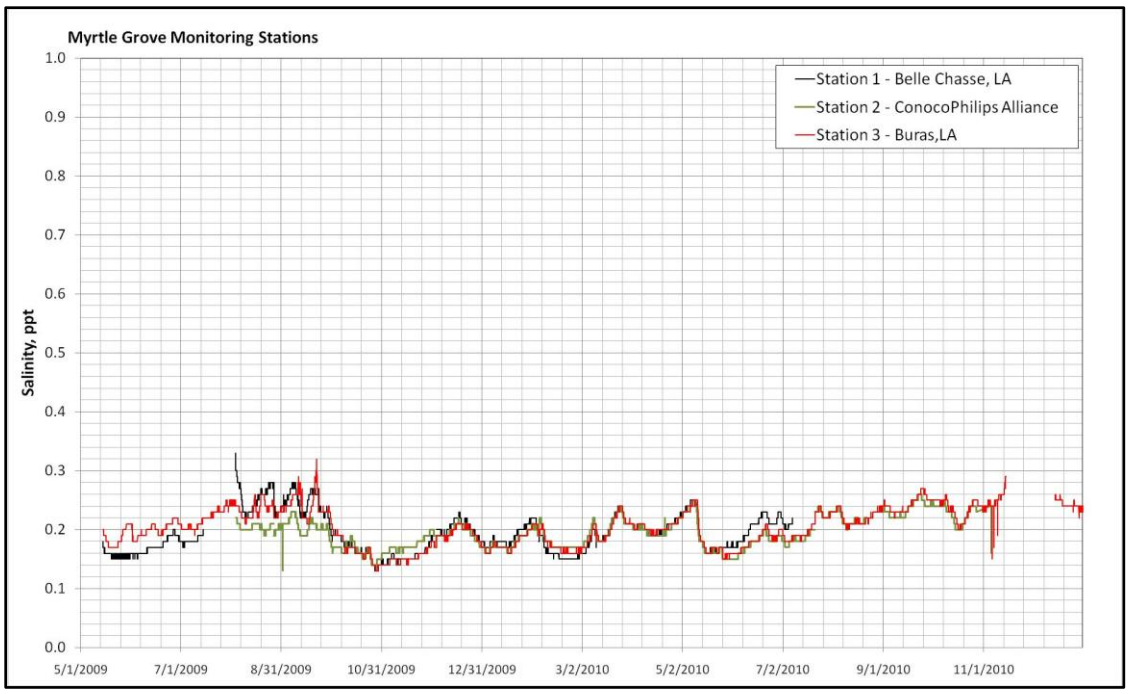

Figure 2-3: Myrtle Grove Diversion Salinity Data

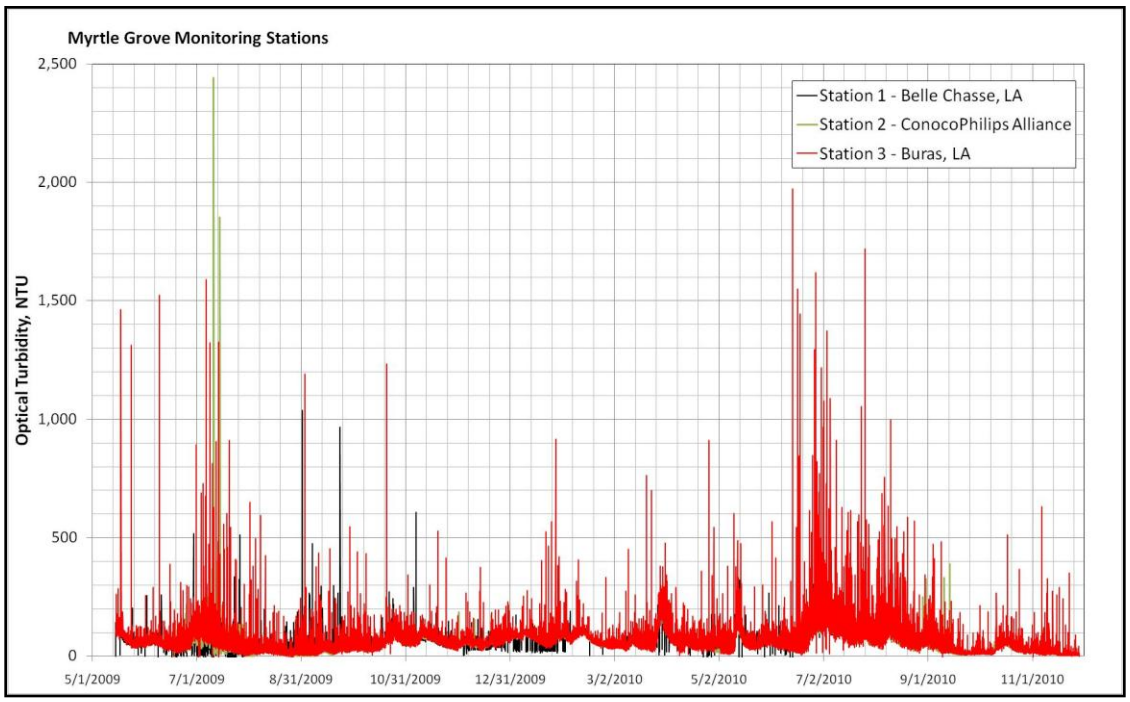

Figure 2-4: Myrtle Grove Diversion Raw Turbidity Data

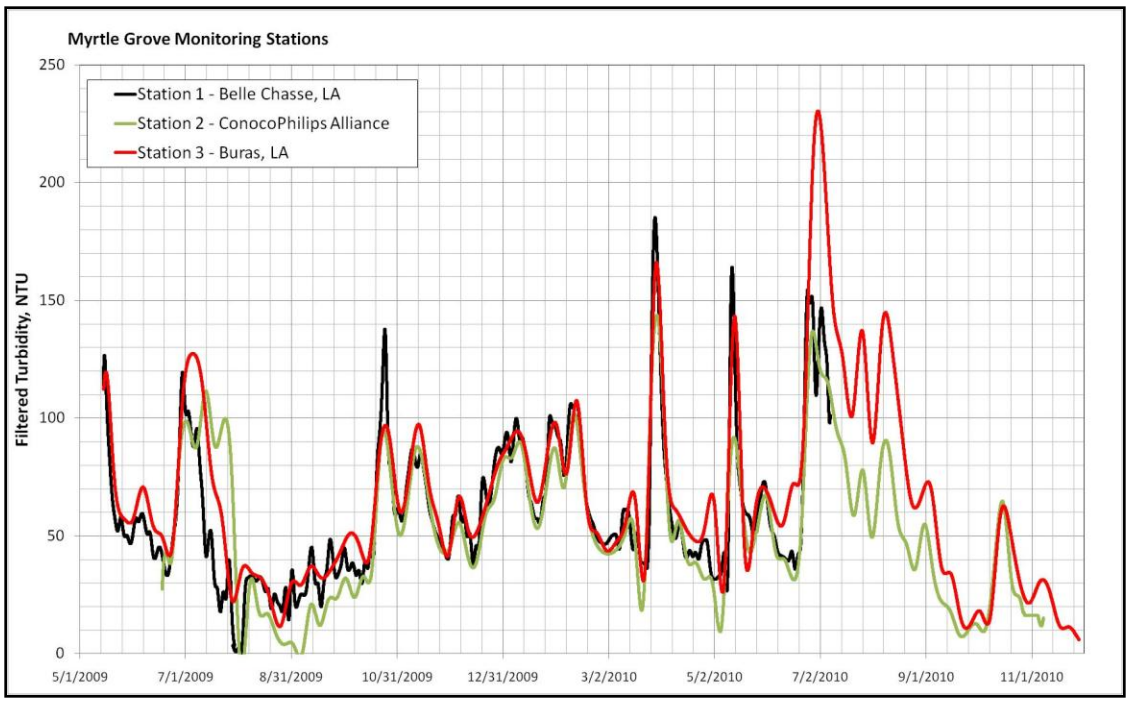

Figure 2-5: Myrtle Grove Diversion Processed Turbidity Data

3. Numerical Modeling

Although the main focus of this report is the three-dimensional modeling effort performed at the local scale around the Myrtle Grove project area, it should be noted that a large scale one-dimensional model was also used. The US Army Corp of Engineers code HEC-RAS (Hydrologic Engineering Center – River Analysis System) was used for this one-dimensional effort. The model extends from Tarbert Landing to the Head of Passes. Detailed information about how the model was setup, calibrated, and validated can be found in Davis (2010) and Meselhe et al (2008). The one-dimensional model was used to develop an overall understanding of the Lower Mississippi River at large temporal and spatial scales. It was also used to provide boundary conditions for the more detailed and local three-dimensional model.

The three-dimensional model is used is to develop an understanding of the complex flow field near the Myrtle Grove river bend and examine the interaction between flow and sediment. It is important to identify a model capable of capturing patterns such as the migration of sand near the bar on the west bank (upstream of the Myrtle Grove bend) and through the river bend then across toward the bar on the east bank (downstream of the Myrtle Grove bend).

The FLOW-3D model by Flow Science is suitable for this study as it has the adequate physical processes and computational efficiency. Further, the field observations were adequate to set up and calibrate the model described above.

Overall, the objectives of this modeling effort include:

- Capture the complex three-dimensional flow field in the vicinity of possible diversion sites.
- Quantify the sand load available for diversion under various flow conditions.
- Quantify the fraction of the available sand load that can be diverted for various conveyance channel alignments and configurations.
- Investigate the potential for shoaling and head-cutting as a result of sediment diversions.

3.1 Hydrodynamics:

FLOW-3D is a three dimensional model where fluid motion is described with non-linear transient, second-order differential equations, namely the Navier Stokes equations. This system of differential equations is solved numerically using algebraic expressions. The expressions provide a numerical solution for all flow variables at every node of the computational grid. Appropriate numerical approximations approach the analytical solution of the governing equations as the grid spacing is reduced. However, high-resolution grids are costly from computational point of view. Thus, a grid refinement exercise should be performed such that an accurate solution is reached while maintaining computational efficiency.

The numerical algorithm used in FLOW-3D is based on both finite difference and finite volume methods applied to a structured computational grid. Structured grids are known for their computational efficiency. The finite volume method used in FLOW-3D derives directly from the integral form of the conservation laws for fluid motion, and therefore, possesses the conservation properties.

The model is capable of capturing complex geometries. This is an essential feature due to the typical irregular river bottom bathymetry including ripples and sand bars, and irregular bank boundaries. The method used in FLOW-3D (called FAVOR™, Hirt and Sicilian, 1985) is an efficient approach that allows for capturing complex geometries while maintaining the use of structured computational grids. The general approach is illustrated in Figure 3.1. Essentially, the cell volume occupied by the fluid is captured, and the ratio of the volume occupied by fluid to the total cell volume is calculated. This ratio is referred to as the Volume of Fluid (VOF) and it is integrated into the mass conservation equations. As such, the geometry of obstacles (river bottom, bank lines, or hydraulic structures) is described through the

area fraction at all cell faces and the fluid volume fraction within the cell. In essence, FLOW-3D takes advantage of the computational speed and efficiency of structured grids while being fully capable of capturing complex geometries. Another advantage of using structured grids is the ease of generating the computational grid.

FLOW-3D is also fully capable of capturing the water free-surface accurately. Previously, FLOW-3D used the two-fluid Volume of Fluid (VOF) method to capture free-surface variations. However, this method may result in diffusion of the interface and is computationally expensive. The new approach adopted in FLOW-3D is called TrueVOF®. This approach computes the advection of fluid to all neighboring cells according to the orientation of the fluid within the cell under consideration. It does not compute the dynamics in the void region (Figure 3.2). Rather, it applies and uses pressure and velocity boundary conditions to capture the sharp free surface interface. This method allows for capturing the irregularity of the model domain boundaries, while taking advantage of the computational efficiency of a structured computational grid.

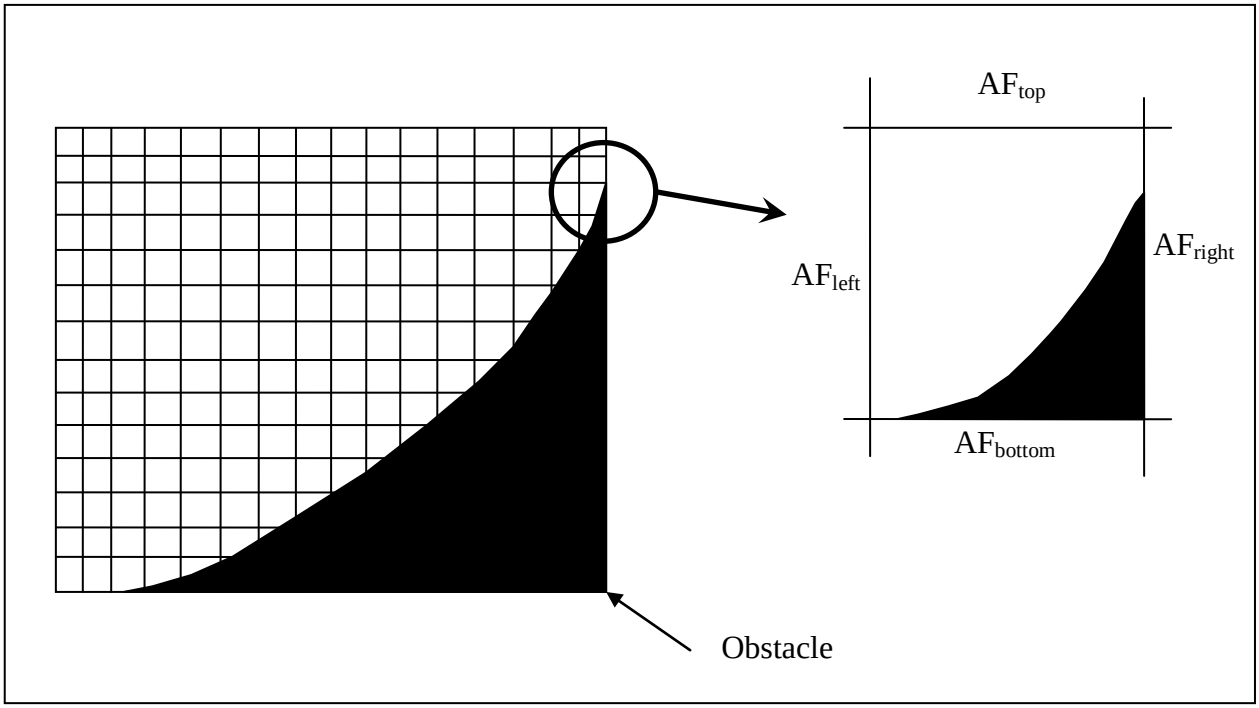

Figure 3.1: Method used in FLOW-3D (FAVOR™) to capture complex geometries. (AF: is the Area of each Face of a grid cell)

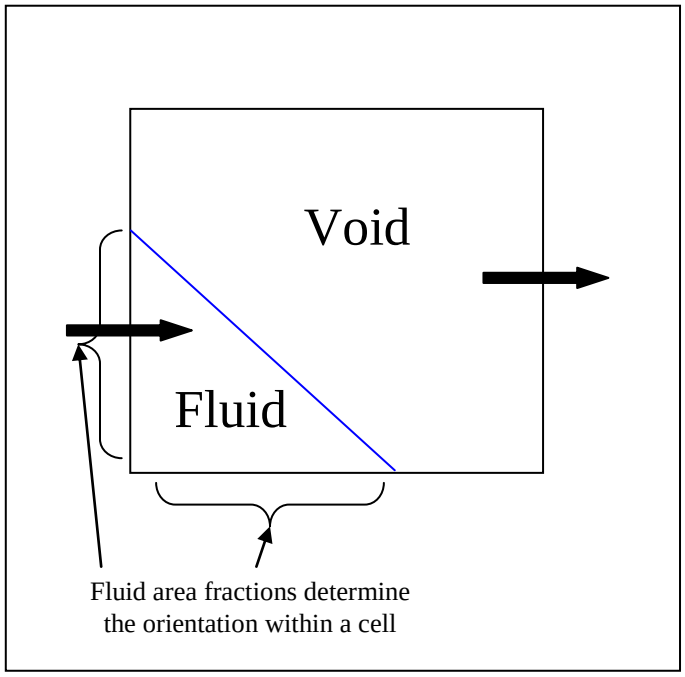

Figure 3.2: Volume of Fluid (TrueVOF®) approach in FLOW-3D to capture the free-surface

A brief summary of the governing equations used in FLOW-3D are included below. For more details, the interested reader is referred to (Flow Science, 2010, FLOW-3D user Manual).

Mass Continuity Equation:

$$V_F \frac{\partial \rho}{\partial t} + \frac{\partial}{\partial x}(\rho u A_x) + R\frac{\partial}{\partial y}(\rho v A_y) + \frac{\partial}{\partial z}(\rho w A_z) = R_{DIF} + R_{SOR} \qquad \textit{Equation 1}$$

Momentum Equations (Navier-Stokes):

$$\frac{\partial u}{\partial t} + \frac{1}{V_F}\left\{uA_x\frac{\partial u}{\partial x} + vA_yR\frac{\partial u}{\partial y} + wA_z\frac{\partial u}{\partial z}\right\} = -\frac{1}{\rho}\frac{\partial p}{\partial x} + G_x + f_x - b_x - \frac{R_{SOR}}{\rho V_F}(u - u_w - \delta u_s)$$

$$\frac{\partial v}{\partial t} + \frac{1}{V_F}\left\{uA_x\frac{\partial v}{\partial x} + vA_yR\frac{\partial v}{\partial y} + wA_z\frac{\partial v}{\partial z}\right\} = -\frac{1}{\rho}\left(R\frac{\partial p}{\partial y}\right) + G_y + f_y - b_y - \frac{R_{SOR}}{\rho V_F}(v - v_w - \delta v_s)$$

$$\frac{\partial w}{\partial t} + \frac{1}{V_F}\left\{uA_x\frac{\partial w}{\partial x} + vA_yR\frac{\partial w}{\partial y} + wA_z\frac{\partial w}{\partial z}\right\} = -\frac{1}{\rho}\frac{\partial p}{\partial z} + G_z + f_z - b_z - \frac{R_{SOR}}{\rho V_F}(w - w_w - \delta w_s)$$

Equation 2

Where V_F is the fluid volume fraction, r is density, u, v, and w are the velocities in the three primary directions (x, y, and z), R_{DIF} is a turbulent diffusion term, and R_{SOR} is a mass source, A_x, A_y, and A_z are the flow area fraction (at the cell face) in the x, y, z directions, respectively. The terms (x, y, & z) for G are body accelerations, f_s are viscous accelerations, b are flow losses across porous media, and ρ is water

density. The last term in the right hand side of Equation 2 is the mass injected into the flow field through general moving objects, where u_w, v_w, and w_w are velocity of the source component, and u_s, v_s, and w_s are the velocity of the fluid at the surface of the moving object, whereas δ define the pressure type of the source.

Turbulence Model:

One of the critical factors that determine the ability of a numerical model to capture complex flow patterns is the turbulence closure model used as part of the flow governing equations. FLOW-3D includes several turbulence closure models, namely Prandtl mixing length, One-equation transport, two-equation k-ε transport, Renormalized group theory (RNG), and Large eddy simulation, LES, models.

The turbulent kinetic energy can be expressed as follows:

$$k_T = \tfrac{1}{2}\left(\overline{u'^2} + \overline{v'^2} + \overline{w'^2}\right) \qquad \textit{Equation 3}$$

where *u', v', w'* are turbulent fluctuations in the x, y, z directions. The two-equation turbulent closure models are widely used due to their relative computational efficiency and adequate performance for wide-range of practical applications. The two transport equations are for the turbulent kinetic energy and its dissipation as expressed below (Harlow and Nakayama, 1967):

$$\frac{\partial k_T}{\partial t} + \frac{1}{V_F}\left\{uA_x\frac{\partial k_T}{\partial x} + vA_y\frac{\partial k_T}{\partial y} + wA_z\frac{\partial k_T}{\partial z}\right\} = P_T + G_T + Diff_{k_T} - \varepsilon_T \qquad \textit{Equation 4}$$

$$\frac{\partial \varepsilon_T}{\partial t} + \frac{1}{V_F}\left\{uA_x\frac{\partial \varepsilon_T}{\partial x} + vA_yR\frac{\partial \varepsilon_T}{\partial y} + wA_z\frac{\partial \varepsilon_T}{\partial z}\right\} = \frac{CDIS1\cdot\varepsilon_T}{k_T}(P_T + CDIS3\cdot G_T) + Diff_\varepsilon - CDIS2\frac{{\varepsilon_T}^2}{k_T}$$

Equation 5

Where PT is the turbulent kinetic energy production, GT is the buoyancy production term, Diff is the diffusion term, and CDIS1, CDIS2 and CDIS3 are dimensionless calibration parameters. For the simulations performed here, the Renormalization-Group (RNG) method (Yakhot and Orszag, 1986; and Yakhot and Smith, 1992) was used. This approach is similar to the popular k-ε model presented above in Equations 4 and 5. The RNG model applies statistical methods to the derivation of turbulent kinetic energy and its dissipation rate. The main difference between the RNG method and the *k*-ε model is that the constants (appearing in the governing equations of the turbulence model) are found empirically in the standard *k*-ε model whereas they are derived explicitly in the RNG model.

The RNG approach appears to have wider applicability than the standard k-ε model. The RNG model is more adequate for riverine applications with flows having strong shear regions. As such, it was selected for the Myrtle Grove study presented here.

3.2 Sediment Transport:

The traditional modeling approach for modeling suspended sediment is treating it as a state-variable using the advection-diffusion equation while including a fall-velocity term. Alternatively, an innovative approach was used here to simulate the transport of suspended sediment. A cluster of discrete mass particles were released at the upstream end of the model domain. These particles were assigned variable sizes and mass (ranging from silt; 32 Micrometer, to coarse sand; 250 Micrometer). This approach is appropriate and suitable to easily investigate the spatial distribution of every sediment size class in the river section. It tracks the amount of each sediment size class diverted through a certain structure design and configuration.

A brief description of the approach is presented here. The approach adopted allows for multiple mass particle species can be included. The fundamental equation used to model mass particles is shown below:

$$\frac{d\boldsymbol{u}_p}{dt} = -\frac{1}{\rho_p}\nabla P + \boldsymbol{g} + \alpha(\boldsymbol{u} - \boldsymbol{u}') + \beta(\boldsymbol{u} - \boldsymbol{u}')|\boldsymbol{u} - \boldsymbol{u}'| \cdot \frac{\rho}{\rho_p} \qquad \textit{Equation 6}$$

Where $u` = u_p + u_{diff}$, u_p and ρ_p are the particle mean velocity and density, respectively, g is gravity a other body forces, u and P are the surrounding fluid velocity and pressure whereas α and β are the drag coefficients divided by the particle's mass. The particle diffusion velocity, u_{diff}, is estimated according to a Monte Carlo technique described below. The drag coefficients, α and β are calibration coefficients. It is also possible to employ a variable drag coefficient (function of Reynolds number of the surrounding fluid).

Regarding the particle diffusion process, the approach adopted in FLOW-3D essentially adds an increment in position to each particle prior to establishing the new position of that given particle. In that sense, each particle is initially considered as a sediment point source. As time passes that source diffuses in all directions forming a sediment cloud with a Gaussian distribution. However, in the numerical simulation, each sediment particle remains a discrete entity. Thus the Gaussian cloud is mapped into a probability function used to transport the particle. The new position of the particle is calculated using a random number generator. Overall, a Monte Carlo approach is used to compute random shifts in the position of a given particle in each of the three coordinate directions.

There are two primary interaction mechanisms between suspended sediment and the surrounding fluid, namely, the momentum exchange and the volume displacement. The latter can typically be ignored since it is reasonable to assume that the volume concentration/displacement is small. For application where the relative difference between the particle density and fluid density is not large (such as the case with sediment and water) it is important to calculate the drag resistance to the particles as they move through the fluid. However, the total loss of momentum by this mechanism is small enough and does not have to be transferred back to the fluid. As such, it is possible (and desirable for computational speed) to compute the fluid motion then use the flow field to move particles.

4. Model Setup

The model domain extends from River Mile (RM) 56.0 (above the head of passes – AHOP) to RM 62.7 AHOP (Figure 4.1). As shown in Figure 4.2, this area has been surveyed using a high-resolution multi-beam survey (see Allison, 2010). This river reach encompasses the potential locations for the sediment diversion site.

Figure 4.1: FLOW-3D model domain from RM 56.0 to RM 62.7

The multi-beam data, collected by a boat, does not capture the shallow areas near the bank lines. Hence it was supplemented by the US Army Corps of Engineers decadal single beam surveys in order to have fully bathymetric information of this river reach. The over bank topographic data was captured by the LiDAR data. The three data sets, namely multi beam, single beam and LiDAR data, were combined as shown in Figure 4.3.

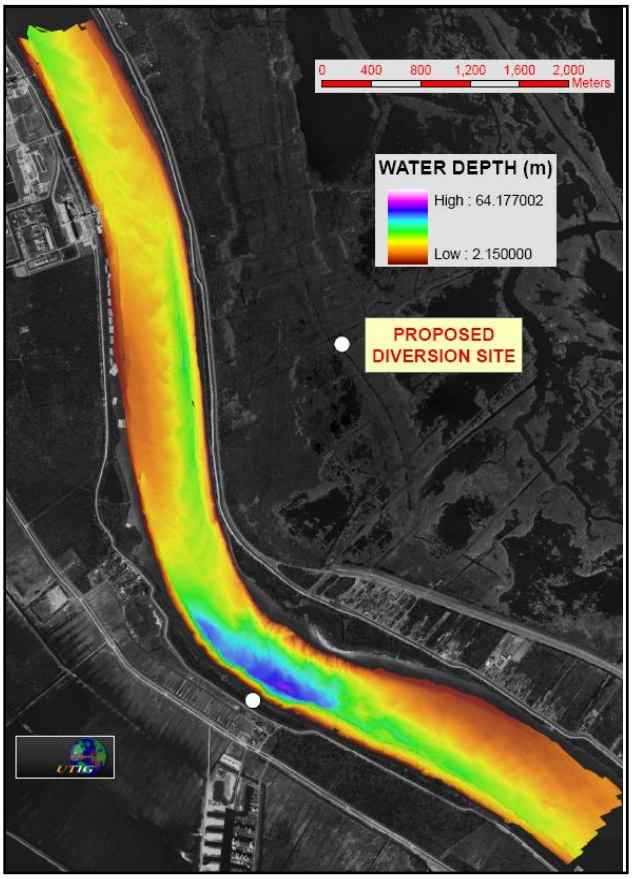

Figure 4.2: Multi-beam bathymetry map of the area of interest

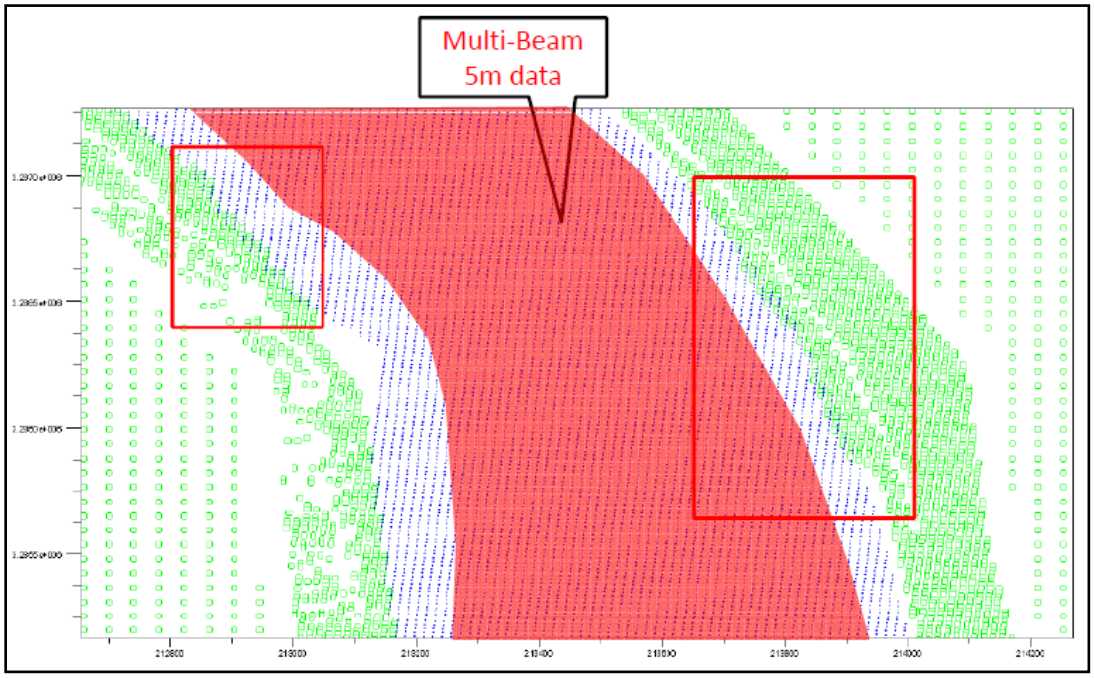

Figure 4.3: Supplementing the multi-beam data with single-beam, and LiDAR data (Red points: multi-beam; blue points: single beam; green points: LiDAR)

A coarse computational grid of 100 m X 100 m X 2.5 m in the (x, y, and z, directions respectively) was used as the starting point for the numerical simulations, where x, y, and z are the east-west, north-south, and vertical directions, respectively. This grid was used to establish a general flow field for the observed water discharges. A finer grid (50 m X 50 m X 2.5 m) was used to refine the flow field in the river reach under consideration. Finally a finer grid of 15 m X 15 m X 2.5 m was used to capture the details of the complex flow pattern observed in the field data. Comparison between the model results and the field data is presented and discussed later.

Boundary Conditions

The model was calibrated using the field data of April 2009. The water flow rate was used as the upstream boundary condition, while the tail water was used as the downstream boundary condition. The water flow during this field campaign was 700,000 cfs (19821.79 m^3/s). There is no tail water information available at the downstream end of the model domain. Numerical simulations using HEC-RAS as well as the measurements at the Conoco-Philips (RM63.2) station were used to estimate a tail-water elevation. The tail-water estimate was 6.23 ft-NAVD88 (1.9 m).

The model was validated against the April 2010 events. The water flow during the April 2010 event was 840,000 cfs (23,786 m^3/s). Again HEC-RAS simulations combined with information from the continuous station at Conoco-Philips (RM 63.2) were used to estimate the tail water. For April 2010, tail water in the range of 5.90 to 6.56 ft-NAVD88 (1.8 to 2.0 m) was used. The results of the calibration and validation simulations are presented in the following sections.

5. Model Calibration and Validation

Flows in rivers encounter resistance or drag proportional to the roughness of the banks and bed. This resistance has two components, namely form and skin drag. The form drag occurs when ripples or dunes exist on the river bottom. Skin drag is proportional to the bed-material and potential vegetation growth on the river banks or bed.

For the RNG turbulence transport models used herein, the roughness of the boundary is incorporated through boundary conditions for the turbulence quantities, namely the kinetic energy and its rate of dissipation. The wall-function approach is a common method to set these boundary conditions. A wall roughness coefficient in FLOW-3D is used as a calibration parameter (Flow Science, 2010). Through numerical experimentations done here, a range of 0.04 to 0.08 for the roughness coefficient proved reasonable for sand-bottom river applications. For a water flow of 700,000 cfs (19821.79 m^3/s), a value of 0.06 provided a water surface slope of 0.000016 (total head drop of 0.17 m across the length of the river reach modeled here). This compares well with a water surface slope of 0.000014 (total head drop of 0.15 m) estimated by a well calibrated and validated HEC-RAS simulation.

Field velocity profiles obtained from a vessel-based ADCP (shown in Figure 5.1) were compared to model derived velocity profiles at selected locations. These locations were selected to match locations where sediment transport measurements were conducted using isokinetic methods described earlier in the manuscript. During the calibration of the model, minor adjustments to the friction factor were conducted until a desirable water surface slope was achieved compared to previously calibrated one dimensional model and observations. During the velocity and sediment calibration, minor adjustments to the diffusion coefficient were conducted (for sediment); however, during model validation no additional adjustments of any parameters in the model were applied. For consistency with the field data report by Allison (2010), the sites used in the calibration and shown in Figure 5.1 are referred to as MGup, MGbend, and MGdown in subsequent plots. Their approximate locations are shown with solid white lines.

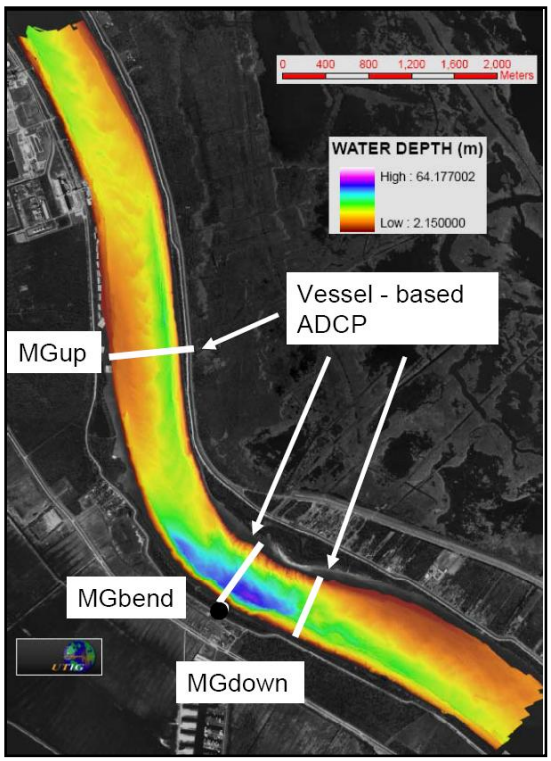

Figure 5.1: Locations were vessel based ADCP were used in the calibration (for consistency with the field data report by Allison 2010, the sites are referred to as MGup, MGbend, and MGdown in subsequent plots – see Allison 2010 for more details).

5.1 Preparation of Hydro-Acoustic Data

Velocities extracted from the vessel-based ADCP surveys were first analyzed by use of a data reduction algorithm. Data reduction was performed in both the horizontal direction (along a river transect) and in the vertical direction (from the water surface down to the river bed). The horizontal averaging was conducted on 10, 20, and 30 meter intervals, while the vertical averaging was performed every 1 m. Since the instrument collects data at 1 Hz, several horizontal ensembles are included in the horizontal averaging, while the 0.5 m bins in the vertical allow for averaging of 2 bins. The reduced data showed velocity trends more effectively and were more appropriate to use for comparison with model derived velocities. A schematic of the location and general methodology is shown in Figure 5.2.

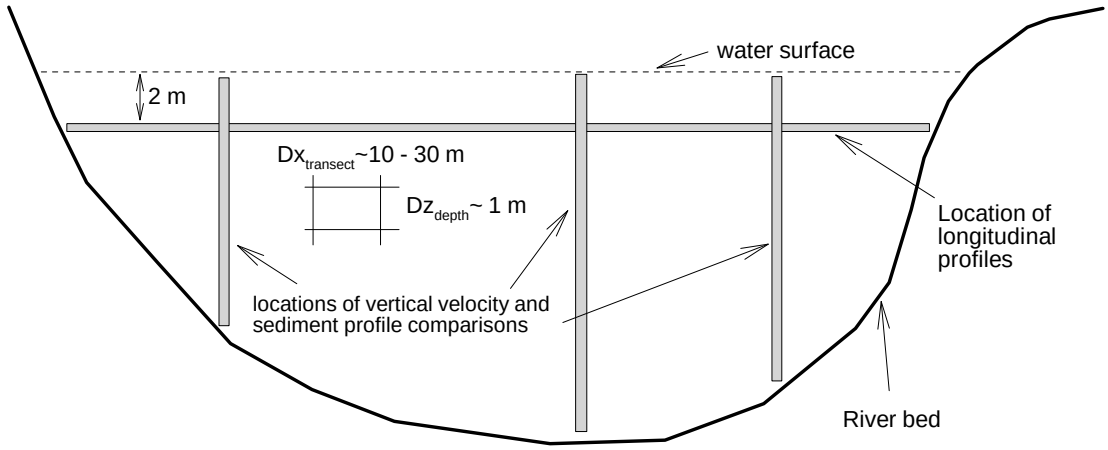

Figure 5.2: Schematic showing typical locations along a river transect where velocity and sediment data were compared during calibration and validation with field observations and the model

At each transect which ADCP data were collected, number of ensembles or number of observations along a river transect as n was defined. Afterward, using a 10 m window, an averaging process of the ensembles across the river was performed. In addition to the averaging process in the transverse direction, a second averaging process over the water depth every one meter or 2 bins, was performed such that:

$$\overline{U_{k,j}}, \overline{V_{k,j}}, \overline{W_{k,j}} = \sum_{k-1}^{40} \sum_{j=1}^{L} \left(\frac{U,V,W}{n,2} \right)$$

Equation 7

Where U,V,W are the east, north, and vertical velocities, L is the length of the transect (or river width), k is the number of bins available in the raw ADCP data (~ 40). Each summation term is divided either by (n) or by 2, which are respectively the number of ensembles contained in a 10 meter segment (horizontal direction), and 2 the number of bins used for the vertical averaging.

5.2 General Circulation Patterns

Once the water surface slope was calibrated, the model's ability to produce known and observed circulation patterns within the domain and study area was evaluated. These features are re-circulation eddies (near the inside of the meander bend) and secondary circulation which is typically along the meander (described further below). Figure 5.3 shows the re-circulation eddy near the east bank just downstream of the meander. The insert shows the map where the re-circulation zone takes place, and panel (a) shows the flow direction, (b) shows the location and (c) shows the size of this recirculation region as recorded in the field.

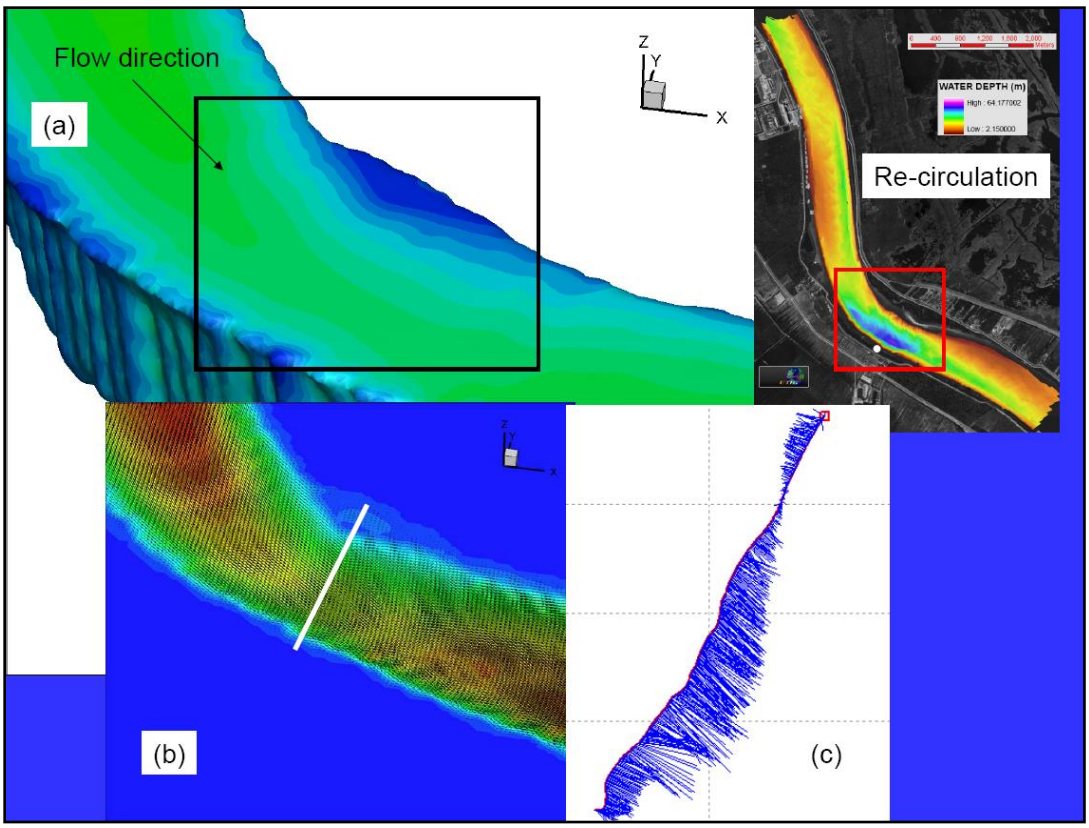

Figure 5.3: Model skill assessments on general circulation features. The model, (a) and (b) above, accurately captures the re-circulation zone observed at the meander near Myrtle Grove (c), where the near surface velocity vector (blue sticks) shows the reverse flow pattern (location of transect is shown in (b) with solid white line).

The other circulation feature used to test the model skill was secondary circulation. Secondary circulation is common near river meanders; capturing this was necessary in assessing the performance of the model. The model appears to capture the general secondary circulation fairly well. Figure 5.4 shows the model derived velocity magnitude (color) and direction (vector) showing secondary circulation at cross section in the meander bend near Myrtle Gove. The location of the cross-sections is shown on the right panel of Figure 5.4. The vectors shown (left) are tangent to the cross-sections, clearly showing the direction of secondary flow. The white solid line indicates conceptually the expected secondary circulation. The model is in good agreement with the expected secondary circulation.

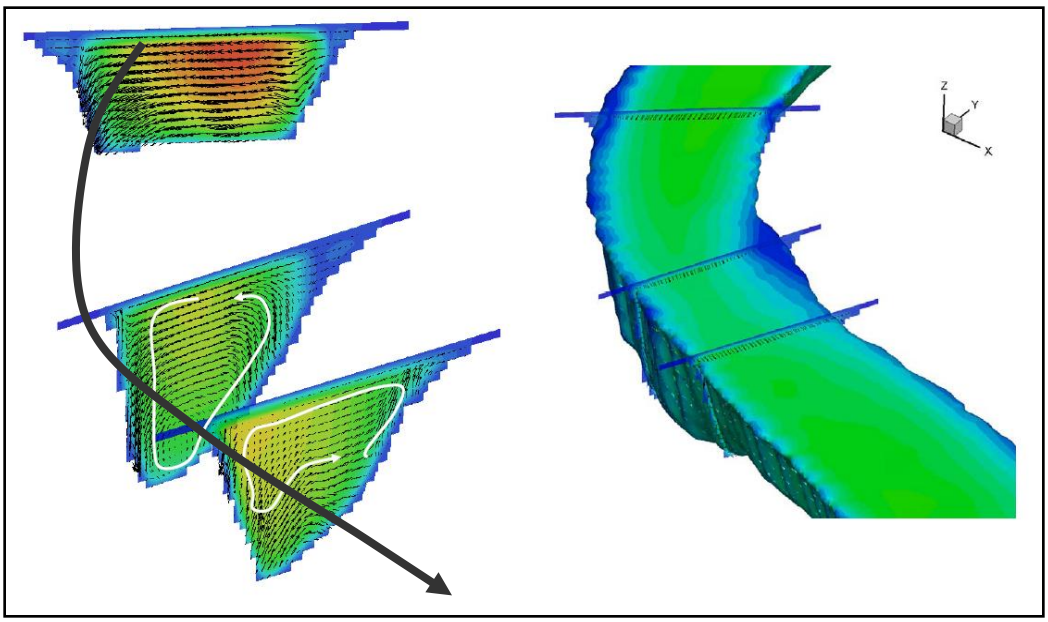

Figure 5.4: Model derived velocity magnitude (color) and direction (vector) showing secondary circulation at the meander bend near Myrtle Gove. The location of the cross-sections (left) is shown in the right panel. The vectors shown (left) are tangent to the cross-sections. The black solid vector indicates the dominant downstream flow vector, while the white solid line indicates conceptually the expected secondary circulation.

5.3 Model Calibration

5.3.1 Velocity Calibration

Velocity comparisons were performed at three locations, (1) upstream of the meander in the vicinity of a point bar (MGup), (2) near the meander in the vicinity of the deep hole (MGbend), and (3) downstream of the meander and the deep hole (MGdown). Figure 5.5 shows the velocity comparison for the calibration period of April 2009 for discharge of 700,000 cfs. Figure 5.5a shows the location of the ADCP transect, while Figures 5.5b and 5.5c show a horizontal velocity profile extracted at 2 meters below the water surface, and a vertical velocity profile extracted in the vicinity of the river thalweg at that location, respectively. The model clearly shows good performance, reproducing field velocities within 10 – 15 cm/s and well within the standard error of the observed fluctuations. A similar performance is shown in Figure 5.6, with a vertical and horizontal profile at MGdown. It can be seen in Figure 5.6 there is a large amount of scatter in the field observations at this location. This can be attributed to the high level of turbulence and secondary motion due to the deep hole in the outside bend of the channel and bend curvature. The performance is considered satisfactory given the flow complexity at meander bends.

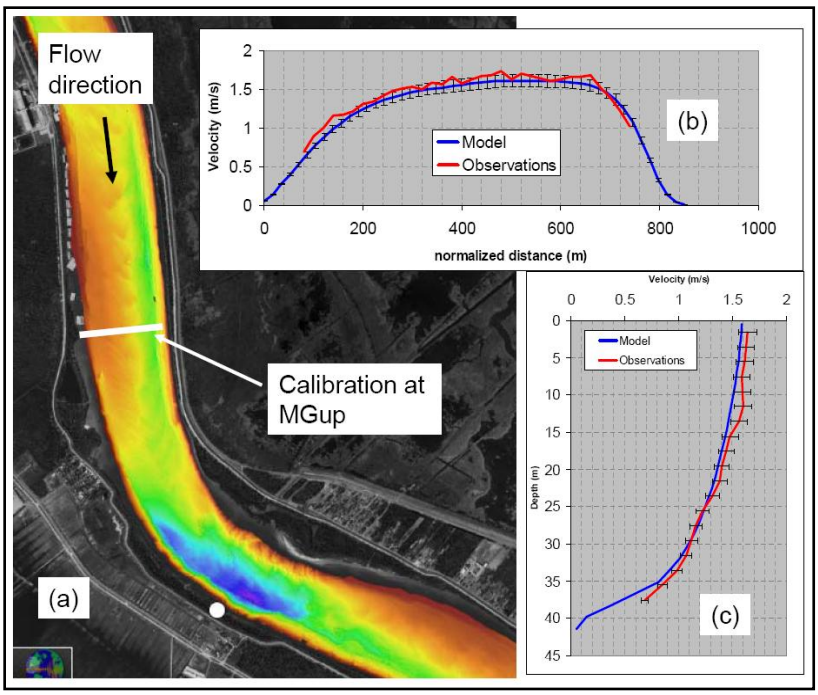

Figure 5.5: Calibration for velocity during April 2009 flows at location upstream of the meander (MGup). (a) location of ADCP data; (b) model compared to observations along a horizontal profile extracted at 2 m depth, and (c) model and observations for a velocity profile near the river thalweg.

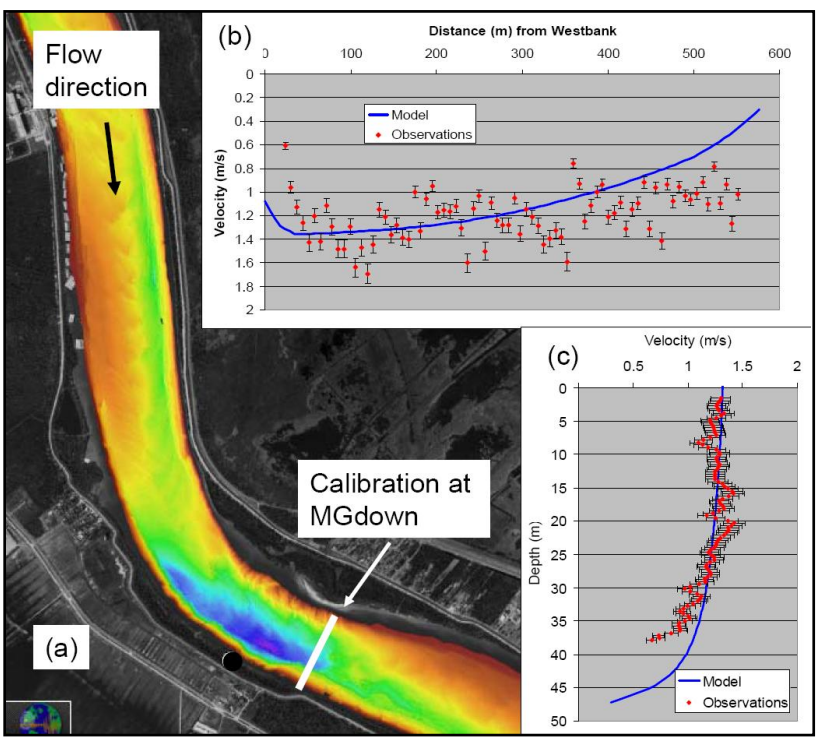

Figure 5.6: Calibration for velocity during April 2009 flows at location downstream of the meander (MGdown). (a) location of ADCP data; (b) model compared to observations along a horizontal profile extracted at 2 m depth, and (c) model and observations for a velocity profile near the river thalweg.

5.3.2 Sediment Calibration

The transport of sediment within the domain utilizes discrete particle transport via a non-passive lagrangian method through an upstream release along the model boundary. At steady state, as many as 500,000 particles are tracked within the domain, which implies a dynamic equilibrium state. Particles are weakly coupled to the flow field, which implies that the particles (in their respective classes) are treated as individual particles, and can affect the flow field by exchanging momentum with the moving fluid. As the particles can affect flow, the flow also affects particle motion. Thus the model produces a realistic representation of the particle density, especially for regions with complex flow field and interaction with solid structures.

For a given number of particles in the domain (P_n), among a number of size classes (m) the sediment reference concentration within the domain would be a function of the total volume of fluid (V_p) and the particle density such that

$$C^m{}_{ref} = P_n V_p \rho_p \qquad Equation\ 8$$

where C_{ref} is the reference concentration for a number of size classes (i), P is the number of particles, V is the volume of particles and ρ is the particle density. To obtain the reference sand concentration, $C^s{}_{ref}$, Equation 9 has been used for m classes of sand size particles.

$$C_{ref}{}^s = \sum_{i=1}^{m} (C_{ref}) \qquad Equation\ 9$$

Hence the normalized concentration is

$$Cs = \frac{C_{ref}}{V_w} = X \qquad Equation\ 10$$

Where C_s is the sand concentration, C_{ref} is the reference concentration, and V_w is the domain water volume. The term, X, is used to derive a conversion factor to transform the model output into concentration directly. The final conversion factor is proportional to the sediment load, which itself is a function of flow intensity. As such, since the sediment load and total water flow have been measured for the calibration and validation events used here, the term X can be easily calculated. For the sediment load used during calibration, this number is 56.7.It is largely derived by a conversion of the total sediment load to the model domain from tons/d to mg/L. Using these steps, the resulting conversion factor is then used to convert the particle density to a sediment concentration.

The lagrangian method used to predict sediment transport utilizes a diffusion coefficient (D), which is often used to predict the dispersive nature of individual particles in natural systems. This coefficient is similar to any other diffusion coefficient in the advection/diffusion equation, and was used in the calibration to produce good model agreement. For the calibration process, the diffusion coefficient was varied from 0.05 – 0.1. To determine the proper diffusion coefficient, three scenarios were run, where D = 0.05, 0.075, and 0.1. Once the value which produced satisfactory results was selected, the coefficient remained constant for the validation process. The results from all three simulations are shown in Figure 5.7 for upstream of Myrtle Grove at MGup, and for downstream of the bend at MGdown. The convention for the plot is left cast is near the westbank, and right cast is near the eastbank. It can be clearly seen in Figure 5.7 that the model has a very good ability to reproduce the sand concentration for the April 2009 flow. A small departure is seen between the three simulations with different diffusion coefficients, except for the left cast at MGdown. The observations at this site fall within the model simulated concentrations and are therefore considered satisfactory.

While overall the sand concentrations are considered satisfactory, there are a couple of points that deserve some discussion. The surface concentrations of sand are predicted fairly well (except for the middle cast of the MGdown transect) while the bottom concentrations are often in slight disagreement between observations. The diffusion coefficient that agreed with observations at the most locations was selected for use in the final simulations. Figure 5.7 (e) shows that the model did not accurately capture the observed concentration of sand in the middle of the MGdown transect; however, as evident from Figure 5.7 (d), and (f), the concentrations on either side of that transect are reproduced very well. Furthermore, the small disagreement between the model and observations near the bed at Figure 5.7 (a) appears not to be a concern since after the transition through the meander, the concentrations are reproduced. It is worth mentioning to the reader that near the bed the uncertainties in observations due to the influence of turbulent fluctuations of the bed material load are much higher.

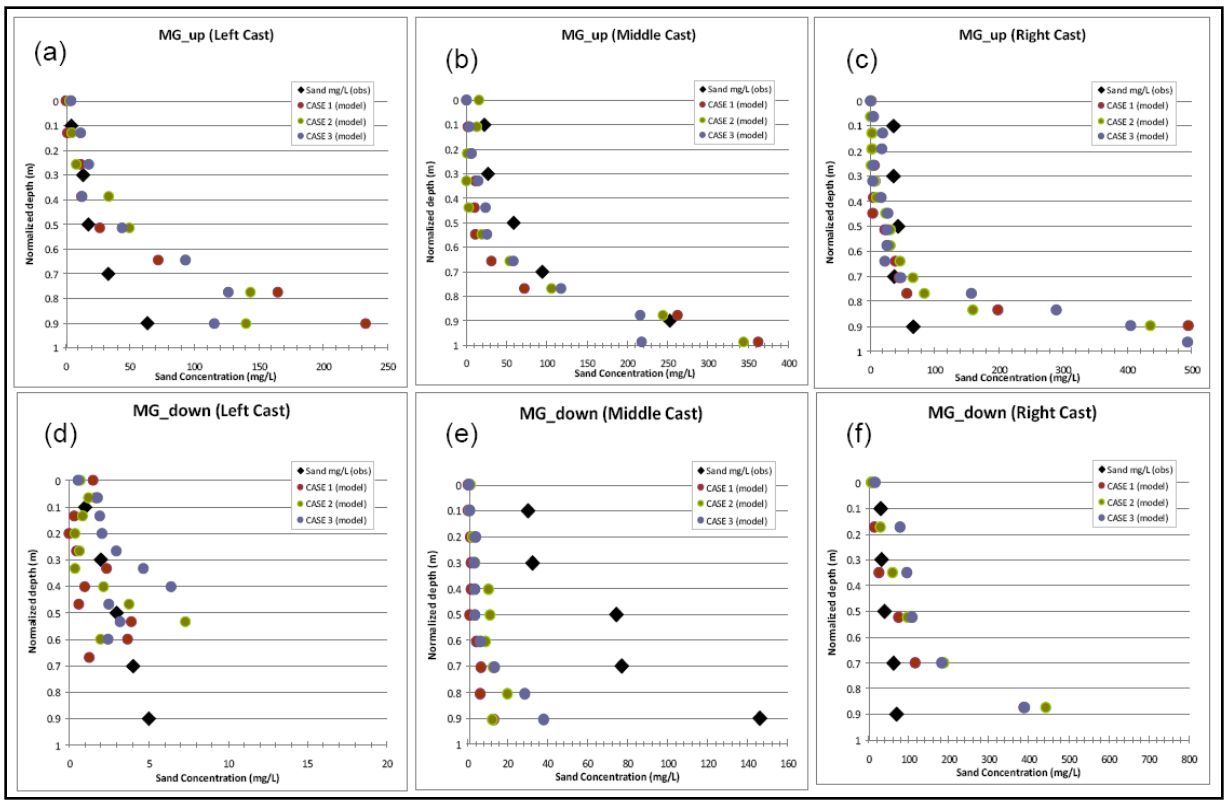

Figure 5.7: Suspended Sand comparison between Model (FLOW3D) and observations (Isokinetic P63) at MGup (upstream of the meander) and MGdown (downstream of the meander) for April 2009 flows during model calibration. Diffusion coefficients for lagrangian methods are: Case 1 (D - 0.05), Case 2 (D - 0.1) and for Case 3 (D – 0.075); Convention is: Left cast is near the west-bank, and right cast is near the east-bank.

5.4 Validation

The spring flood of 2010 was selected to perform a validation of the model for velocity distribution and sediment transport. The date of the validation was mid April (field data were available from April 13th through the 16th) in 2010. Not all locations of the model domain were occupied during the April 2010 field surveys; hence comparisons with observations are shown here for MGup location only. During the validation, no adjustment of any parameter was performed.

5.4.1 Velocity Validation

Velocities (both vertical and horizontal profiles) were extracted at the locations where field data were available and compared to model simulated velocities. Figure 5.8 shows the velocity comparisons between observations in April 2010 during stationary ADCP surveys. The surveys have approximately 15 – 30 minute long sampling periods and approximately 800 – 1700 data-points of velocity for each 0.5 m

depth (0.5 m of depth equals one bin). The panels (a) through (c) in Figure 5.8 show these velocities as points, while the error bars show the standard deviation for each bin for the entire sampling period. It is evident that the velocity profiles are fairly stable, and generally exhibit different variance. For instance, at the shallower MGup 1 location, towards the westbank over the point bar, there is more variance (as indicated by the error bars) near the surface of the water compared to the thlaweg at site Mgup 3 where ADCP data show higher variance near the bed. Generally however, there is a standard deviation of at least 0.2 - 0.4 m/s.

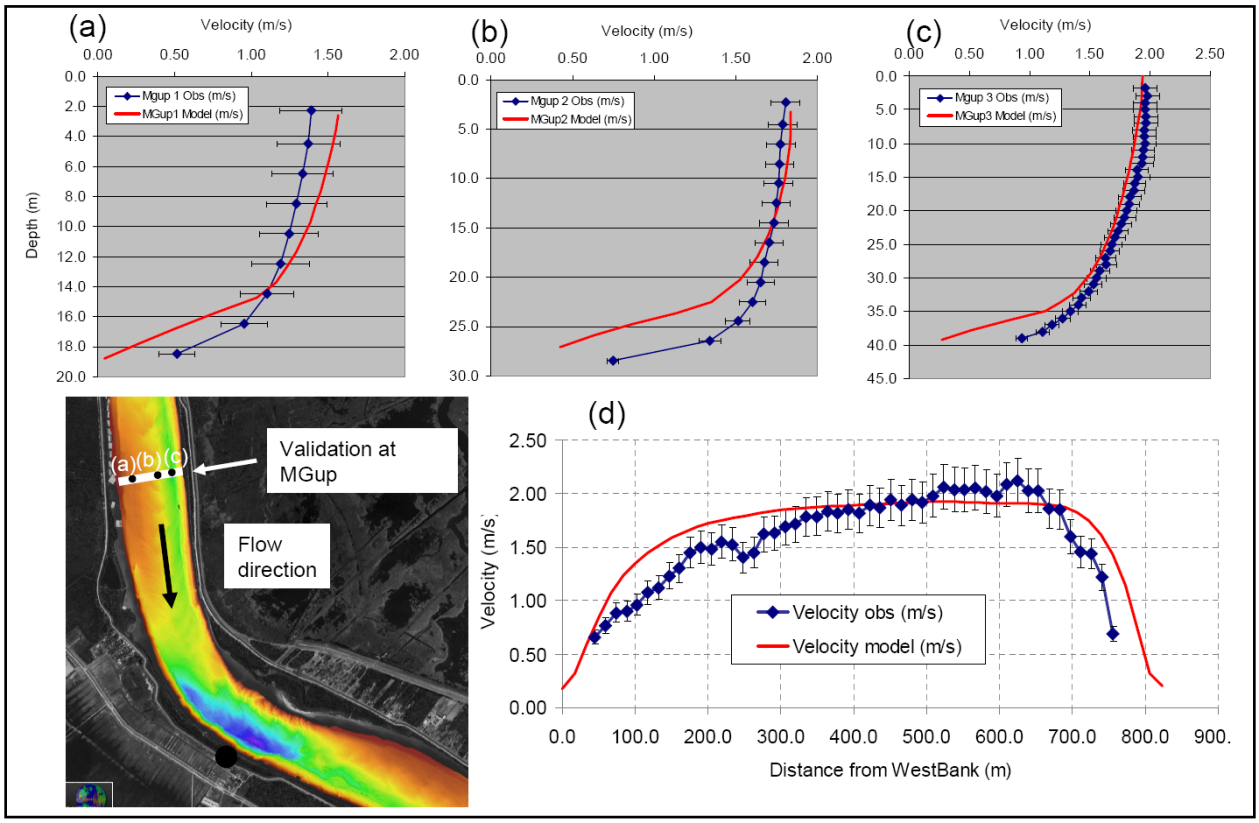

Figure 5.8: Velocity validation during the spring April 2010 flows. Panels (a) through (c) are from time-averaged stationary ADCP surveys at MGup location 1, 2 and 3 (from Allison 2010) and shown in solid circles in regional view. Panel (d) shows the horizontal velocity comparison between the model and observations at 2 m below the water surface. Data for this comparison were from ADCP transect surveys (from Allison 2010).

The model's performance during validation appears to be satisfactory and generally within or near the edge of the uncertainty or dispersion observed in the field. For instance, as can be seen in Figure 5.8a, the model slightly under predicts the velocity near the bed, and agrees well with the measurements at depths of approximately 12-16 meters, but the predictions near the surface tend to over predict by 0.2 m/s. A slightly different behavior is seen at the center point (MGup2 - Figure 5.8b) where the surface velocity

are in very good agreement with observations, while near the bottom the model slightly under-predicts the velocity field. In the thalweg of the river (MGup3, Figure 5.8c), the model is in very good agreement with observations with a small under-prediction of approximately 0.05 m/s. This is verified further by comparison of the horizontal velocity profile shown in Figure 5.8d. It can be clearly seen here that the edge velocities near the banks compare very well with observations, while the velocity field near the deeper part of the river is generally in agreement except for a slight under-prediction in the deepest part of the channel. This is consistent with the velocity profile shown in Figure 5.8a, and b. Overall however, both vertical and horizontal velocity profiles agree well with observations and are satisfactory.

5.4.2 Sediment Validation

A similar methodology to that previously described during calibration was utilized to process, analyze and extract sediment concentrations for use in comparisons with model predictions. Figure 5.9 shows the suspended sediment concentration predicted by the model during the validation period (red triangles), compared to field observations (blue diamonds) collected at the upstream cross-section at (MGup sites). The model's ability to reproduce observed concentrations appears satisfactory, with good agreement throughout the water column at all three sites. There is a small over-prediction of sand concentrations near the bed at MGup 1 (over the point bar), as can be seen in Figure 5.9a, but the model does well in predicting sand concentrations for 90% of the water column. At the center of the river (site MGup 2 – Figure 5.9b) the model is predicting sand concentrations very well near the surface and near the bed, but slightly under-predicts at around 30% depth. This however, is potentially due to a higher uncertainty in the field observations as shown by a sudden increase in sand concentration at the 30% depth. At MGup 3 (Figure 5.9c) a similar performance by the model and a similar increase in the observed suspended sand concentrations is seen around 30% of the water column. Besides this disagreement at this depth, the model skill in predicting sediment concentrations throughout the water column is very reasonable. It is important to mention that there are no duplicates isokinetic sampling or multiple samples at any locations, and therefore difficult to assess the field uncertainty. However, from Allison (2010) the expected uncertainty is proportional to the river turbulence, flow depth, and many other parameters, and can be of the order of 5 – 10 % of the measured concentrations.

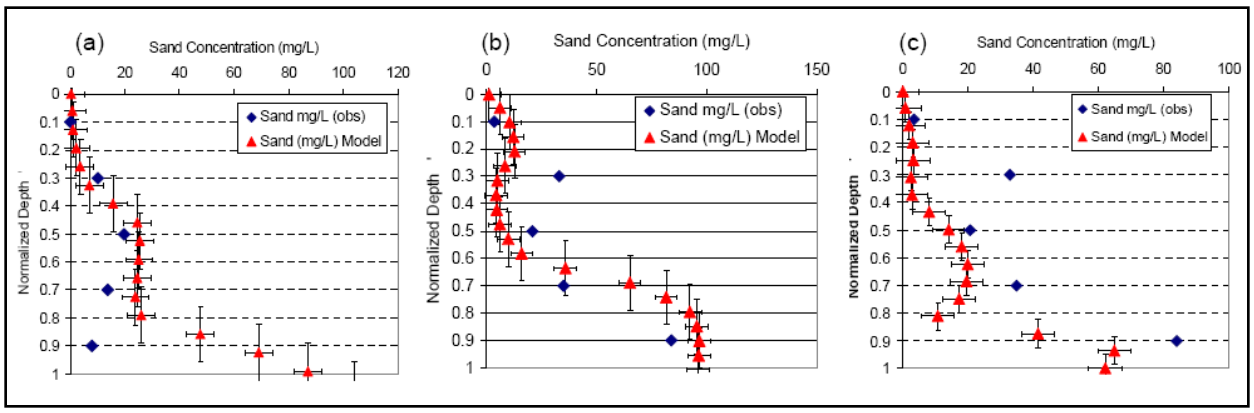

Figure 5.9: Suspended sediment (sand only) validation during the April 2010 flows. The field data were collected using P-6 sampler on April 14, 2010 (Allison 2010). The locations are respectively (a) MGup1, (b) MGup 2, and (c) MGup3.

6. Diversion Scenarios

Once calibrated and validated, the model was used to test various diversion designs. The numerical experimentations included various alignments, intake locations, and size of the intake channel and diversion structure.

For cross section details see the report "*Conceptual Designs Myrtle Grove Sediment diversion Structure River Mile 60.2 AHP – Volume II Revised with Appendix C and Appendix D*" prepared by BCG Engineering & Consulting, Inc. It should be noted that the diversion structures are designed for a maximum capacity of 15,000, 45,000 or 75,000 cfs when the river is flowing at 1,000,000 cfs or higher. The river flow conditions tested for a flow discharge of 700,000 cfs and as such, the diversion structures were operating at lower than the design capacity.

6.1 Original (US Army Corps of Engineers) Alignment (RM 60.2) [OA-RM60.2-15K]:

The general layout of the design is shown below in Figures 6.1 and 6.2. This design is intended to divert a maximum of 15,000 CFS when the Mississippi River discharge is at 1,000,000 CFS. A HEC-RAS model was used to design the geometry (size and slope) of the diversion channel such that it would carry a flow of 15,000 CFS. It should be noted that the HEC-RAS did not include (explicitly) the energy losses due to bends along the length of the diversion channel and due to the transitions from one cross section shape/size to another.

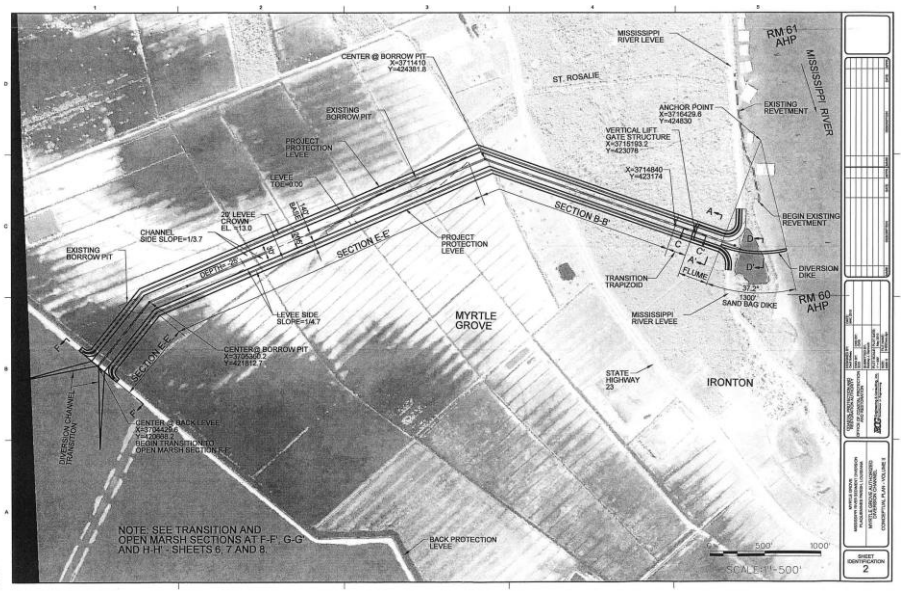

Figure 6.1: Layout of the Original (USACOE) Alignment – RM 60.2 – 15 K CFS [OA-RM60.2-15K]

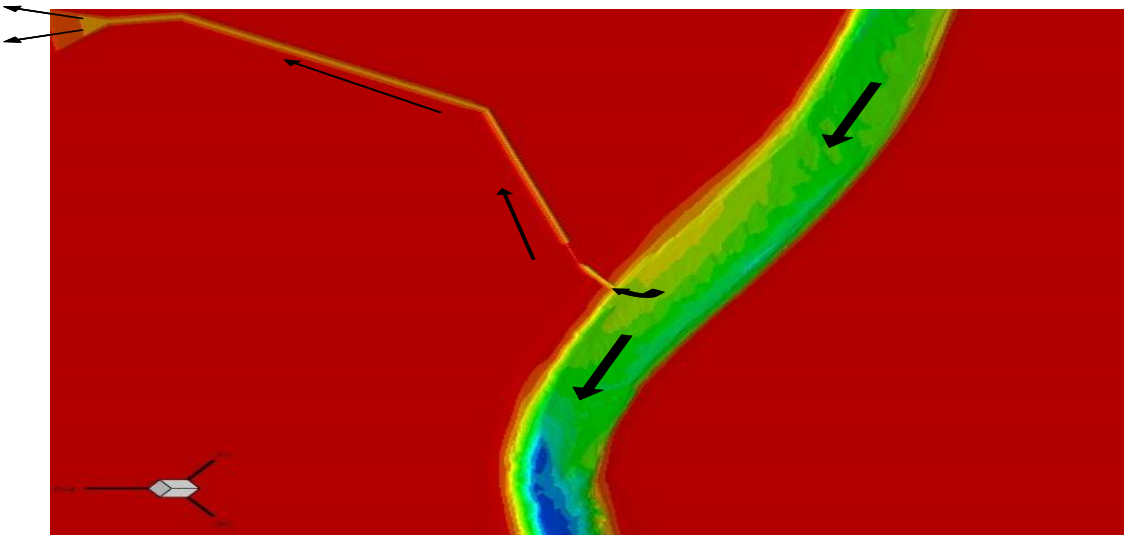

Figure 6.2: Image showing the diversion channel in the numerical model [OA-RM60.2-15K]

The April 2009 event (same event that was used to calibrate the model) was used to assess the performance of this design. A tail-water elevation of 5 ft-NAVD88 (1.524 m) was set at the downstream end of the diversion channel (bay side). This estimate reflects typical conditions observed in the bay side. The mouth of the intake channel at the river side is set at -40 ft-NAVD88. The channel gradually slopes up. At the release point at the bay side, the channel invert is -25 ft-NAVD88 (-7.62 m). Immediately after the entrance from the river side, the channel has rectangular cross section with a width of 30 ft (9.14 m). The channel then transitions into a trapezoidal cross section (bottom width = 30 ft (9.14 m), and side

slope of 3:1 (H:V). The channel has two bends as shown in Figures 6.1 and 6.2. The channel cross section transitions into a trapezoidal cross section with bottom width = 30 ft (9.14 m), and a compounded side slope of 3.7:1 and 4.7:1 (H:V). For cross section details see the report "*Conceptual Designs Myrtle Grove Sediment diversion Structure River Mile 60.2 AHP – Volume II Revised with Appendix C and Appendix D*" prepared by BCG Engineering & Consulting, Inc.

A nested grid (multi-grid) approach was used to capture the flow field within the narrow intake of the diversion channel. Figures 6.3 through 6.5 show the details of the flow field as captured by the numerical model. The energy losses captured by FLOW-3D due to the bends and cross section transitions resulted in a reduction of the diversion channel capacity. For the river conditions used in this case (River Flow = 700,000 CFS (19,821 m^3/s), River tail-water = 6.23 ft-NAVD88 (1.9 m), diversion channel tail water = 5 ft-NAVD88 (1.524 m)), the flow captured by the diversion channel was 11,369 CFS (322 m^3/s).

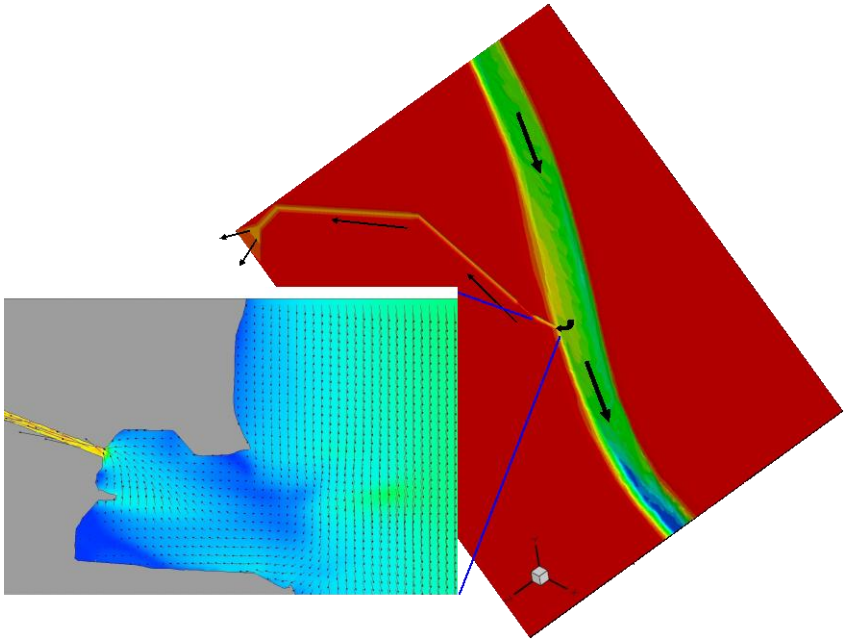

Figure 6.3: Water drawn into the intake of the diversion channel [OA-RM60.2-15K]

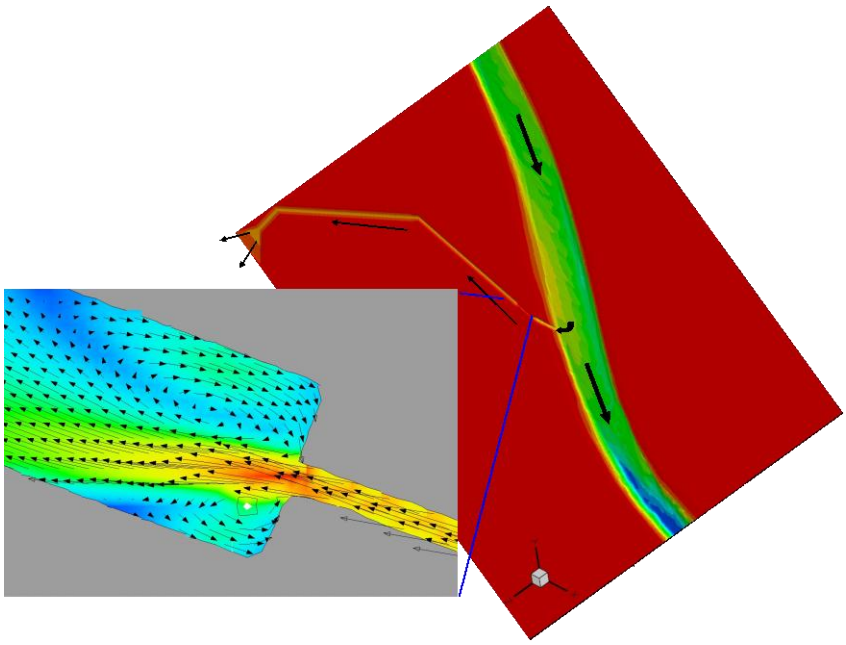

Figure 6.4: Details of the flow field at transition zone from the rectangular segment of the diversion channel into the trapezoidal segment [OA-RM60.2-15K]

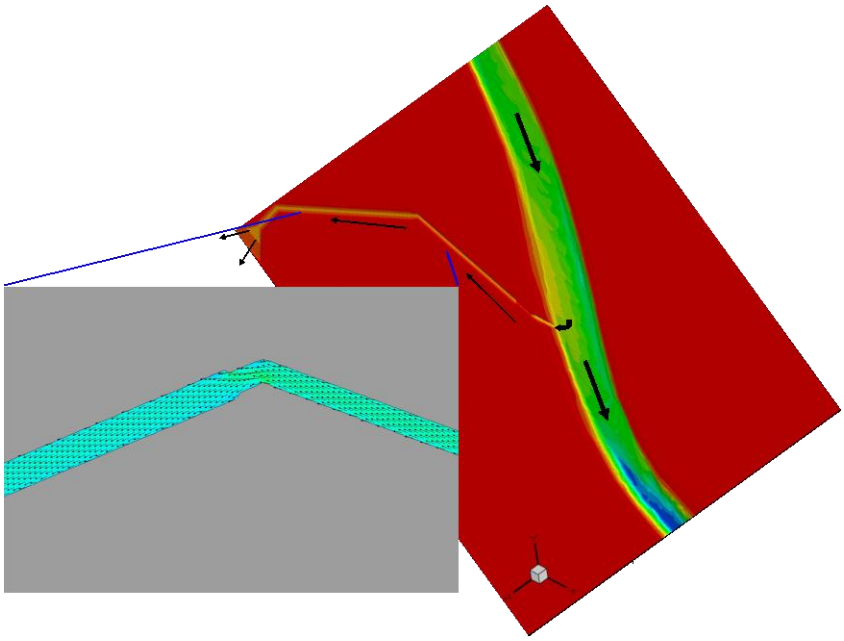

Figure 6.5: Details of the flow field at the 120 bend within the diversion channel [OA-RM60.2-15K]

As indicated previously, the sediment modeling approach used here is suitable to assess the ability of the diversion channel to capture various sediment size classes. The sediment load for every size class that was captured in the diversion channel has been calculated. A summary of the sediment load captured in the diversion channel is shown in Table 6.1.

Table 6.1: Summary of the sediment load (for every size class) captured by the diversion channel OA-RM60.2-15K.

	Mississippi River (Main Stem)	Diversion Channel OA-RM60.2-15K	Ratio
Water Discharge (m3/s)	19,821	322	1.62
Water Discharge (CFS)	700,000	11,369	1.62
Sediment Load (metric tons/d) - 32 Micron	233,539	2,786	1.19
Sediment Load (metric tons/d) - 63 Micron	10,839	104	0.96
Sediment Load (metric tons/d) - 96 Micron	21,816	144	0.66
Sediment Load (metric tons/d) - 125 Micron	34,437	133	0.39
Sediment Load (metric tons/d) - 250 Micron	23,460	2	0.01
Total 63 - 250 Micron Load (metric tond/d)	90,554	383	0.42

The main objective of this diversion project is to build land. Thus, it is a goal to maximize the sediment load captured in the diversion channel. It is also critical to maximize the ratio of sediment load to water, or Sand/Water Ratio (SWR), captured in the diversion channel to minimize (or completely eliminate) the potential of shoaling downstream of the diversion intake. The following simple indicator was used to assess the performance of each diversion design tested in this study:

SWR = (Sand Load Diverted/Sand Load in the River) / (Water Discharge Diverted/Water Discharge in the River).

Where SWR is the Sand/Water Ratio. As indicated above, the goal is to maximize SWR since the higher the value of SWR the lower the potential of shoaling in the river. As shown in Table 6.1, this design extracted 1.62% of the main river water discharge, while it only extracted 1.19% of the main river silt load, and 0.42% of the main river sand (size range 63 – 250 microns) load. As such, the SWR is 0.26. This is not a healthy ratio and is indicative of a poor efficiency of this design to capture an adequate amount of sediment.

6.2 Modified Alignment (RM 60.7):

A modified alignment was proposed to increase the diversion's ability to capture sediment and improve the sediment-water ratio in the diversion channel as compared to the sediment-water ratio in the main stem of the river. The intake of the diversion channel was moved upstream approximately 2,640 ft (800 m). This move placed the intake at the tail end of the sand bar on the right-descending bank (Alliance South). Further, bends were removed from the alignment (see Figure 6.6), reducing the energy losses along the length of the diversion channel.

Several designs were tested for this alignment. The first test was for a diversion channel with a capacity of 45,000 cfs (1,274 m^3/s). This design was tested with and without the earthen guide dike to assess its ability to enhance the performance of the diversion. Other tests include a diversion channel with a capacity of 15,000 and 75,000 cfs. For these tests, the channel was designed using HEC-RAS.

The first test performed for this alignment was the 45,000 cfs with the guide dike in place. The bottom width of the trapezoidal channel was 90 ft (27.4 m) and the side slopes were 3:1 (H:V). The channel transitioned to a trapezoidal channel with a bottom width of 90 ft (27.4 m) and a compounded side slope.

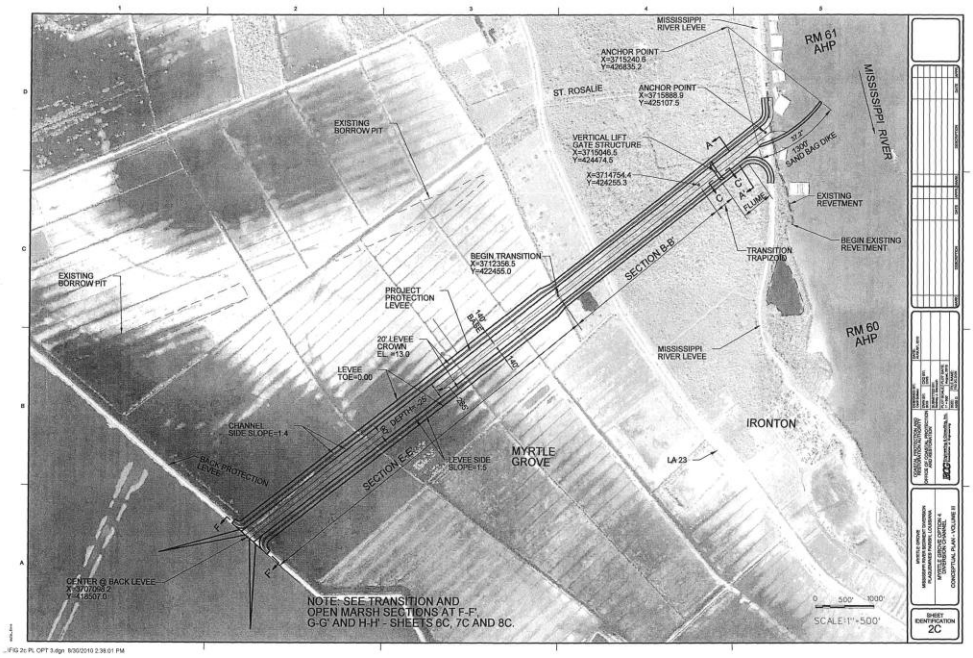

Figure 6.6: Layout of Modified Alignment (RM 60.7) [MA-RM60.7-45K]

The same river conditions that were used for the Original Alignment were used here for the Modified Alignment (River Flow = 700,000 CFS (19,821 m^3/s), River tail-water = 6.23 ft-NAVD88 (1.9 m), diversion channel tail water = 5 ft-NAVD88 (1.524 m). As mentioned before, for a nominal discharge of 1,000,000 cfs in the river, the design capacity of the diversion channel is 45,000 cfs, however, under the flow conditions used in this test (river flow of 700,000 cfs), the flow captured by the diversion channel was 33,735 CFS (955 m^3/s). A summary of the sediment load captured in the diversion channel is shown in Table 6.2.

As shown in Table 6.2, this design extracted 4.82% of the main river water discharge and 6.55% of silt, and 4.14% of sand (size range 63 – 250 microns). This performance is much improved (SWR = 0.85) compared to the Original Alignment.

Table 6.2: Summary of the sediment load (for every size class) captured by the diversion channel MA-RM60.7-45K.

	Mississippi River (Main Stem)	Diversion Channel MA-RM60.7-45K	Ratio
Water Discharge (m3/s)	19,821	955	4.82
Water Discharge (CFS)	700,000	33,735	4.82
Sediment Load (metric tons/d) - 32 Micron	233,539	15,306	6.55
Sediment Load (metric tons/d) - 63 Micron	10,839	663	6.12
Sediment Load (metric tons/d) - 96 Micron	21,816	1,230	5.64
Sediment Load (metric tons/d) - 125 Micron	34,437	1,637	4.75
Sediment Load (metric tons/d) - 250 Micron	23,460	218	0.93
Total 63 - 250 Micron Load (metric tond/d)	90,554	3,748	4.14

During the simulation of the 45,000 cfs Modified Alignment design, it was observed that the earthen dike at the intake of the diversion created a large eddy in the main stem of the river and a sediment buildup on the downstream side of the diversion. To avoid this problem, the dike was removed, and a simulation was performed without the dike in place. This simulation was labeled “ND” for “No Dike” (ND-RM60.7-45K). Table 6.3 provides a summary of the sediment load captured in the diversion channel without the dike. Removing the dike slightly lowered the water discharge to 4.73% of the total river discharge. The dike created a stagnation point and built up the water stage at its upstream face, creating a favorable head-differential and improving the total water discharge in the diversion channel. However, this advantage is not significant as can be seen in the with- and without-dike simulations, 4.82% and 4.73% respectively.

Table 6.3: Summary of the sediment load (for every size class) captured by the diversion channel ND-RM60.7-45K.

	Mississippi River (Main Stem)	Diversion Channel ND-RM60.7-45K	Ratio
Water Discharge (m3/s)	19,821	937	4.73
Water Discharge (CFS)	700,000	33,075	4.73
Sediment Load (metric tons/d) - 32 Micron	233,539	13,819	5.92
Sediment Load (metric tons/d) - 63 Micron	10,839	619	5.71
Sediment Load (metric tons/d) - 96 Micron	21,816	1,150	5.27
Sediment Load (metric tons/d) - 125 Micron	34,437	1,675	4.86
Sediment Load (metric tons/d) - 250 Micron	23,460	528	2.25
Total 63 - 250 Micron Load (metric tond/d)	90,554	3,972	4.39

The ND-RM60.2-45K alignment extracted slightly lower silt load; 5.92% of the main river silt load compared to 6.55% obtained with the MA-RM60.2-45K. However, the ND-RM60.2-45K alignment improved the sand load (range of 63 – 250) extracted in the diversion channel compared to the MA-RM60.2-45K alignment, namely 4.39% and 4.14%. Overall, for this design the SWR is 0.93.

Overall, the dike does not appear to be a critical component of the design. In fact, it is detrimental for some size classes. Hence, it is recommended to remove the dike, given the negative impacts it has on the flow field in the river and the lower capture of larger sediment grain sizes.

6.3. Additional Runs

In this section, the results of two additional simulations are presented, namely, ND-RM60.7-15K, and ND-RM60.7-75K. The two simulations were performed using the same flow conditions of all the other diversion scenarios. As such, direct comparisons can be made among all the diversion scenarios. Table 6.4 and 6.5 show the results of these two simulations.

The results presented in Tables 6.4 and 6.5 clearly show that the performance of the modified alignment is better than the original alignment. The results presented in Table 6.4 should be compared to those presented in Table 6.1. The ND-RM60.7-15K diversion delivered approximately 150% silt load compared to the OA-RM60.2-15K diversion. The level of improvement increased for the larger sediment size classes. The SWR for the 15,000 cfs diversion improved from 0.26 for the original alignment to 0.60 for the modified alignment. However, it should be noted that even with the modified alignment, the SWR

is low. To minimize shoaling and other undesirable impacts, a SWR value of at least 1 is needed. The results presented in Tables 6.3, 6.4, and 6.5 indicate that the larger the amount of water diverted, the higher the value of SWR. Table 6.5 shows that the 75K diversion extracted 9.77% of sand the river while only extracting 8.7% of the river water, i.e. SWR of 1.12. This positive ratio (higher than 1) is encouraging as it implies little potential problems with shoaling in the river as a results of the diversion.

Table 6.4: Summary of the sediment load (for every size class) captured by the diversion channel ND-RM60.7-15K.

	Mississippi River (Main Stem)	Diversion Channel ND-RM60.7-15K	Ratio
Water Discharge (m3/s)	19,821	361	1.82
Water Discharge (CFS)	700,000	12,733	1.82
Sediment Load (metric tons/d) - 32 Micron	233,539	4,189	1.79
Sediment Load (metric tons/d) - 63 Micron	10,839	188	1.74
Sediment Load (metric tons/d) - 96 Micron	21,816	335	1.53
Sediment Load (metric tons/d) - 125 Micron	34,437	420	1.22
Sediment Load (metric tons/d) - 250 Micron	23,460	44	0.19
Total 63 - 250 Micron Load (metric tond/d)	90,554	987	1.09

Table 6.5: Summary of the sediment load (for every size class) captured by the diversion channel ND-RM60.7-75K.

	Mississippi River (Main Stem)	Diversion Channel ND-RM60.7-75K	Ratio
Water Discharge (m3/s)	19,821	1,725	8.70
Water Discharge (CFS)	700,000	60,918	8.70
Sediment Load (metric tons/d) - 32 Micron	233,539	24,789	10.61
Sediment Load (metric tons/d) - 63 Micron	10,839	1,156	10.67
Sediment Load (metric tons/d) - 96 Micron	21,816	2,357	10.81
Sediment Load (metric tons/d) - 125 Micron	34,437	3,726	10.82
Sediment Load (metric tons/d) - 250 Micron	23,460	1,607	6.85
Total 63 - 250 Micron Load (metric tond/d)	90,554	8,847	9.77

7. Conclusions

A three-dimensional numerical model has been set up for the Myrtle Grove sediment diversion project. The model was calibrated and validated against detailed field measurements of velocity and sediment concentrations.

The FLOW-3D suite of models has the adequate physical processes and was selected for this study. The model results compared well with the field observations. The complex flow features observed in the field, such as the strong eddy at the Myrtle Grove bend and the complex sediment concentration vertical profiles, were reproduced by the model.

The model was used to assess the performance of various diversion alignments and sizes. Two alignments were tested. The Original Alignment did not perform well in terms of the water-to-sediment ratio in the diversion channel compared to in the main stem of the Mississippi River. The performance of this alignment was impacted by the location of the intake and the orientation of the alignment (adverse angle to the flow direction in the river). The intake is located on the outside of the Myrtle Grove bend (right descending bank), where the sand material has already started to migrate from the right to the left descending bank. This alignment design also contained two bends that caused energy losses, reducing the water and sediment carrying capacity. A summary of the diversion scenarios is shown in Table 7.1.

The Modified Alignment moved the diversion channel intake to RM60.7. This allowed for capturing the sand material coming off the sand bar before it starts the migration towards the left descending bank. Further, the Modified Alignment had a favorable angle relative to the flow direction in the river. Hence, it produced a much more favorable water-to-sediment ratio. As seen in Table 7.1, for the 15,000 cfs diversion, the modified alignment improved the SWR to 0.60 compared to the 0.26 for the original alignment. Further, it can be seen in the larger the diversion size the higher the SWR. Comparing the 15,000 cfs, 45,000 cfs, and 75,000 cfs for the modified alignment (all without the guide dike), the SWR improved from 0.60 to 0.93 to 1.12, respectively.

It was observed that the dike component of the Modified Alignment design had an adverse effect on the flow field in the river (large eddy and potential sediment buildup on the downstream side). Thus, the dike was removed and the performance of the diversion was tested without it (run ND-RM60.7-45K). The results indicate that the dike does not enhance the performance of the diversion significantly. In fact, it is detrimental for some of the larger sediment size classes. As seen in Table 7.1, removing the dike

improved the SWR from 0.85 to 0.93. Therefore, it is recommended to remove the dike from the design, considering the advantage of avoiding problems in the river and reduction in project cost due to removing a component.

Table 7.1: Summary of the sediment load (for every size class) captured by the three alternative diversion channel alignments

	Mississippi River (Main Stem)	Diversion Channel OA-RM60.2-15K	Diversion Channel ND-RM60.7-15K	Diversion Channel MA-RM60.7-45K	Diversion Channel ND-RM60.7-45K	Diversion Channel ND-RM60.7-75K
Water Discharge (m3/s)	19,821	322	361	955	937	1,725
Water Discharge (CFS)	700,000	11,369	12,733	33,735	33,075	60,918
Sediment Load (metric tons/d) - 32 Micron	233,539	2,786	4,189	15,306	13,819	24,789
Sediment Load (metric tons/d) - 63 Micron	10,839	104	188	663	619	1,156
Sediment Load (metric tons/d) - 96 Micron	21,816	144	335	1,230	1,150	2,357
Sediment Load (metric tons/d) - 125 Micron	34,437	133	420	1,637	1,675	3,726
Sediment Load (metric tons/d) - 250 Micron	23,460	2	44	218	528	1,607
Total 63 - 250 Micron Load (metric tond/d)	90,554	383	987	3,748	3,972	8,847
Sediment/Water Ration (SWR)		0.26	0.60	0.85	0.93	1.12

Additional intake designs should be considered to improve the SWR. Since the upper layers of the water column carry the least amount of sand, it is recommended that an orifice-like entrance be tested. Such design will draw water from the deeper and more sand rich layers Significant improvement to the SWR will be cost effective if it successfully removes (or at least minimizes) any shoaling and navigation concerns.

It should be noted that the analysis performed thus far focused on optimizing the design of the intake structure in terms of size and location. Further three-dimensional and one-dimensional analyses are required to address the long-term and large-scale potential shoaling problems that may result from constructing the diversion.

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APPENDIX F

DELTA BUILDING DIVERSION

AT MYRTLE GROVE

MORPHOLOGICAL MODELING

Prepared for Louisiana Office of Coastal Protection and Restoration

April 4, 2011

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1 INTRODUCTION

The Delta Building Diversion at Myrtle Grove (BA-33) project was initially proposed to be another large freshwater diversion project in the Barataria Basin in addition to the Davis Pond Diversion project. The Myrtle Grove Diversion project would involve installation of a diversion structure on the west bank of the Mississippi River between Ironton on the south and the Alliance Refinery on the north; a conveyance channel with parallel mainline flood control levees and an outflow channel with guide levees.

As initial studies for the project progressed, it was decided to also analyze the land-building potential if the project was modified to also act as a sediment diversion. Extensive data collection efforts, sediment transport analyses, numerical modeling and physical modeling of the Mississippi River; and a preliminary design of the diversion structure and conveyance channel were completed by others.

This report summarizes the analyses of the Barataria Basin that have been accomplished by Moffatt & Nichol related to the Myrtle Grove Diversion project.

2 PHASE I – CWPPRA MODELING

Initially, Moffatt and Nichol (M&N) was retained by Louisiana Department of Natural Resources - LNDR (now Office of Coastal Protection and Restoration - OCPR) to perform alternative modeling for the ***Coastal Wetlands Planning, Protection, and Restoration Act (CWPPRA) Delta Building Diversion at Myrtle Grove***. This effort was undertaken to provide the ability to more completely evaluate the effects of the proposed project on a basin-wide scale.

Moffatt & Nichol developed and completed an estuarine hydrodynamic model for evaluating the Barataria Basin effects due to operating both the Myrtle Grove and Davis Pond diversions at varying and increasing flow rates. The model is capable of:

- Simulating existing water movements and salinity distributions within the basin;
- Future use by OCPR to project future hydrodynamic conditions for both with and without implementation of restoration strategies;
- Easily upgraded and / or refining as new data become available and modeling technology and capabilities improve;
- And evaluating the effects of proposed Myrtle Grove project on a basin-wide scale.

2.1 Model Description

The numerical model developed and documented in the Moffatt & Nichol August 2005 report to the OCPR titled "Barataria Basin: Hydrodynamic & Salinity Model Development," was used for this study. Some modifications were made to the original model grid to increase resolution in the vicinity of the Myrtle Grove diversion and to eliminate one discharge boundary condition where limited data was available. Additionally, rainfall runoff from areas within the basin not included in the model grid was added.

The numerical modeling system used in this study consisted of the US Army Corps of Engineers (USACE) finite element hydrodynamics (RMA-2) and salinity (RMA-4) models. Figure 2-1 presents the model mesh which contains 19,112 elements and 56,236 nodes. The offshore area of the model domain was extended at its southwest corner to encompass the outlet of the Bayou LaFourche channel. This extension has the advantage of rendering the previous open water boundary at the outlet of the Southwest (SW) Canal (near Leeville) redundant since it is now part of the model domain; a move prompted by the dearth of flow data at the SW Canal for the entire simulation period. The Bayou LaFourche channel was extended northward until just before the GIWW consistent with the location of an existing lock there that interrupts its interaction with the channel further north.

The Barataria Basin model currently has thirteen hydrodynamic boundary conditions as shown in Figure 2-2. They are an open water level boundary in the Gulf of Mexico, a discharge boundary at the intersection of the GIWW with Bayou Lafourche, discharge boundaries at both the Davis Pond Diversion and the proposed Myrtle Grove Diversion, and additionally nine discharge boundaries to simulate the freshwater streamflows from the catchment areas of Lac des Allemands (6), Lake Cataouatche (2) and the northeast end of GIWW (1) as recommended in the Barataria Basin Report (Moffatt & Nichol, 2005) (Figure 2-3).

The local bathymetry and meshes have been refined, especially those near the Myrtle Grove diversion, to provide a smooth flow transition into the basin consistent with field conditions. The dimensions of the outlet channel from the Myrtle Grove diversion are as per the conceptual design provided by the USACE, while those at the catchment flow outlets are suitably sized to prevent super-critical flow.

The wind and precipitation/evaporation distributions are based on a zone-based approach as shown in Figure 2-4. Four wind stations covering the entire model domain are: Grand Isle (GDIL1), Boothville (161157), Lake Salvador near Lafitte (DCPBA05), and New Orleans International Airport (166660). The wind adjustment factors for elevation, duration and temperature were made. The resulting wind time series were then subjected to the 4-hour moving averaging. Additionally, the maximum wind speed was capped at 20 mph (1-hour average) as previous experience has indicated that the RMA-2 code cannot handle high wind events without numerical instability problems when large sections are subject to wetting and drying (even if the marsh porosity approach is used). This capping has the effect of removing episodic wind events that are of short duration, and hence are unlikely to have a significant effect on the long-term salinity regime after an extreme event during which the diversions would not be operating.

The precipitation distribution is based on a similar zoned approach where the entire model domain is covered by 5 stations: Mississippi River @ Bonnet Carre Spillway (7374370), Mississippi River @ New Orleans (7374510), Mississippi River at Venice (7374550), Bayou LaFourche south of Golden Meadow (7381305), and Boothville (161157). For evaporation, the bulk aerodynamic method used by Park (2002) is adopted. By this method, the evaporation rate becomes a function of air and dew temperature, atmospheric pressure at the sea surface, and the wind speed. The time series data for the air and dew temperatures and the atmospheric pressure are taken from the Grand Isle station and deemed representative of the entire model domain. The wind data are based on the measurement at the four wind stations discussed above, but unsmoothed (instantaneous hourly readings) and uncapped. Since there is overlap in the precipitation (5) and wind (4) zones, the computed evaporation rates are applied to the precipitation zones in the manner summarized in

Table 2-1. The net precipitation is then computed as the difference between the measured precipitation and the computed evaporation rate on an hourly basis.

A simulation time period of one calendar year was chosen to assess the alternative plans. This simulation period is from August 2002 to July 2003 which included the two primary model calibration periods when extensive data collection efforts were undertaken. It should be noted that two tropical storms (Isidore and Lili) directly impacted the basin during late September and the beginning of October 2002, while Tropical Storm Bill impacted the basin at the end of June and beginning of July 2003. The CWPPRA modeling effort was finished in October 2008 and documented in the Moffatt & Nichol report to OCPR titled “Delta Building Diversion at Myrtle Grove (BA-33) Alternative Modeling” (Moffatt & Nichol, 2008).

Table 2-1. Evaporation Zones

Precipitation Zone	Applied Evaporation Zone (based on wind)
7374370	Average of 16660 and DCPBA05
7374510	16660
7381305	Average of DCPBA05 and GDIL1
161157	Average of GDIL1 and 161157
7374550	161157

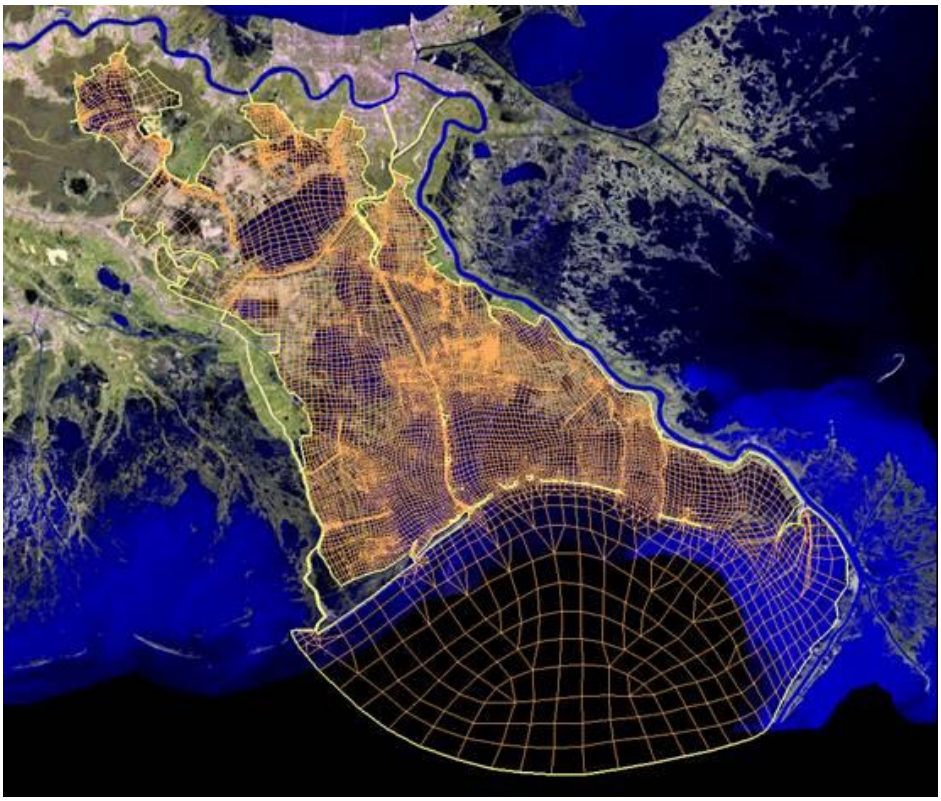


Figure 2-1. Barataria Basin Finite Element Grid

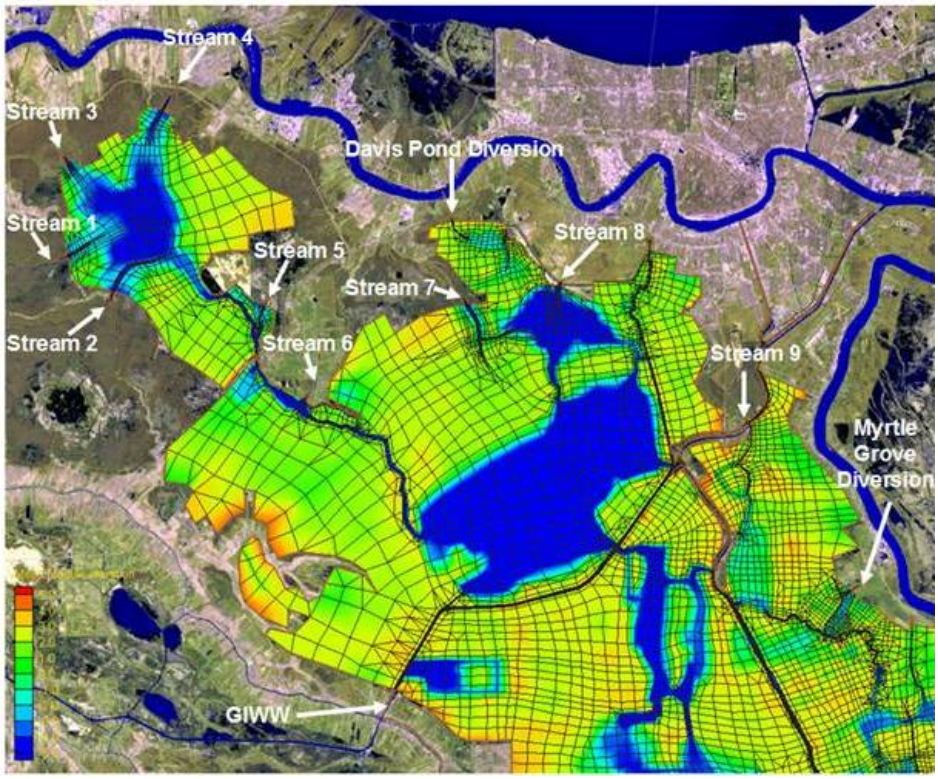


Figure 2-2. Model Boundary Condition Locations

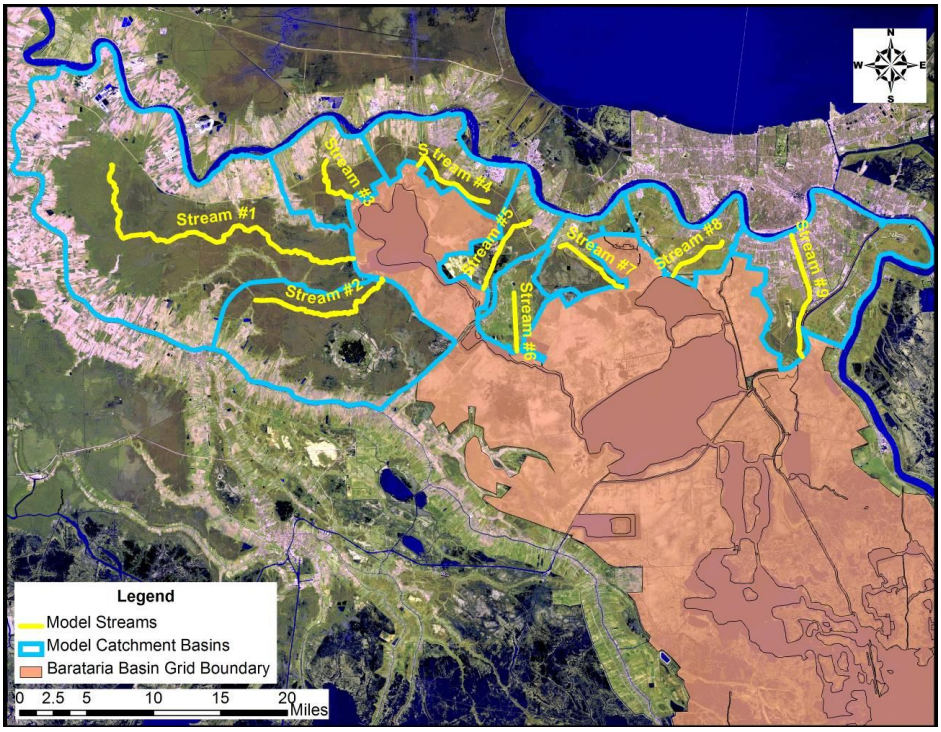


Figure 2-3. Runoff Catchments and Streams

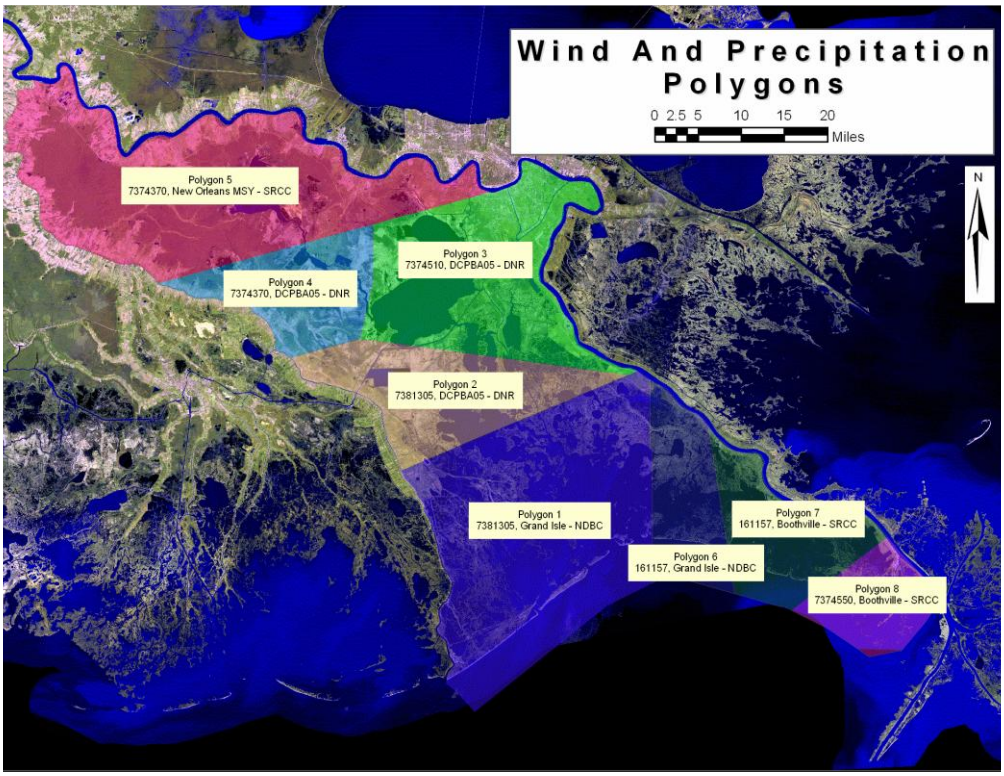


Figure 2-4. Wind and Precipitation Zones

2.2 Model Results

For this alternative modeling study, four different Myrtle Grove diversion scenarios were modeled in conjunction with three appropriate diversion regimes at Davis Pond as shown in Table 2-2. These diversion flow rates were provided by the USACE. The baseline condition for salinity change comparison was taken as the "existing" condition runs with only nominal diversions at Myrtle Grove and Davis Pond.

Additionally, three extra runs were made to distinguish the effects of the Davis Pond Diversion from those due to the Myrtle Grove Diversion. For these three runs, nominal discharge boundary conditions at the Myrtle Grove Diversion were applied coupled with the three different Davis Pond Diversion discharge scenarios.

In order to investigate a larger Myrtle Grove diversion effect, another scenario run with double-high Myrtle Grove diversion (diversion flows are twice the flow rates of the high Myrtle Grove scenario) in combination with a high Davis Pond diversion was also performed.

Therefore, a total of seventeen one-year RMA-2 and RMA4 runs were performed:

- "Existing" Condition (EXCO)
- High Davis Pond & High Myrtle Grove (DHMH)
- High Davis Pond & Medium Myrtle Grove (DHMM)
- High Davis Pond & Low Myrtle Grove (DHML)
- Medium Davis Pond & High Myrtle Grove (DMMH)
- Medium Davis Pond & Medium Myrtle Grove (DMMM)
- Medium Davis Pond & Low Myrtle Grove (DMML)
- Low Davis Pond & High Myrtle Grove (DLMH)
- Low Davis Pond & Medium Myrtle Grove (DLMM)
- Low Davis Pond & Low Myrtle Grove (DLML)
- High Davis Pond & Nominal Myrtle Grove (DHMN)
- Medium Davis Pond & Nominal Myrtle Grove (DMMN)
- Low Davis Pond & Nominal Myrtle Grove (DLMN)
- High Davis Pond & Alternative R1 Myrtle Grove (DHMR1)
- Medium Davis Pond & Alternative R1 Myrtle Grove (DMMR1)
- Low Davis Pond & Alternative R1 Myrtle Grove (DLMR1)
- High Davis Pond & Double-High Myrtle Grove (DHM2H)

After the one year runs were completed, the following data were extracted from the RMA-2 and RMA-4 results:

- The average salinities on a monthly, semi-annual (December – May), and an annual basis;
- The average, minimum and maximum water levels on a monthly, semi-annual (December – May), and an annual basis;
- The average and maximum current velocity magnitudes on a monthly, semi-annual (December – May), and an annual basis;
- The 5 ppt and 15 ppt salinity contour lines; and
- Water level and velocity magnitude results from seventeen specific locations throughout the basin (Figure 2-5).

Table 2-2: Discharge Time Series at the Diversions (cfs)

Month	Davis Pond Diversion				Myrtle Grove Diversion				
	Existing	High	Medium	Low	Existing	High	Medium	Low	R1
Jan	Nominal 10	8,000	6,000	4,000	Nominal 10	16,500	5,300	10	19,881
Feb		10,560	7,920	5,280		18,300	6,400	10	33,063
Mar		10,560	7,920	5,280		19,500	7,700	10	39,546
Apr		10,560	7,920	5,280		19,420	7,500	10	39,546
May		10,560	7,920	5,280		19,200	7,000	10	39,546
Jun		10,560	7,920	5,280		18,950	4,800	10	10
Jul		6,000	4,500	3,000		14,840	3,300	2,500	10
Aug		4,000	3,000	2,000		9,740	3,300	1,500	10
Sep		4,000	3,000	2,000		9,550	3,000	10	10
Oct		4,000	3,000	2,000		9,400	3,000	1,000	10
Nov		6,000	4,500	3,000		9,330	3,000	1,000	10
Dec		8,000	6,000	4,000		12,900	4,000	10	19,881

Note: The medium and high flows for the Davis Pond diversion are computed as 150% and 200%, respectively, of the corresponding low flows; for Myrtle Grove diversion R1, the monthly discharges are the maximum values.

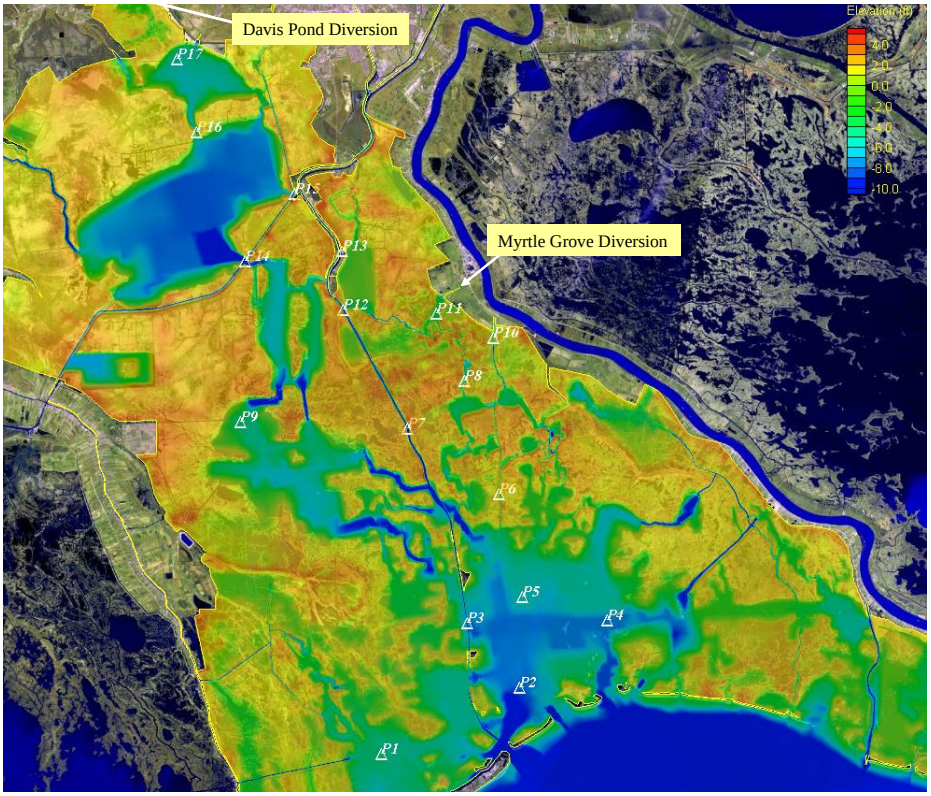


Figure 2-5. Location Map for Hydrodynamics Results

Figure 2-6 through Figure 2-8 present the impact of the proposed Myrtle Grove diversions on the 5 ppt and 15 ppt salinity contours in the basin on an annually averaged basis for the 2002-2003 time frame modeled. These figures indicate that by comparing the results to the nominal Myrtle Grove diversion scenarios:

- In the case of the high Myrtle Grove diversion, the 5 ppt and 15 ppt salinity contour lines would retreat to the south by about 4 miles and 1.5 miles, respectively; further when combined with the high Davis Pond diversion than if there was no diversion at Myrtle Grove and only a high Davis Pond diversion (DHMH-DHMN). The retreats would be about 4.5 miles and 2 miles if combined with a medium Davis Pond diversion (DMMH-DMMN); and the retreats would increase to 6 miles and 2.5 miles combined with a low Davis Pond diversion (DLMH-DLMN). In other words, the impact of a high Myrtle Grove diversion increases with a decrease in the diversion at Davis Pond.
- Under the medium Myrtle Grove diversion scenarios, the 5 ppt and 15 ppt lines would move about half the distance of corresponding high Myrtle Grove diversion cases.
- The low Myrtle Grove diversion shows little impact on the salinity levels in the basin.
- For the R1 diversion case at Myrtle Grove, the annually averaged 5 ppt contour lines retreat to the south less than the high Myrtle Grove cases, and the 15 ppt contours are close to the high Myrtle Grove cases. The reason is that there are no diversions from June to November.
- For the double-high Myrtle Grove diversion case, both the 5 ppt and 15 ppt contours retreat further to the south.

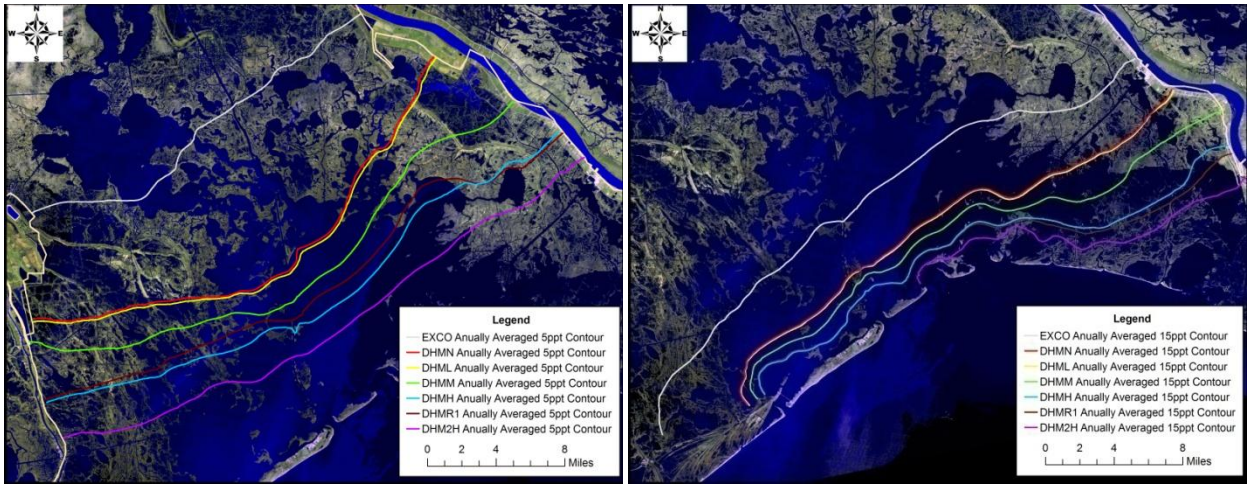


Figure 2-6. Annually Averaged Contour Changes under High Davis Pond Diversion Rate

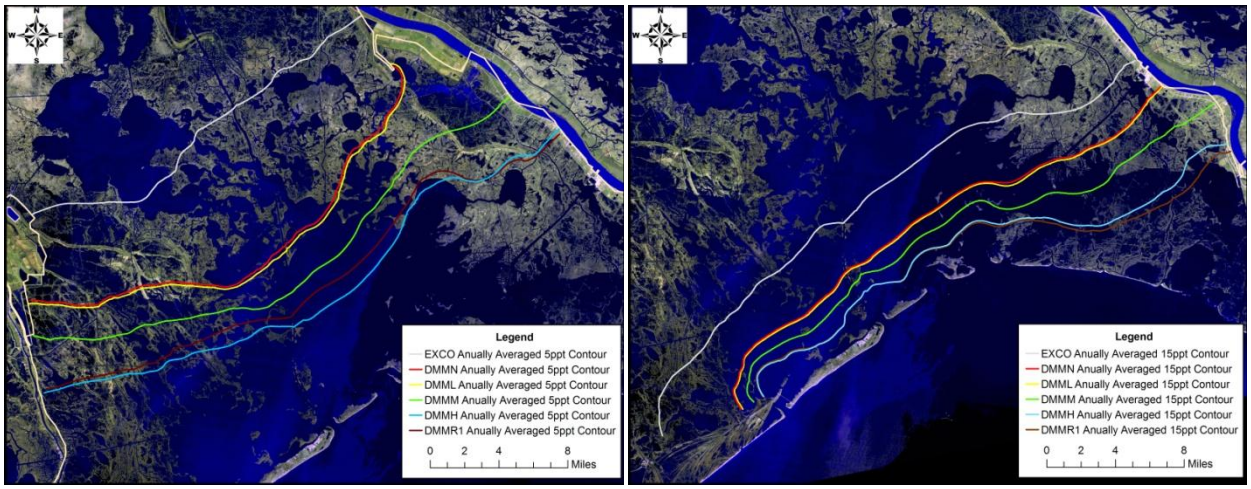


Figure 2-7. Annually Averaged Contour Changes under Medium Davis Pond Diversion

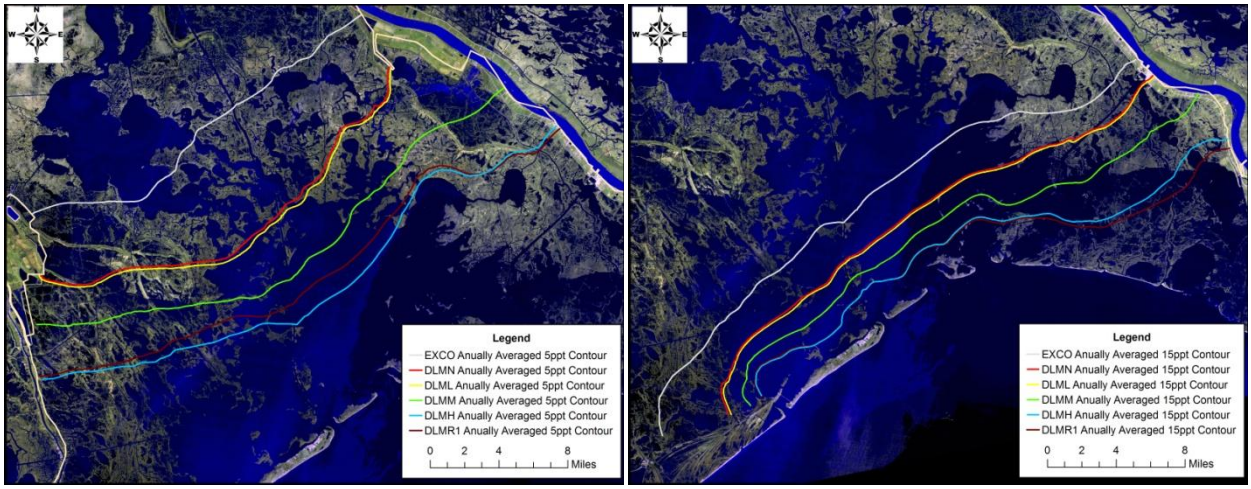


Figure 2-8. Annually Averaged Contour Changes under Low Davis Pond Diversion

The following conclusions were drawn after analyses and review of the alternative modeling results:

- The impacts on salinity levels in the Barataria Basin from the Myrtle Grove project depend on the diversion regimes at Davis Pond. The effects of the Myrtle Grove project are reduced under higher Davis Pond diversion scenarios.
- The Myrtle Grove project under low diversion has negligible impact on salinity levels in the Barataria Basin regardless of the Davis Pond Diversion operational level.
- High Myrtle Grove diversions could reduce annual average salinity levels over 6 ppt depending upon the magnitude of diversions at Davis Pond while medium Myrtle Grove diversions would only reduce the annual average salinity by less than 4 ppt.
- The high Myrtle Grove diversion scenario would push the annual 5 ppt and 15 ppt salinity level contours twice as far southward as the medium Myrtle Grove diversion case; regardless of the magnitude of the different Davis Pond diversion.
- On a semi-annual (December – May) basis, the R1 and double-high Myrtle Grove diversion scenarios push the 15 ppt salinity line to near the backside of the barrier islands except in the immediate vicinity of the passes, and the far eastern section of the basin.
- From a hydrodynamic point of view, on average, the larger diversions from Myrtle Grove and Davis Pond cause significant water surface elevation and current magnitude increases in the regions adjacent to the diversion sites.

Recommendations include:

- The Low Myrtle Grove diversion scenario is not an effective option to reduce salinity levels in the Barataria Basin and should not be considered further.
- The Medium Myrtle Grove scenario only has minimal effects on salinity levels and may not be cost effective.
- The High Myrtle Grove diversion scenario is effective at reducing salinity levels, albeit less so when higher diversions occur at Davis Pond.
- Higher diversions such as the R1 and double-high scenarios should be given further consideration due to their significant potential impacts on reducing salinity levels throughout the Barataria Basin.
- The Myrtle Grove diversions do not significantly impact areas southeast of Port Sulphur, and thus additional diversions in the vicinity of Port Sulphur, Empire and Fort Jackson would be necessary to reduce salinities in this area.
- Further investigation is warranted to determine if the increases in water levels and current velocities results from the higher diversion are within acceptable limits, and / or what operational restrictions may be required if they are not acceptable.

3 PHASE II – HYDRODYNAMIC PRE-SCREENING RUNS

The work described in Phase I was built upon to pre-screen Myrtle Grove diversion flows for the State and other non-governmental organizations (NGO) in order to determine feasible diversion flows for a sediment diversion. The RMA2 model was used to evaluate maximum diversion magnitudes of 15,000 cfs; 45,000 cfs; 75,000 cfs; 150,000 cfs; 240,000 cfs and 300,000 cfs (Figure 3-1) on water level and velocity increases in the basin. The purpose of these runs was to establish the upper limit of diversion magnitudes for further modeling by identifying the flows with significant, insurmountable impacts. Wind and rain were not evaluated in these runs. The results of this effort were documented in a Moffatt & Nichol report submitted in May 2009.

The diversion was assumed to be operated from February to July. The Myrtle Grove diversion hydrographs were developed based on the hydrograph from the Mississippi River.

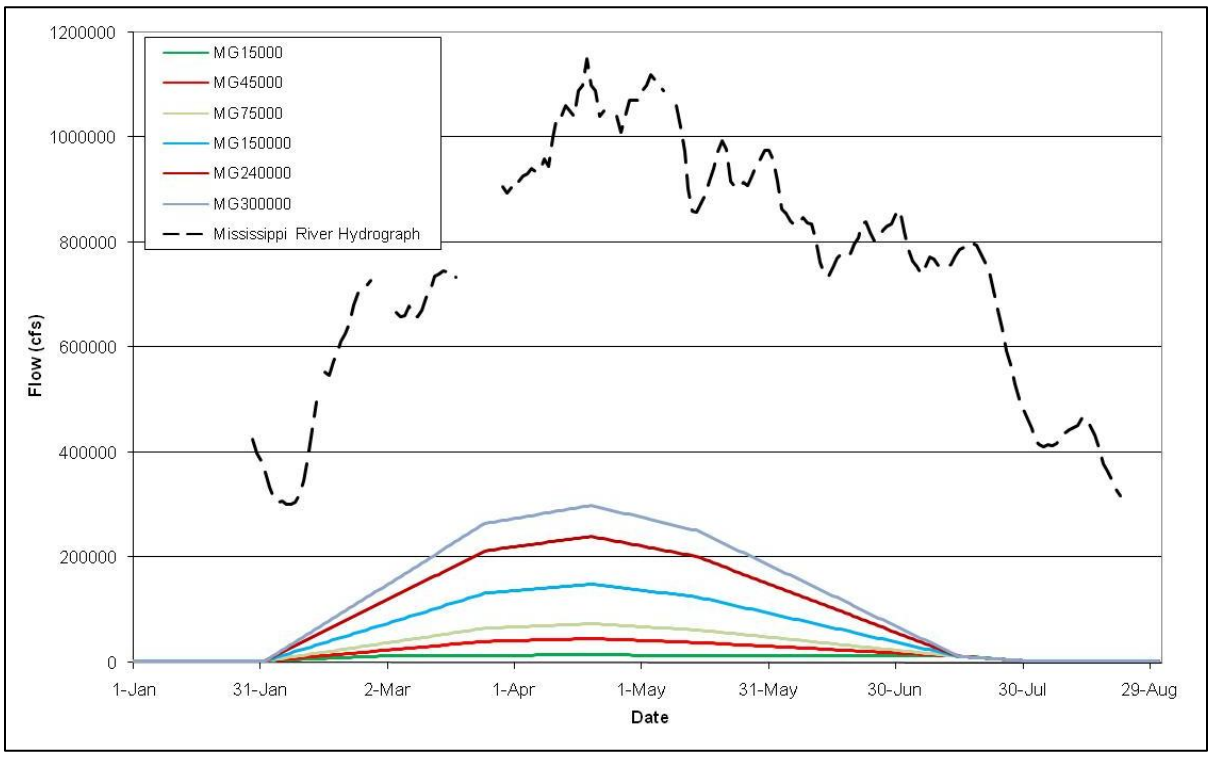
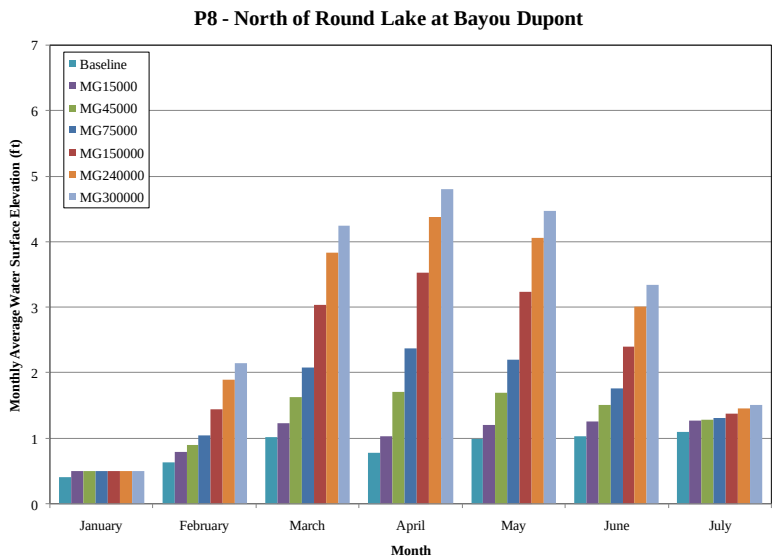


Figure 3-1. Myrtle Grove Diversion Hydrographs for Pre-screening Model Runs

At first, a modeling run with nominal freshwater discharges from the Davis Pond and Myrtle Grove diversions was conducted to act as the baseline condition. The offshore water level boundary condition and GIWW discharge boundary condition are the same as the ones in the Phase I study. Next, the six Myrtle Grove diversion scenarios were modeled in conjunction with a nominal David Pond diversion.

After the model runs finished, the monthly average, maximum and minimum water surface elevations, as well as the monthly average and maximum current velocity magnitude for each diversion scenario for the period of February to July were extracted and compared to the baseline condition. The results from the 17 points in Figure 2-5 were also extracted. Figure 3-2 to Figure 3-7 demonstrate the monthly average and maximum water surface elevations and velocity magnitudes for all six diversion scenarios at three different locations: P8 – North of Round Lake at Bayou Dupont, P11 – South of Diversion Canal at Bayou Dupont, and P12 – Barataria Waterway South of the Town of Lafitte. Figure 3-8 and Figure 3-9 present the monthly average water surface elevation and velocity magnitude changes relative to the baseline condition caused by different Myrtle Grove diversion magnitudes in April when the diversion reaches its peak.

It is evident that the highest diversions could cause significant flooding problems and severe currents in the regions adjacent to the diversions. On average, near the Town of Lafitte in April when the diversion reaches its peak flow, the 45,000 cfs and 75,000 cfs diversion flows could increase water levels by 1 and 2 ft, respectively; but for the 150,000 cfs and 300,000 cfs diversions, the peak water levels could increase by 3 and 5 ft, respectively. The increased velocity with the larger diversions could cause local scour and damage the existing marshes as well as creating a potential safety hazard.



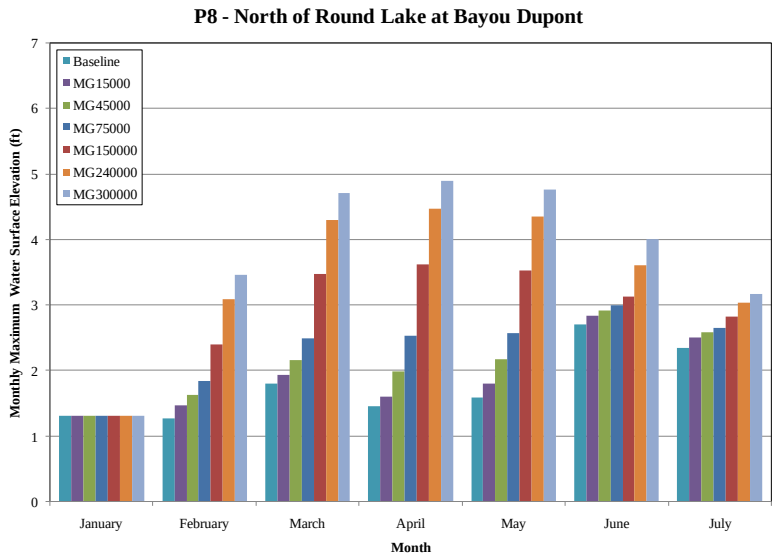
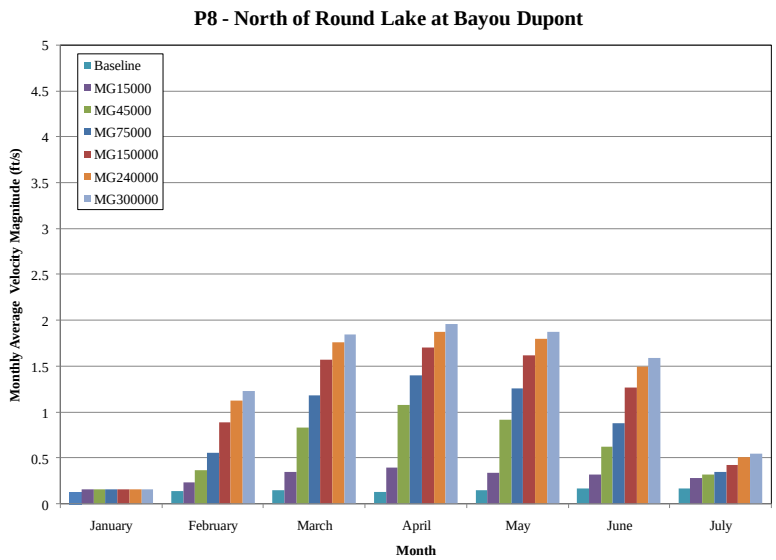


Figure 3-2. Monthly Average and Maximum Water Surface Elevations near North of Round Lake



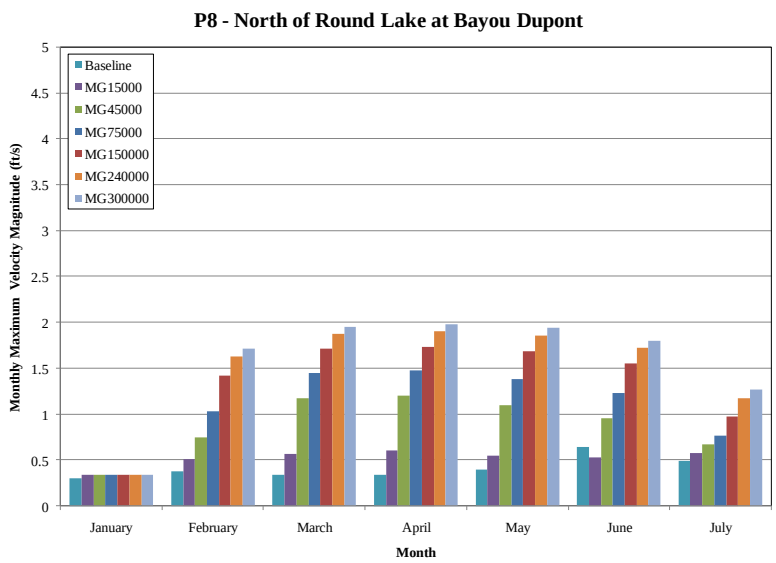
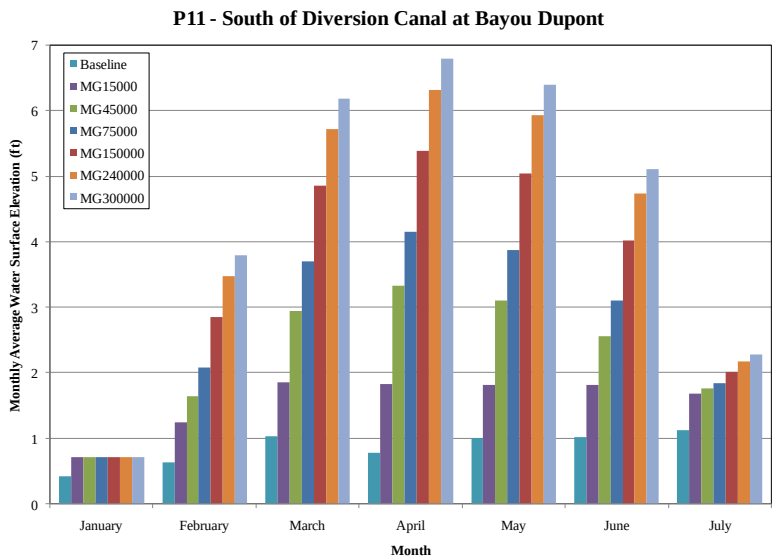


Figure 3-3. Monthly Average and Maximum Velocity Magnitudes near North of Round Lake



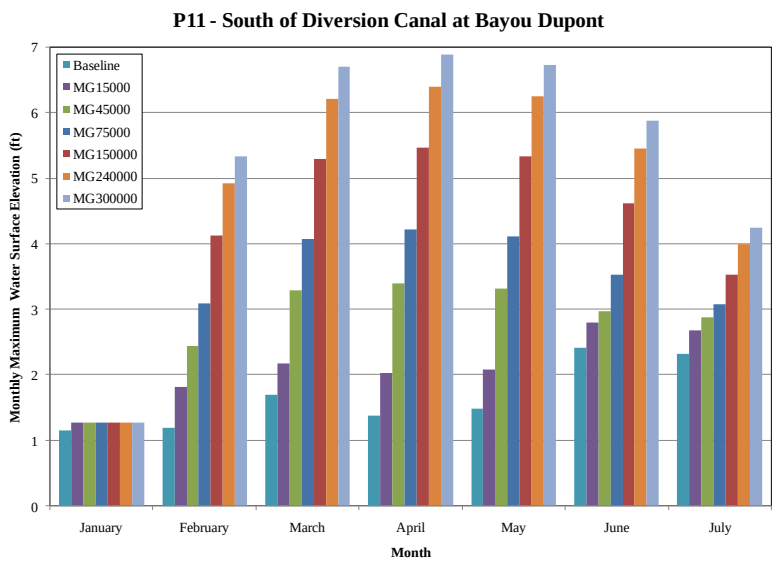
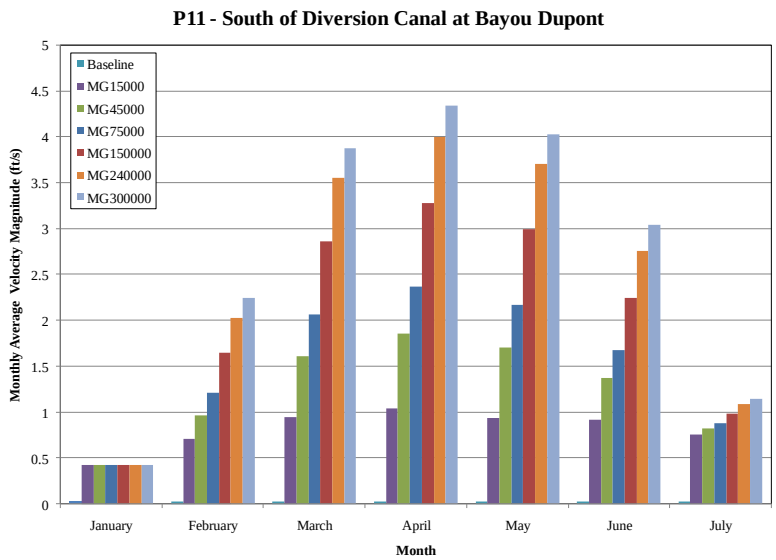


Figure 3-4. Monthly Average and Maximum Water Surface Elevations near South of Diversion Canal



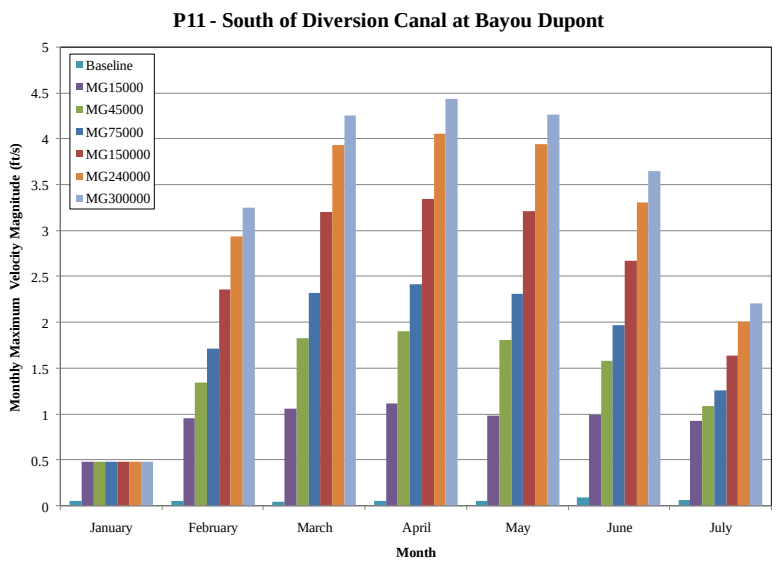
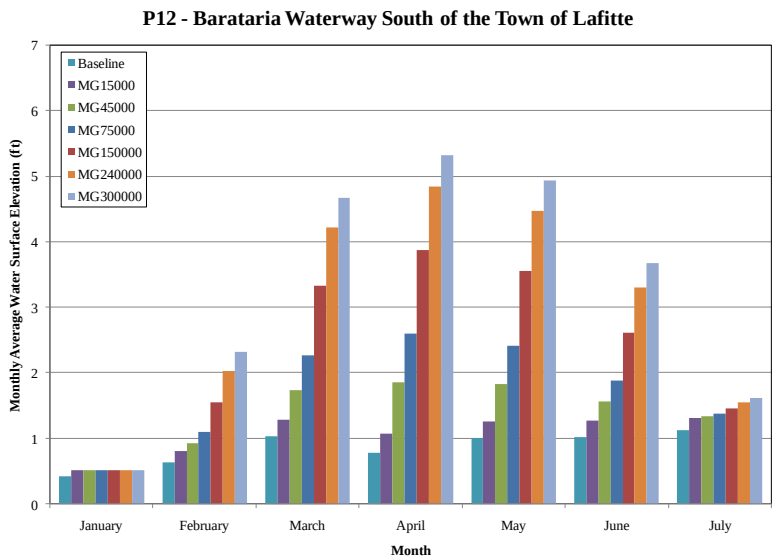


Figure 3-5. Monthly Average and Maximum Velocity Magnitudes near South of Diversion Canal



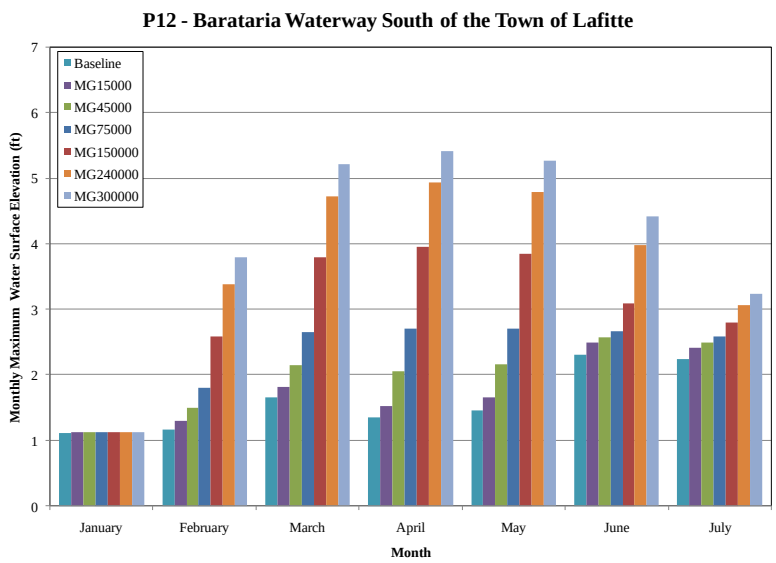
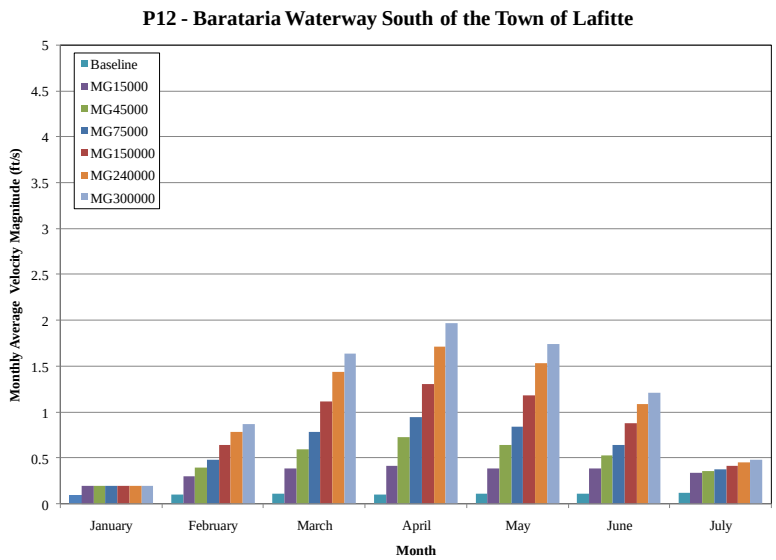


Figure 3-6. Monthly Average and Maximum Water Surface Elevations on Barataria Waterway



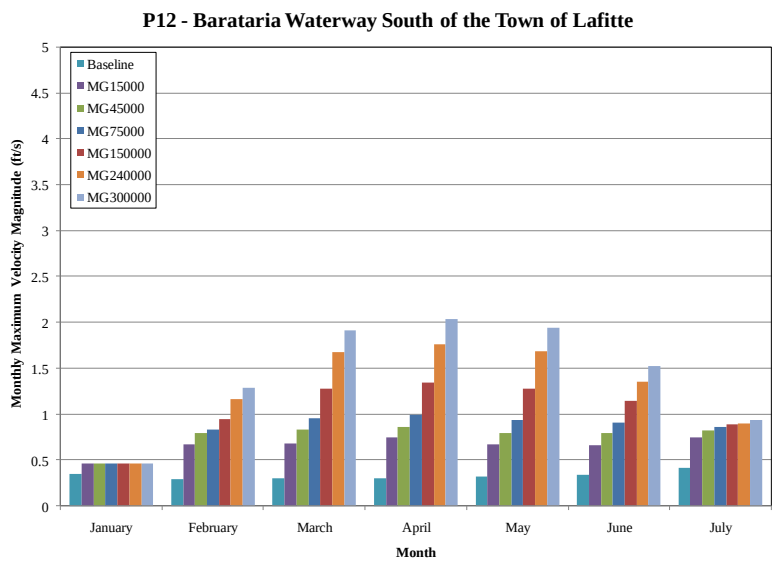


Figure 3-7. Monthly Average and Maximum Velocity Magnitudes on Barataria Waterway

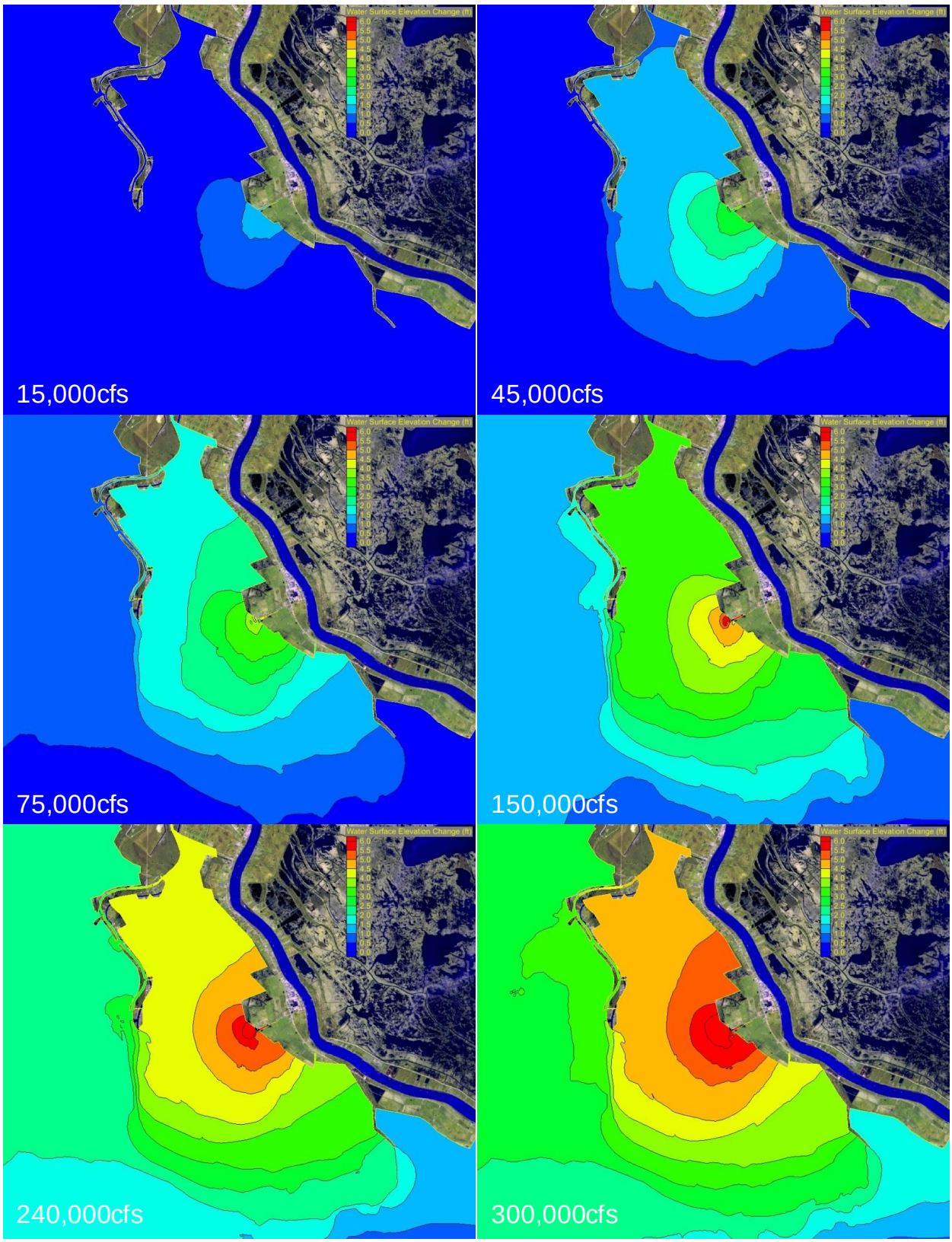


Figure 3-8. Average Water Surface Elevation Change

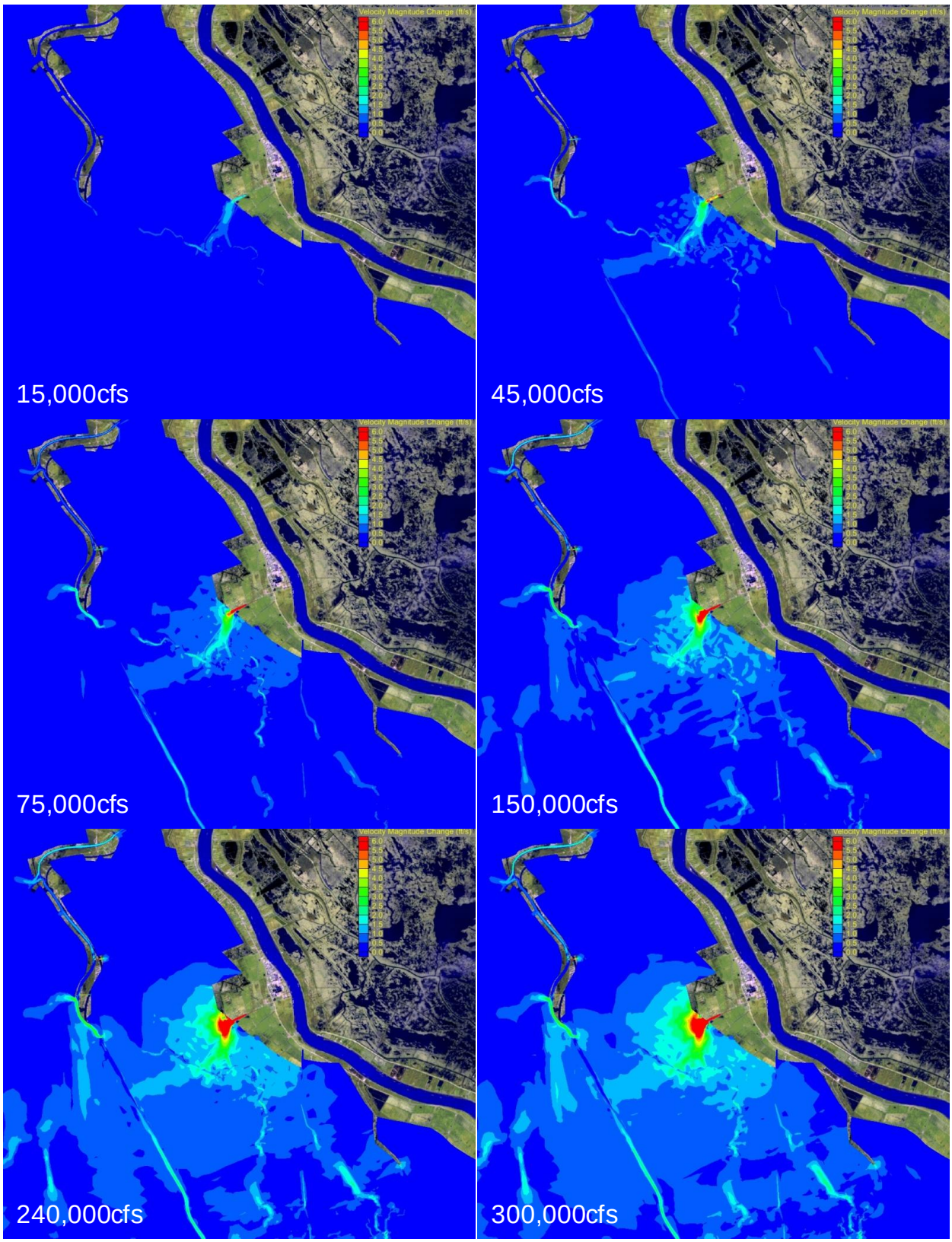


Figure 3-9. Average Velocity Magnitude Change

4 PHASE III – PULSING DIVERSION

Pulsed operation of controlled diversions offer a possible strategy for maintaining a wide range of salinity conditions while at the same time delivering river sediments into a wetland region (Nuttle and Habib, 2008). For example, if diversions are operated alternating between periods of full discharge and zero discharge, then one might expect surface water salinity in the basin to fluctuate between less saline and more saline conditions, depending on the residence time of water in the basin and the frequency with which the diversion discharge is pulsed. Additionally, the duration of elevated water levels and current velocities might be mitigated if the system responds fairly quickly.

Based on the results from Phase II, the hydrodynamic effects of three pulsing diversions were investigated in this phase of the study. The maximum diversion magnitudes are 15,000 cfs; 45,000 cfs and 75,000 cfs and the pulsing interval was set at two weeks with the diversion operating and two week with a minimum diversion flow of 5,000 cfs. Figure 4-1 shows the proposed diversion hydrographs. Wind and rain effects were not considered in these runs.

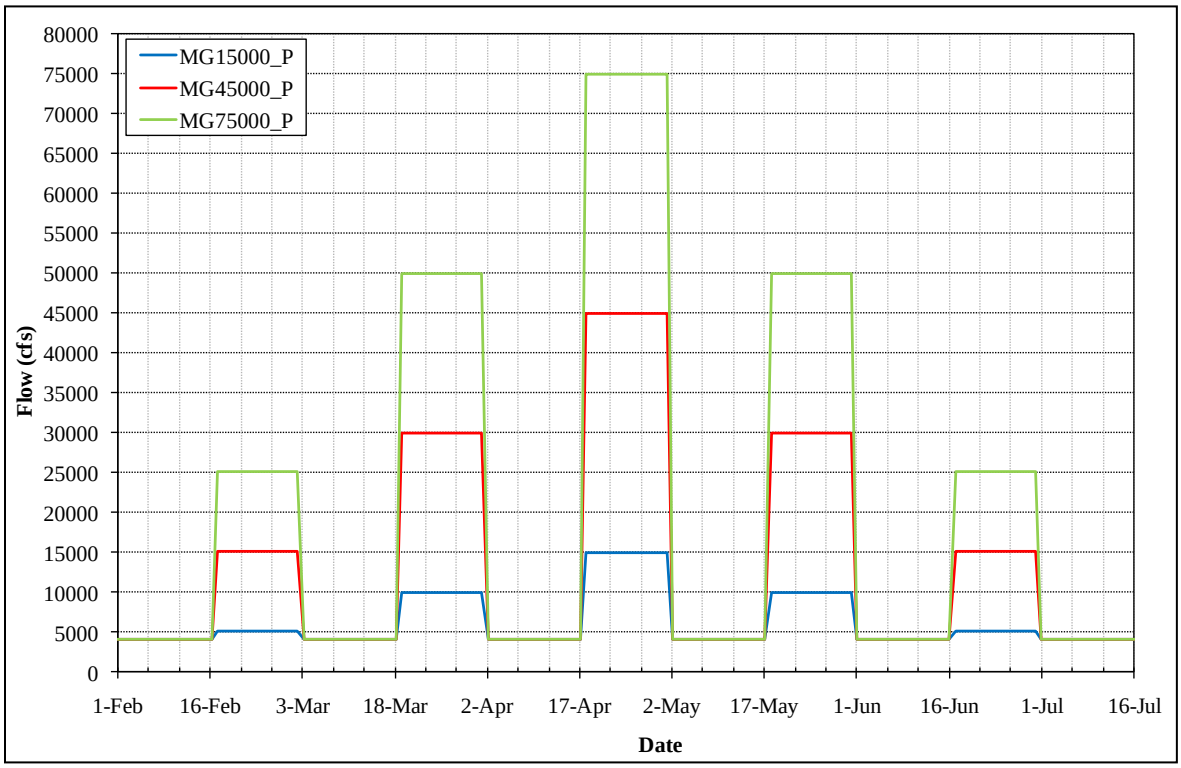


Figure 4-1. Pulsing Diversion Scenarios at Myrtle Grove

The baseline condition was kept the same as used in the Phase II study, and the three pulsing Myrtle Grove diversion scenarios were modeled in conjunction with a nominal David Pond diversion. After the model runs finished, the monthly average, maximum and minimum water surface elevations, as well as the monthly average and maximum current velocity magnitudes for each diversion scenario for the period of February to July were extracted and compared to the baseline condition. The results from the 17 points in Figure 2-5 were also extracted.

Figure 4-2 through Figure 4-4 demonstrate the modeled water surface elevation time series for the three pulsing diversion scenarios at three different locations: P8 – North of Round Lake at Bayou Dupont, P11 – South of Diversion Canal at Bayou Dupont, and P12 – Barataria Waterway South of the Town of Lafitte. The monthly average and maximum water surface elevations and velocity magnitudes at the same locations are shown in Figure 4-5 through Figure 4-10. Figure 4-11 presents the monthly average water surface elevation and velocity magnitude changes relative to the baseline condition caused by different Myrtle Grove diversion magnitudes in April when the diversion reaches its peak.

Review of these results clearly indicates that the system reacts fairly quickly to the diverted flows with water levels and currents rising rapidly upon initiation of the diversion and then decreasing rapidly once the diverted flows are reduced.

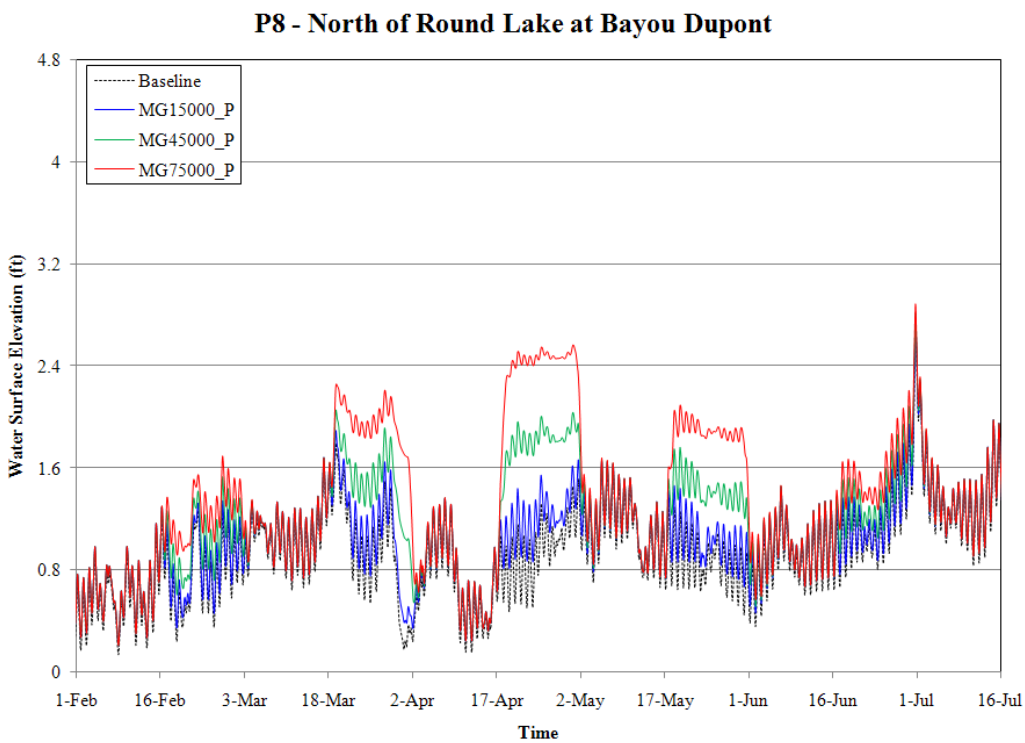


Figure 4-2. Water Surface Elevation near North of Round Lake

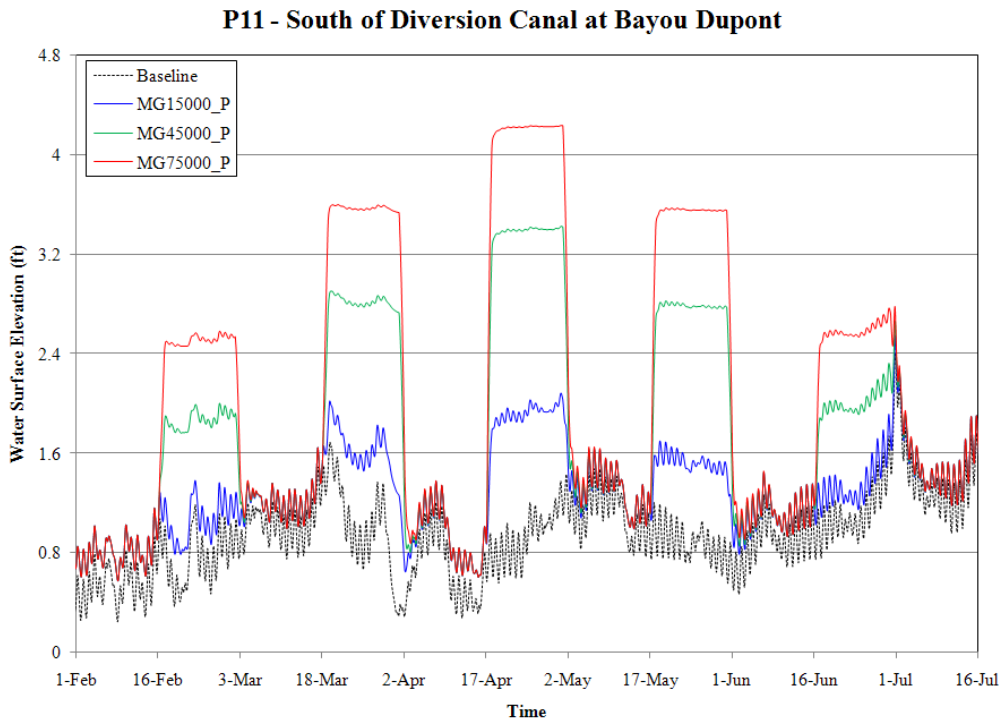


Figure 4-3. Water Surface Elevation near South of Diversion Canal

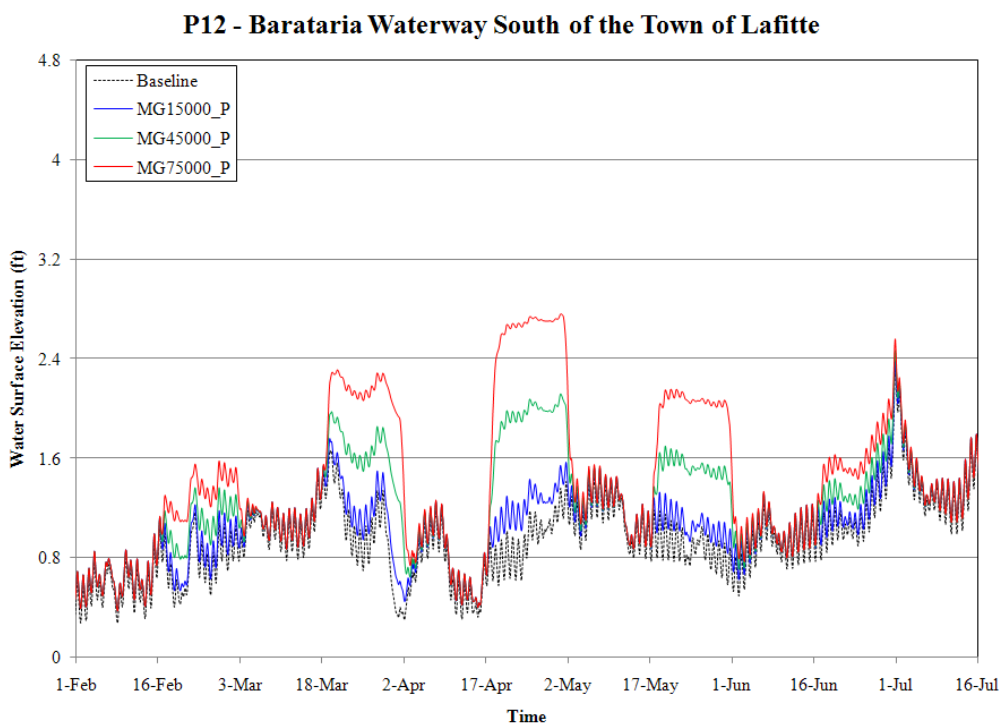
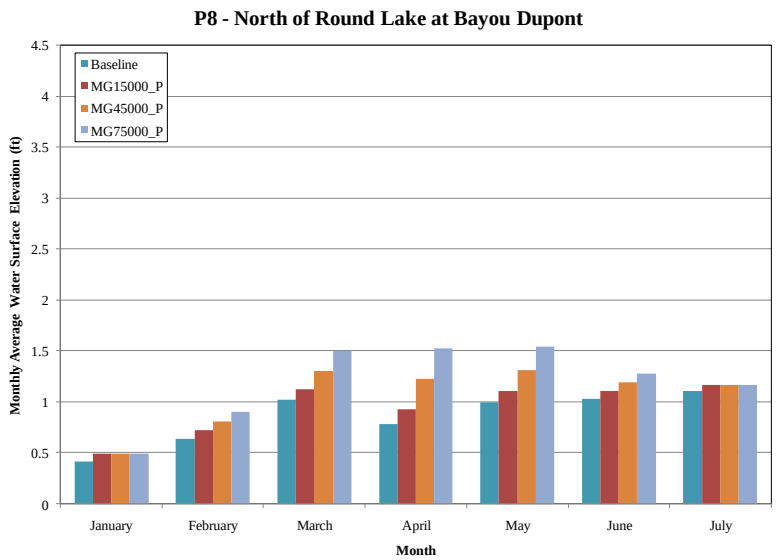


Figure 4-4. Water Surface Elevation on Barataria Waterway



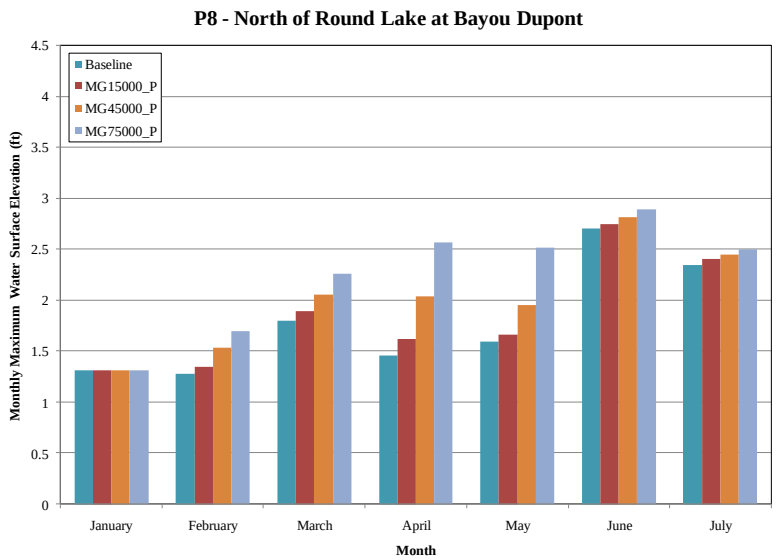
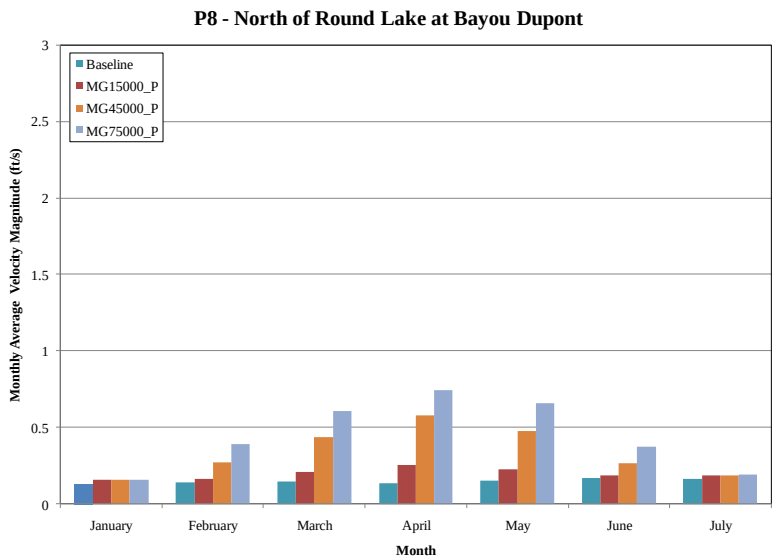


Figure 4-5. Monthly Average and Maximum Water Surface Elevations near North of Round Lake



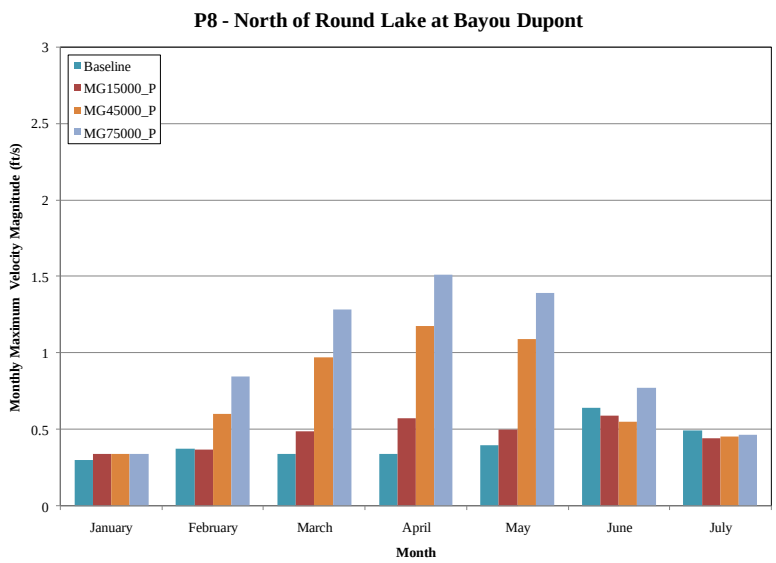


Figure 4-6. Monthly Average and Maximum Velocity Magnitudes near North of Round Lake

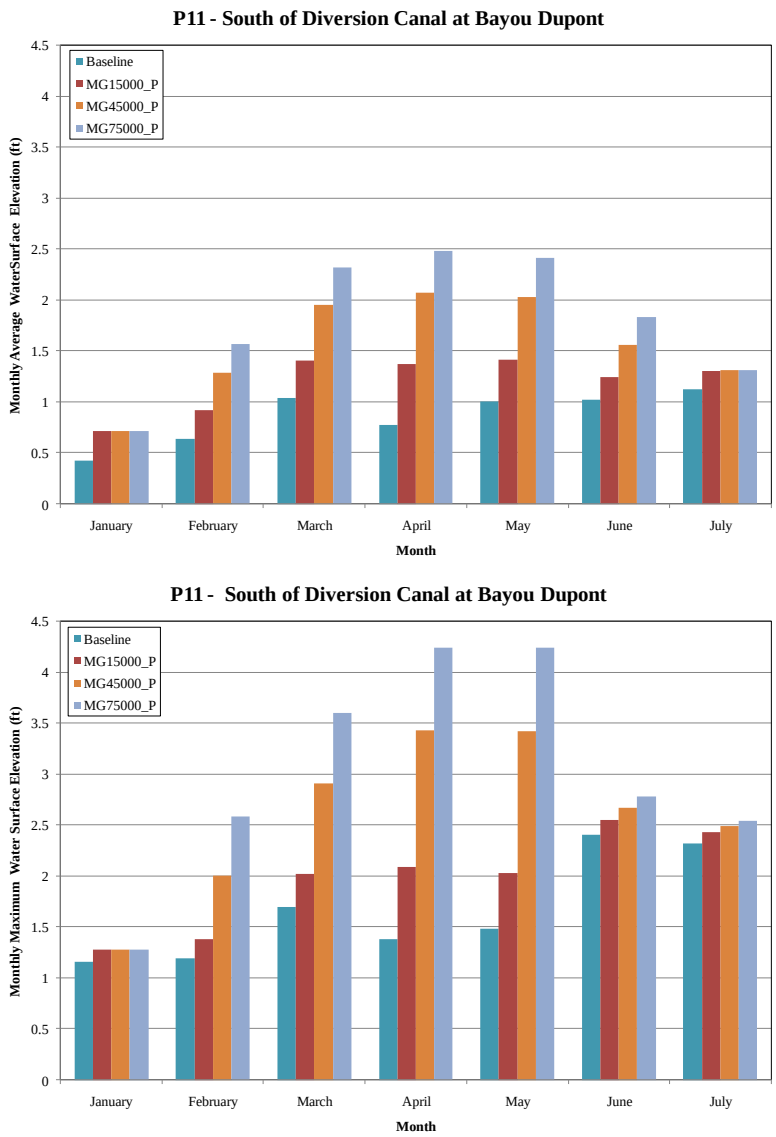
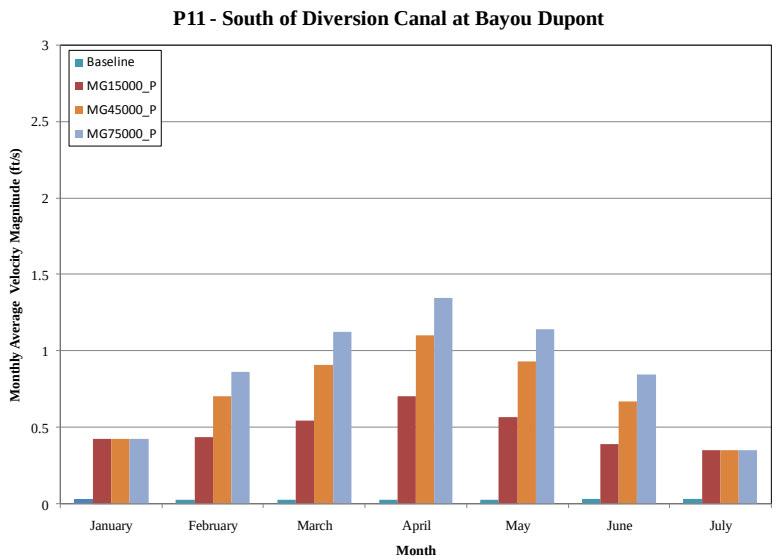


Figure 4-7. Monthly Average and Maximum Water Surface Elevations near South of Diversion Canal



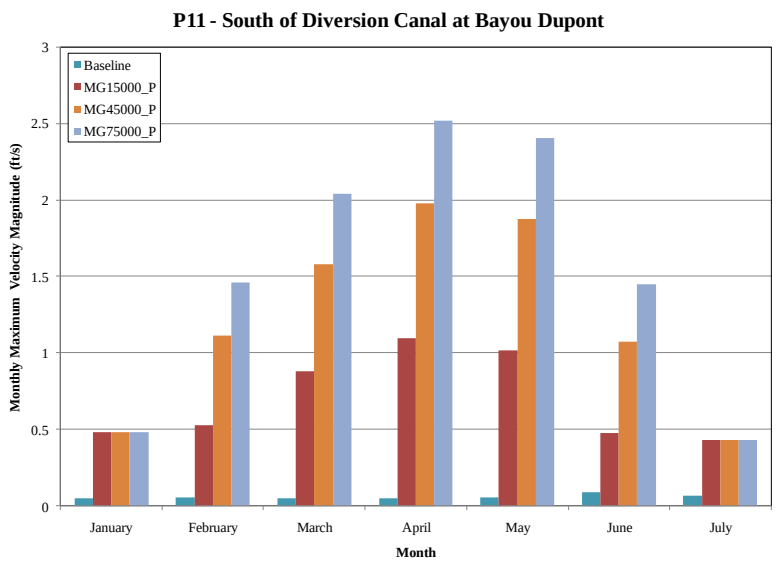


Figure 4-8. Monthly Average and Maximum Velocity Magnitudes near South of Diversion Canal

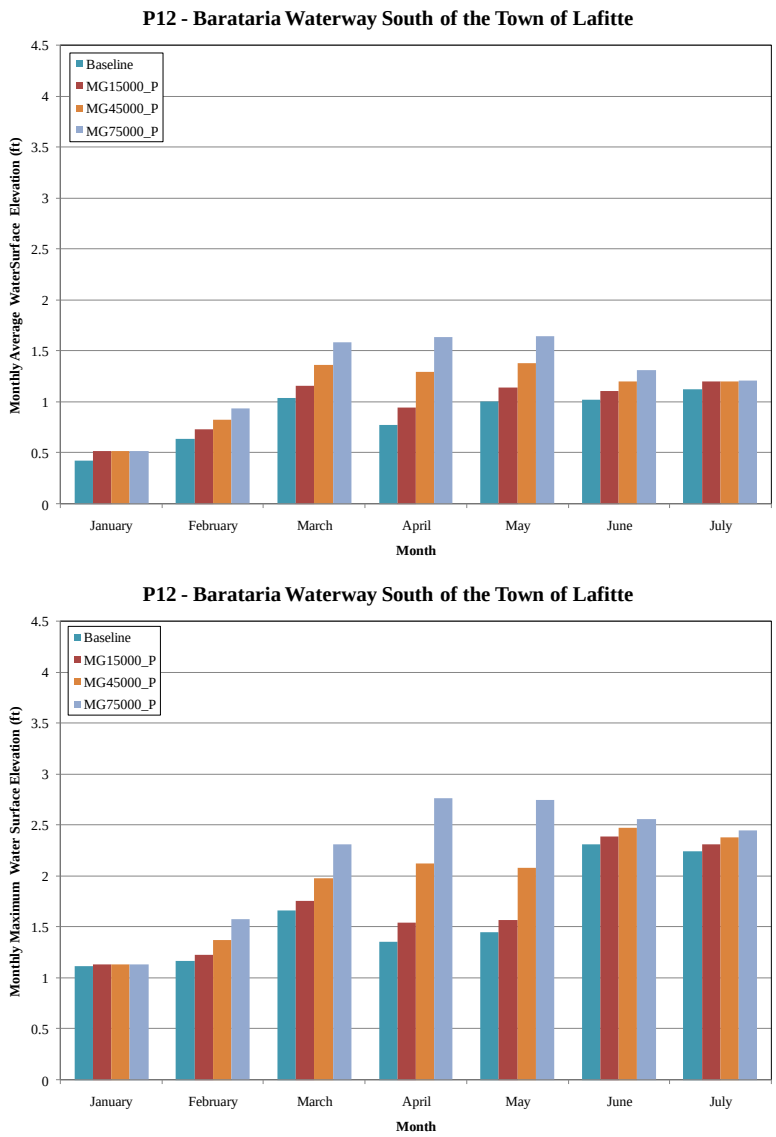


Figure 4-9. Monthly Average and Maximum Water Surface Elevations on Barataria Waterway

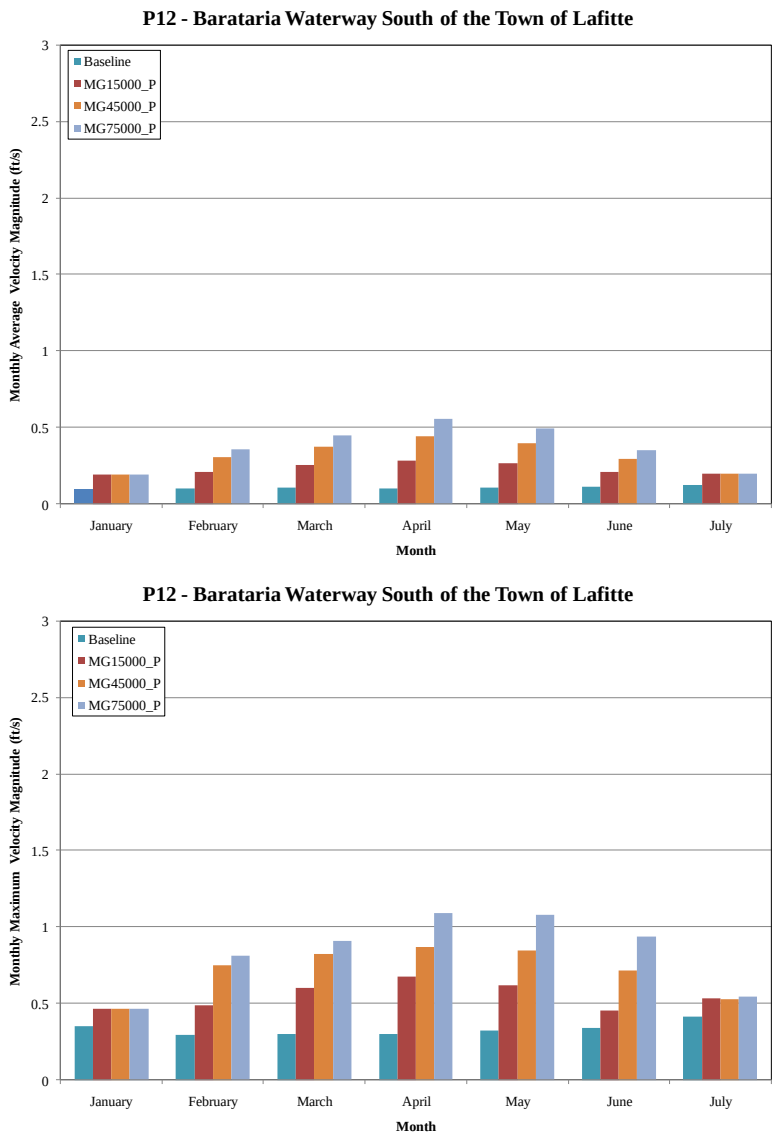


Figure 4-10. Monthly Average and Maximum Velocity Magnitudes on Barataria Waterway

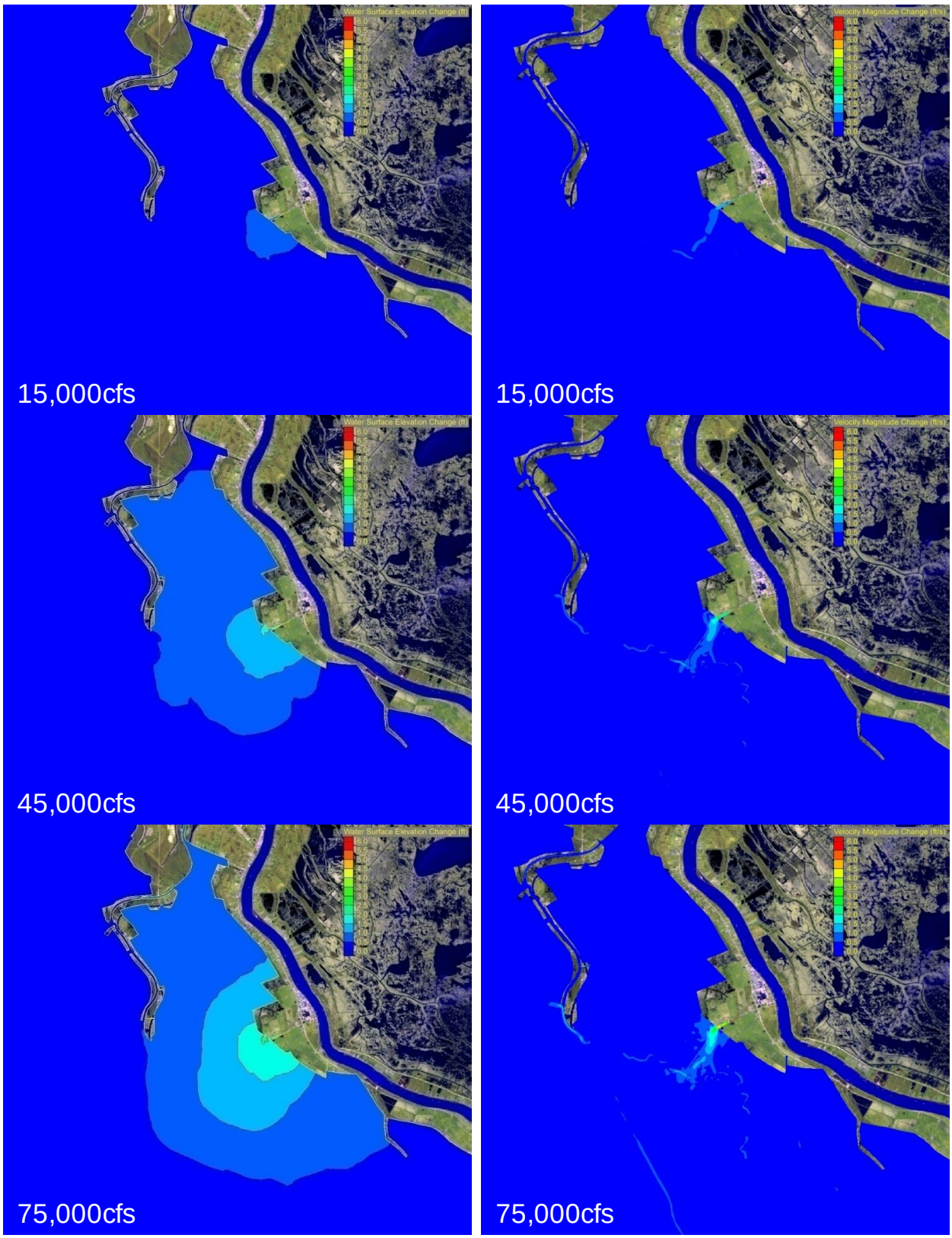


Figure 4-11. Monthly Average Water Surface Elevation and Velocity Magnitude Changes

5 MISSISSIPPI RIVER LONG DISTANCE SEDIMENT PIPELINE

The long distance sediment pipeline (LDSP) project is another land/marsh building project currently being planned in the Barataria Basin. The LDSP would transfer dedicated dredged sediment from the Mississippi River to an "Area of Need" identified in the Barataria Basin within three parishes: Plaquemines, Jefferson and Lafourche.

Although several Coastal Wetlands Planning, Protection, and Restoration Act (CWPPRA), CIAP and Water Resources Development Act (WRDA) projects plan to use river sediments, the LDSP project is unique in terms of pumping distances/volumes and the areal scale of placement sites. Existing approaches to the above-mentioned projects employ conventional bidding strategies where mobilization/demobilization and construction is contracted on a project-by-project basis. In contrast, the LDSP approach will construct multiple projects along a common corridor alignment with a single sediment delivery system. Additionally, the LDSP can be used synergistically to construct multiple projects with a significant savings in overall costs. In addition to these synergistic benefits, the LDSP will complement freshwater diversions, such as the one proposed at Myrtle Grove. The LDSP Project offers significant possibilities in regards to the scale, economy, and speed of coastal restoration in the Barataria Basin.

Part of the study related to the Myrtle Grove diversion is described herein. The entire project is documented in the Moffatt & Nichol September 2010 Report to the Office of Coastal Protection and Restoration (OCPR) titled "Mississippi River Long Distance Sediment Pipeline Louisiana – Overall Planning and Conceptual Development."

The currently preferred pipeline laying approach is to construct ridges using sediment from the pipeline. The ridges will have a crest elevation of +5ft NAVD88. Using the numerical modeling system (RMA-2 and RMA-4) as described previously, the effects of the LDSP ridges on basin hydrology and salinity impacts were examined.

A total of five pipeline corridors alternatives were delineated, each of them crossing different areas within the overall placement sites; see Figure 5-1 (copied from Figure 6-2 in the Moffatt & Nichol September 2010 Report). The five alignments are: Straight Shot, Cheniere Traverse Ridge, Bayou Dupont Ridge, Central Alignment, and Southern Alignment. Only two of the alignments, Cheniere Traverse Ridge (designated as "north ridge") and Southern Alignment (designated as "south ridge"), were investigated further. To represent the ridge scenarios under consideration, the mesh elements along the ridge lines were removed from the computation by setting their material type to zero except for the main waterways which were initially left open. Figure 2-2 shows these waterways along the north and south ridges.

For the runs related to the Myrtle Grove diversion, the one year period from August 2002 to July 2003 as used in the previous phases was simulated. The 45,000 cfs pulsing diversion hydrograph in Figure 4-1 was used at Myrtle Grove. For each different simulated condition, the "with ridge" scenarios are compared with the corresponding "no ridge" scenarios. Monthly maximum water surface elevation changes and monthly maximum salinity changes were extracted to illustrate the potential effects of the ridges on the basin hydrology and salinity.

Under the proposed 45,000 cfs pulsing diversion at Myrtle Grove, the salinity levels in the basin are significantly reduced by the diversion. Adding the north ridge further reduces the salinity 1-2 ppt south of the ridge along the back levee. Figure 5-3 and Figure 5-4 present the monthly maximum salinity change results caused by the north ridge and the south ridge with the Myrtle Grove diversion, respectively. The south ridge traps the diverted freshwater on the north side of the ridge, causing significant localized reductions in salinity at its east end. Conversely, significant increases occur on the south side of the ridge at its east end.

A concern with the proposed Myrtle Grove diversion is that some potential flooding issues may arise in the area between the diversion and Lafitte. Figure 5-5 shows that the water surface elevation could rise significantly in the region above Myrtle Grove in April 2003 when the diversion reaches its highest capacity. Figure 5-6 compares the water level changes caused by the north ridge and the south ridge in April 2003. It shows that the north ridge could reduce the flooding by 0.5ft in the area of concern, while the south ridge would have almost no impact on reducing the water levels.

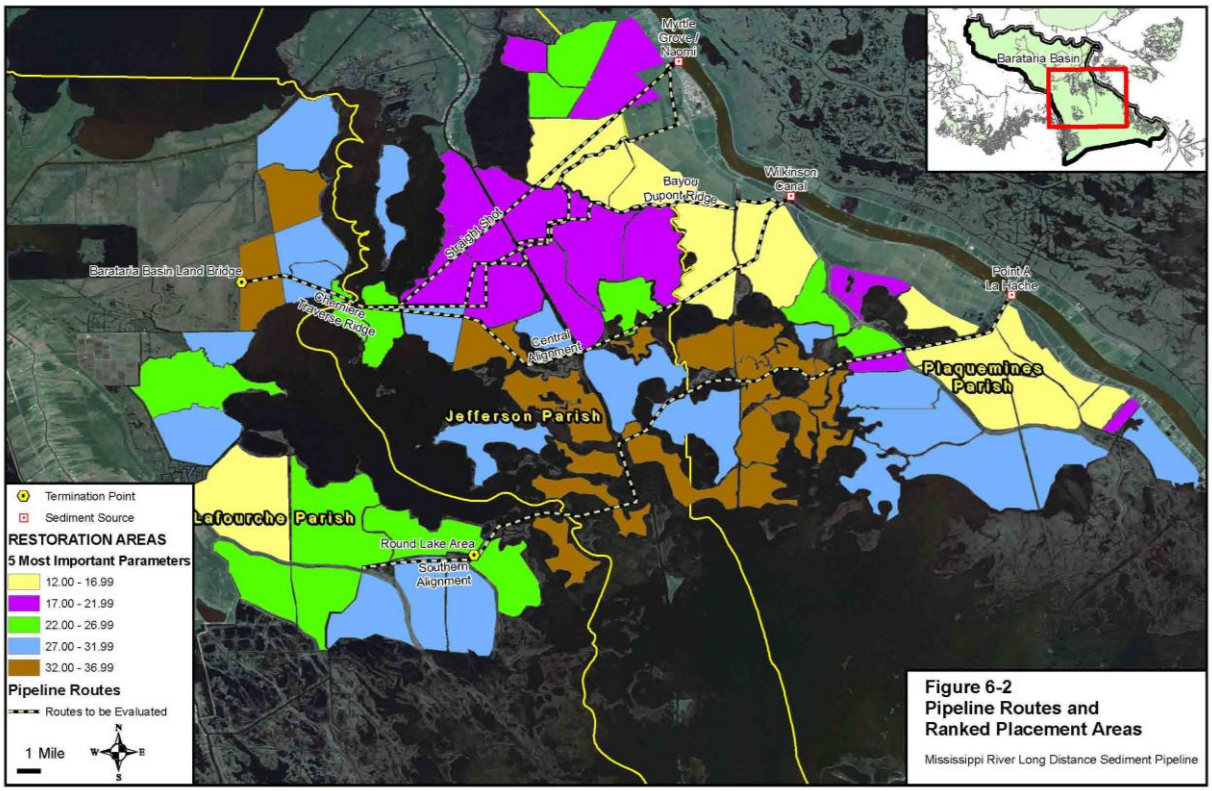


Figure 5-1. Pipeline Routes and Related Placement Areas

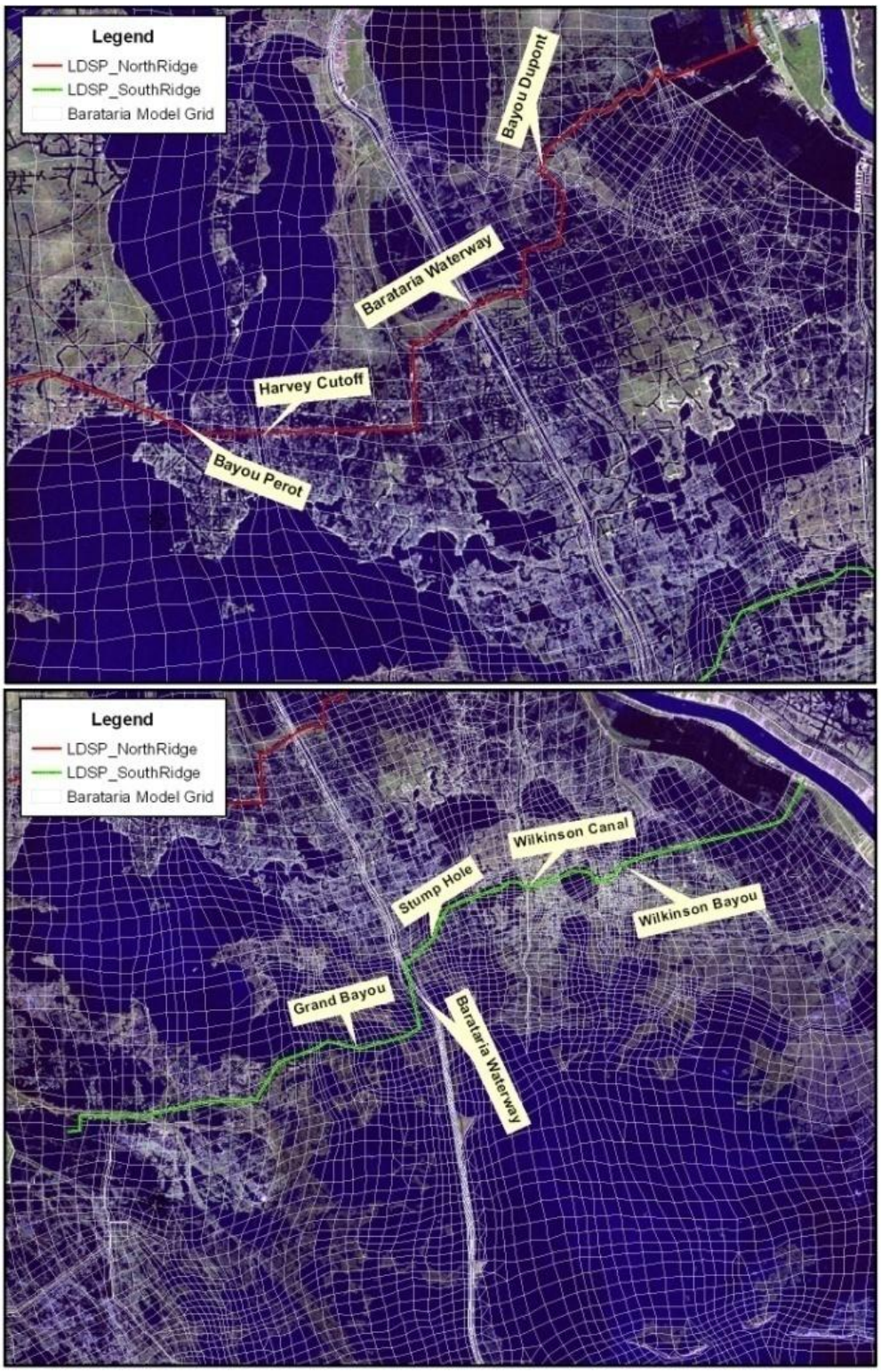


Figure 5-2. Barataria Model Finite Element Grid – Ridges and Waterways

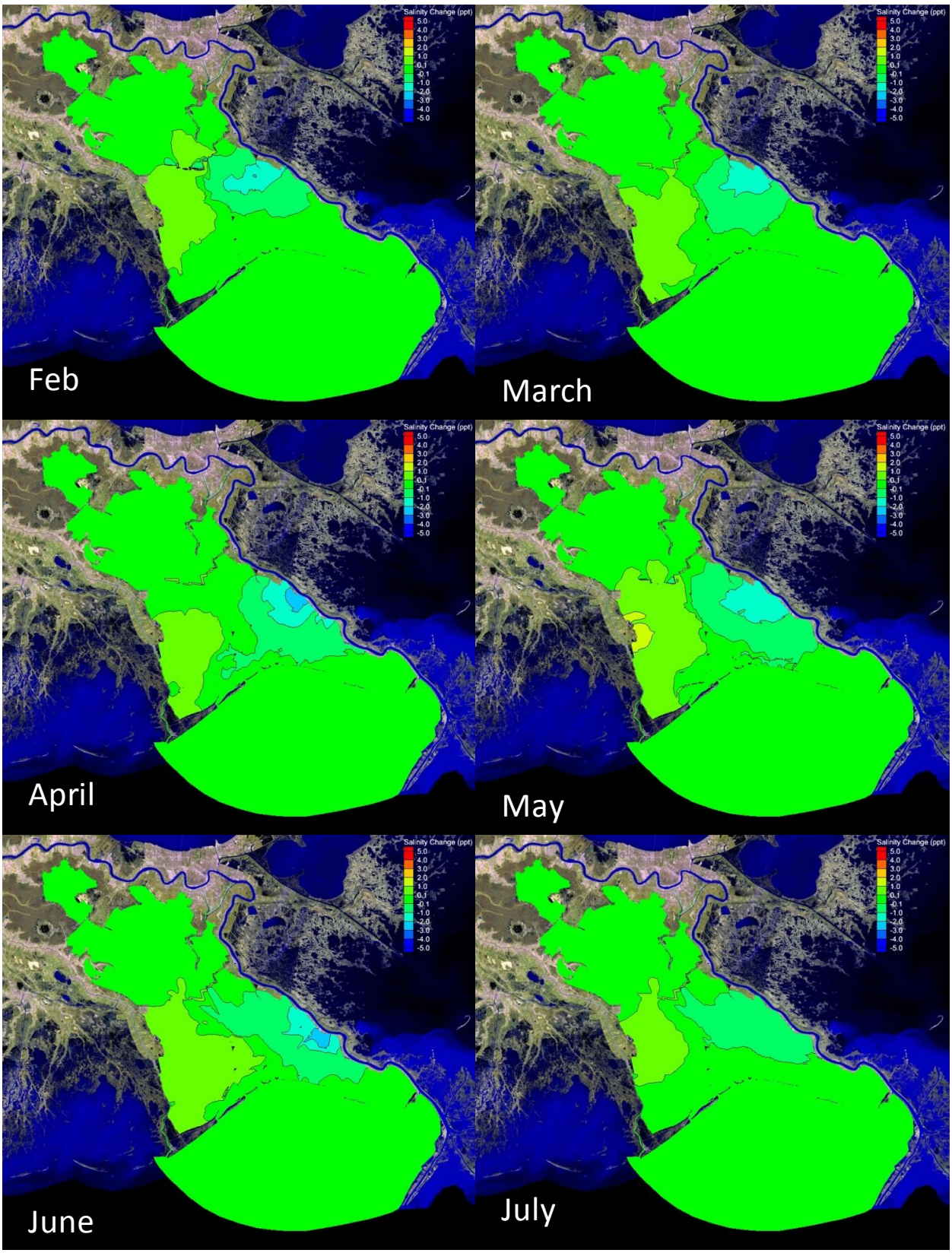


Figure 5-3. Monthly Maximum Salinity Changes Caused by North Ridge

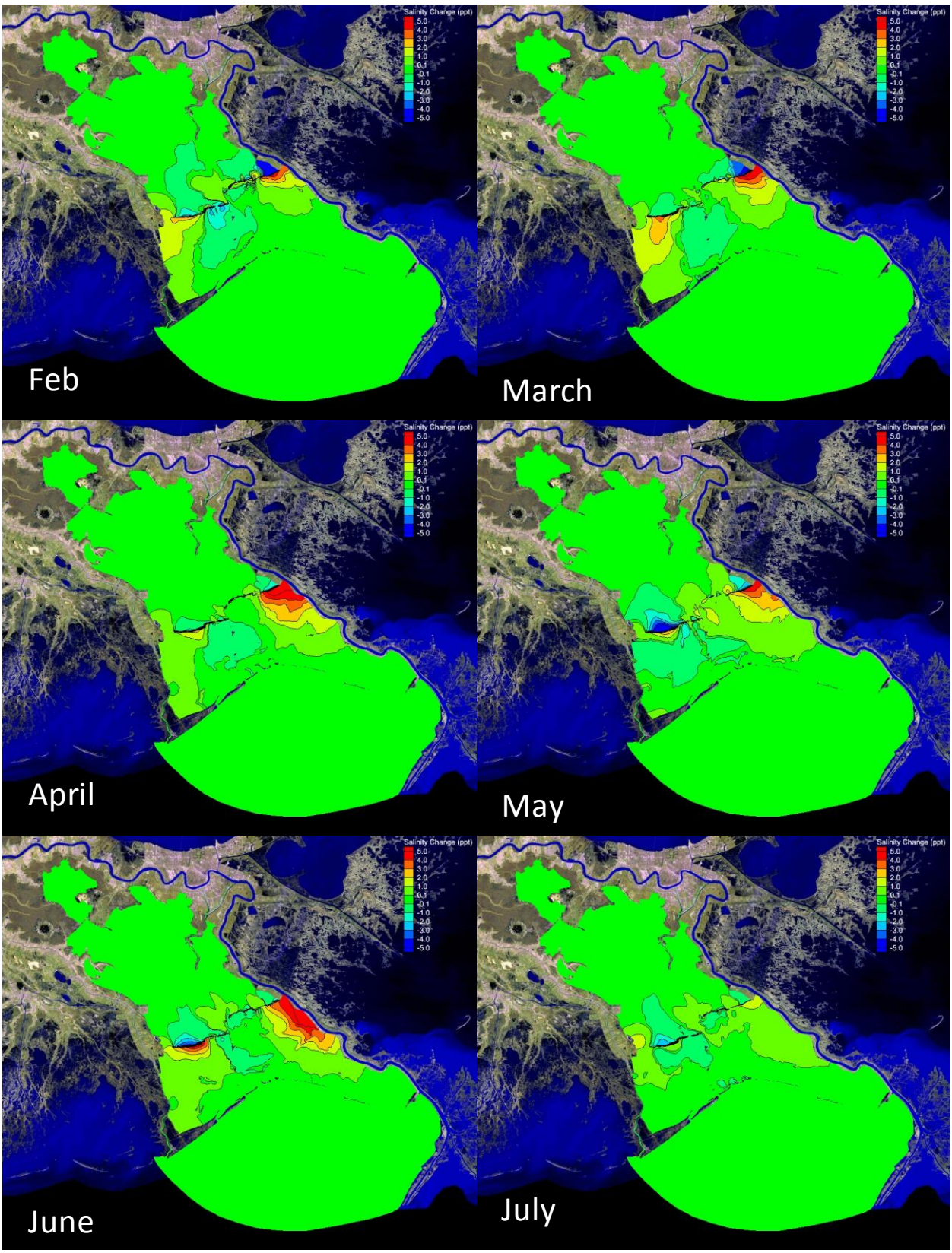


Figure 5-4. Monthly Maximum Salinity Changes Caused by South Ridge

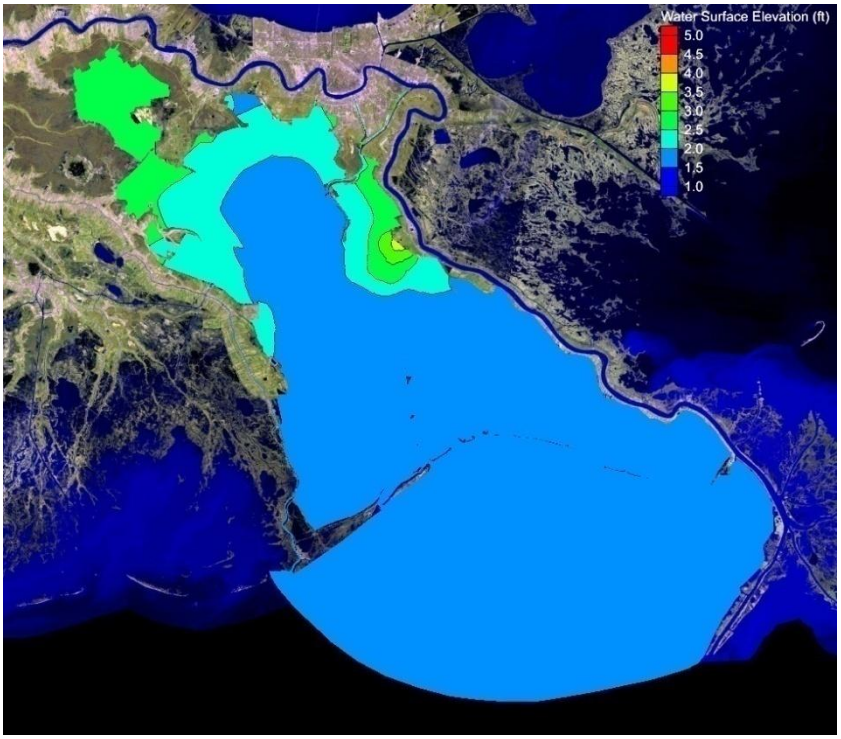


Figure 5-5. Monthly Maximum Water Surface Elevation

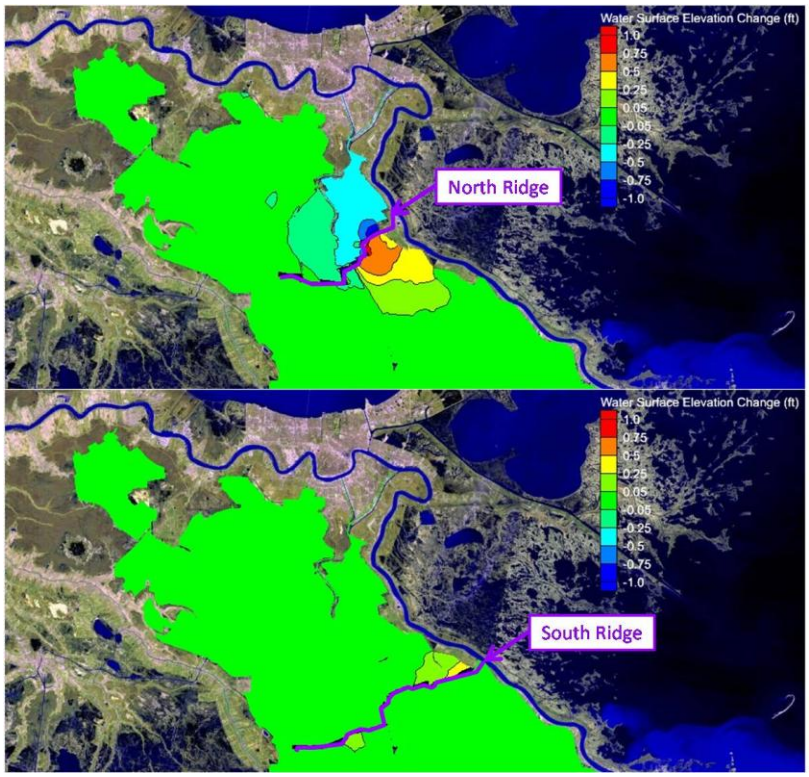


Figure 5-6. Monthly Maximum Water Surface Elevation Changes

6 TAIL WATER AT MYRTLE GROVE DIVERSION

To provide boundary conditions for the diversion structure and channel modeling efforts, hydrodynamic model runs using RMA-2 were conducted to estimate the basin stage as a function of the diverted flow at Myrtle Grove. The diverted flow conditions are: 5,000 cfs, 10,000 cfs, 15,000 cfs, 30,000 cfs, and 45,000 cfs. The offshore water level boundary condition was set to a constant value as 0.7ft NAVD88, the mean water level in April 2003. There were no wind, precipitation and rainfall runoff inputs. For each diversion flow condition, four day periods were run to obtain "steady-state" conditions, and the water levels at the discharge point were extracted from the results. Figure 6-1 presents the tail water levels.

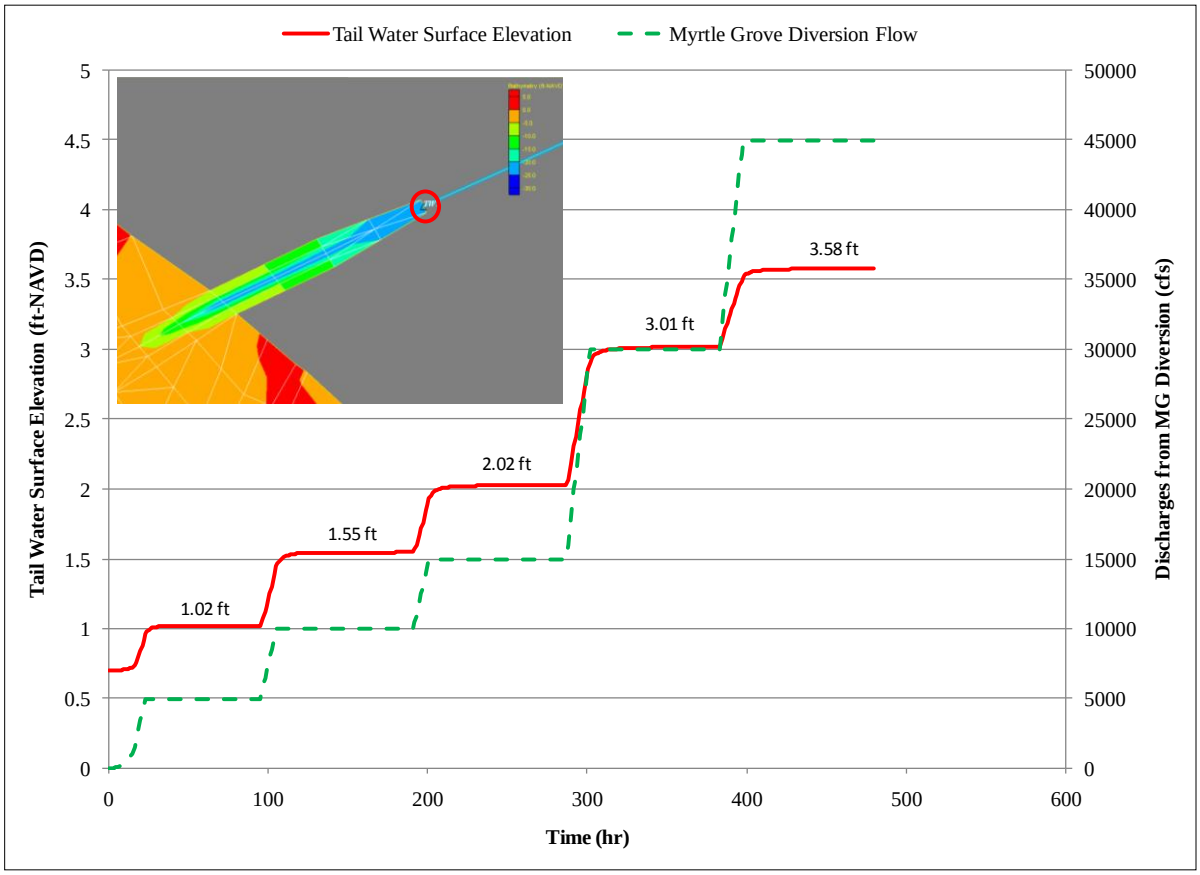


Figure 6-1. Tail Water Levels at Discharge Point

7 MORPHOLOGICAL MODELING

7.1 Introduction

The Delft3D modeling system was used to obtain a general understanding of the potential land-building capacity within the Barataria Basin of the Myrtle Grove Sediment Diversion. The Mississippi River 3D Model developed by others provided sediment loadings through the diversion under different discharge conditions and was used as inputs for the Delft 3D model. The Delft system was then applied to model the sediment transport and morphological changes due to the sediment diversion over a period of 45 years. This allows for a better informed decision as to whether enough sediment can be diverted at Myrtle Grove and delivered a significant distance into the Barataria Basin to build land; or whether the primary benefit of this diversion will remain the introduction of freshwater into the basin.

Delft3D is a 2D/3D modeling system from WL| Delft Hydraulics that can investigate hydrodynamics, sediment transport and morphology and water quality for fluvial, estuarine and coastal environments. The software is used, and has proven its capabilities, on many projects around the world. Delft3D consists of different modules to model the particular physics of the water system and pre- and post processing. The different modules operate as one system with automatic data transfer. The three modules applied in this study include Delft3D-FLOW, Delft3D-SED and Delft3D-MOR.

The Delft3D-FLOW module is the heart of Delft3D and is a multi-dimensional (2D or 3D) hydrodynamic (and transport) simulation program which calculates non-steady flow and transport phenomena resulting from tidal and meteorological forcing on a curvilinear, boundary fitted grid. This module provides the hydrodynamic basis for other modules such as water quality, ecology, waves and morphology. The flow model can be used to predict the flow in shallow seas, coastal areas, estuaries, lagoons, rivers and lakes. It aims to model flow phenomena of which the horizontal length and time scales are significantly larger than the vertical scales.

The sediment transport module, Delft3D-SED, supports both bed-load and suspended load transport of non-cohesive sediments and suspended load of cohesive sediments. A model may contain a mixture of up to 99 suspended fractions if computer memory and simulation time allow. For long-term development of the bottom topography or coastal morphology, the Delft3D-MOR module with advanced on-line coupling capabilities with the flow and wave modules can be applied.

7.2 Model Mesh Development

Delft3D uses curvilinear, structured boundary fitted grids. Due to the complex geometry of the Barataria Basin and the limited computer resources available, it is impractical to use just one fine resolution grid for this land building modeling purpose. Domain decomposition is a technique implemented in Delft3D to divide a model domain into several smaller sub-domains. The subdivision is based on the horizontal and vertical model resolution required for adequately simulating physical processes. Then the computations can be carried out separately on these sub-domains. The communication between the sub-domains takes place along internal open boundaries or so-called dd-boundaries.

Domain decomposition allows for local grid refinement, both in horizontal direction and in vertical direction. Grid refinement in the horizontal direction means that in one sub-domain smaller mesh size (fine grid) are used than in another sub-domain (coarse grid). Decomposition is widely recognized as an efficient and flexible tool for the simulation of complex physical processes. The advantage of a multi-domain modeling approach can be summarized by:

1. Modeling flexibility
    - Coupling of different models
    - Coupling of models of different dimension
    - Coupling of models with different, independently generated sub-grids
2. Modeling accuracy
    - Smooth, structured grids per sub-domain
    - Local grid refinements
3. Efficiency
    - Reduced memory demands by decomposition into smaller sub-domains
    - Parallel execution of sub-domains
    - Better software engineering and maintenance due to modular approach

Of course, there are some restrictions when using domain decomposition in order to have a stable, accurate overall model. Some of these restrictions are listed below:

- At sub-domain interfaces the grids should be nicely connected (no overlap and no holes between sub-domains).
- Grid lines in the coarse domain should be continued in the fine sub-domain. Thus, there should be a 1-to-N refinement, with N an integer number.
- The grid orientation should be the same.
- Sub-domain interfaces should be straight lines (no staircase interface).
- At sub-domain interfaces the depth values in corresponding depth points should be identical in both domains. Furthermore, for the refined grid, it is proposed that the depth values in the intermediate points are determined by linear interpolation.
- The same simulation period and time step should be used in all sub-domains.
- The same processes for transport should be used in all sub-domains.
- No permanently dry points are allowed along sub-domain interfaces.

Figure 7-1 shows the model grids developed for this project. There are a total of 12 sub-domains. C2 represents the Barataria Waterway up to the intersection with the GIWW, which is represented by the C4 grid. I1 is the transition link between C2, C4 and C5 (extension from the GIWW after the intersection with the Barataria Waterway). O1 is the offshore sub-domain where water level boundaries are applied. M1 is the region north of the GIWW, and M2 is the region west of the Barataria Waterway. Five sub-grids represent the area east of the Barataria Waterway. M62a and M62b are two very fine resolution grids representing the areas adjacent to the Myrtle Grove diversion in order to have detailed sediment transport and morphological change patterns. The other three sub-domains are M5, M61 and M63. The grids and domain decomposition linkages were generated using the Delft3D pre-processing module RGFGRID. The pre-processing module QUICKIN was used to obtain the grid bathymetry values. The bathymetry from the RMA-2 model was used to generate the Delft model bathymetry as shown in Figure 7-2.

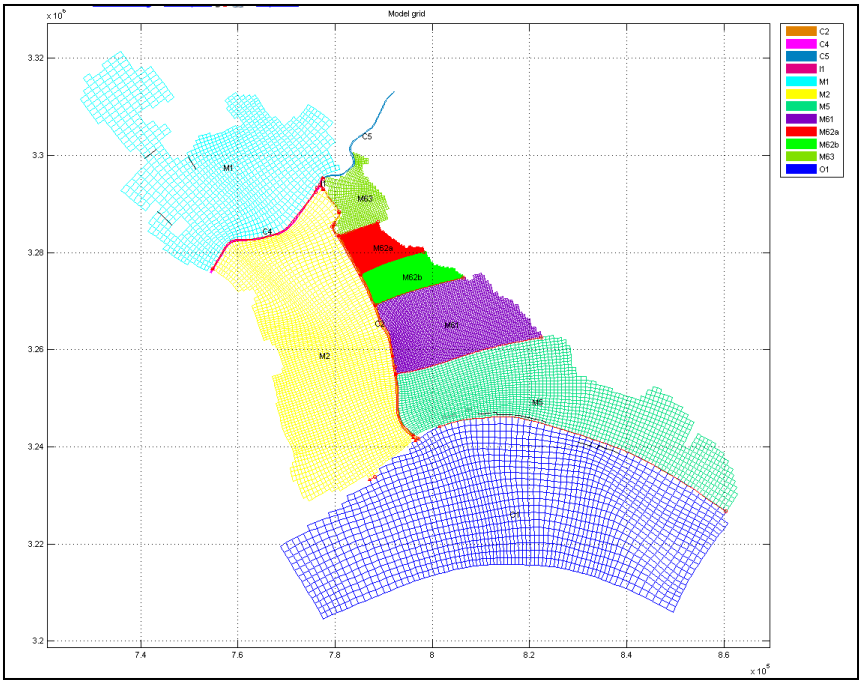


Figure 7-1. Delft3D Model Grid

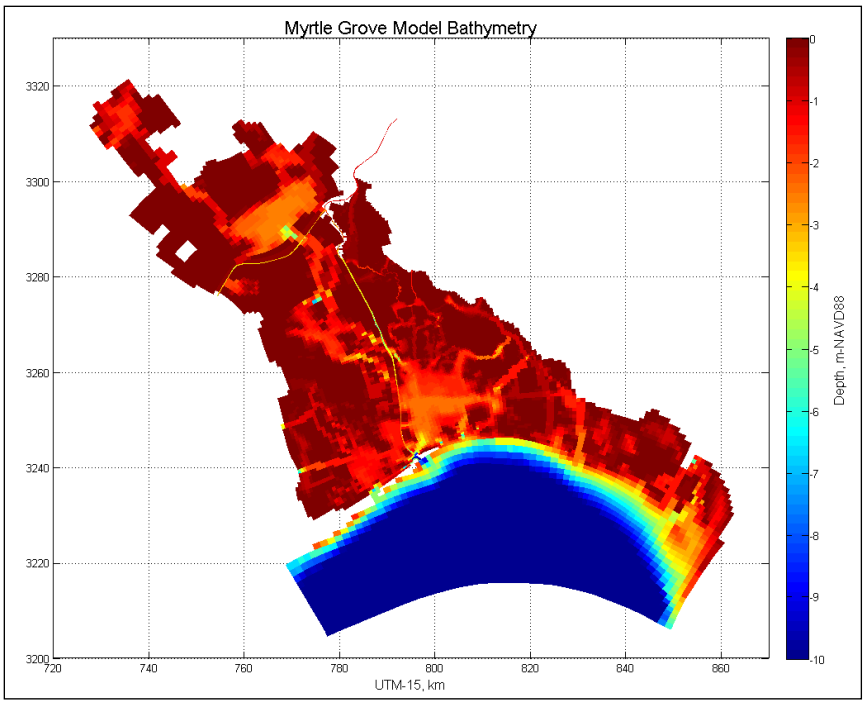


Figure 7-2. Delft3D Model Bathymetry

7.3 Hydrodynamic Model Calibration

As stated previously, the flow module provides the hydrodynamic basis for other modules such as water quality, ecology, waves and morphology. Therefore, it is important to ensure that the hydrodynamics of the basin can be accurately represented by the developed model.

The model was calibrated using the same calibration periods as in the Moffatt & Nichol 2005 study, September 2002 and December 2002. The differences between the Delft model calibrations and RMA-2 calibrations are the wind and precipitation boundary conditions. In the RMA-2 calibrations, the wind and precipitation/evaporation distributions were based on a zone-based approach. However, for the Delft model calibrations, space constant wind and precipitation conditions were applied as it requires significant effort and time to generate space varying wind and precipitation inputs for the Delft model, and based on the scope of this project, it was determined that this effort was not necessary.

Figure 7-3 and Figure 7-4 show the water level gages and discharge cross-sections collected in the Moffatt & Nichol 2005 study. Figure 7-5 and Figure 7-6 present the water level and discharge calibration results, respectively. The Delft model results have good agreement with the measurements.

Figure 7-3. Water Level Gages

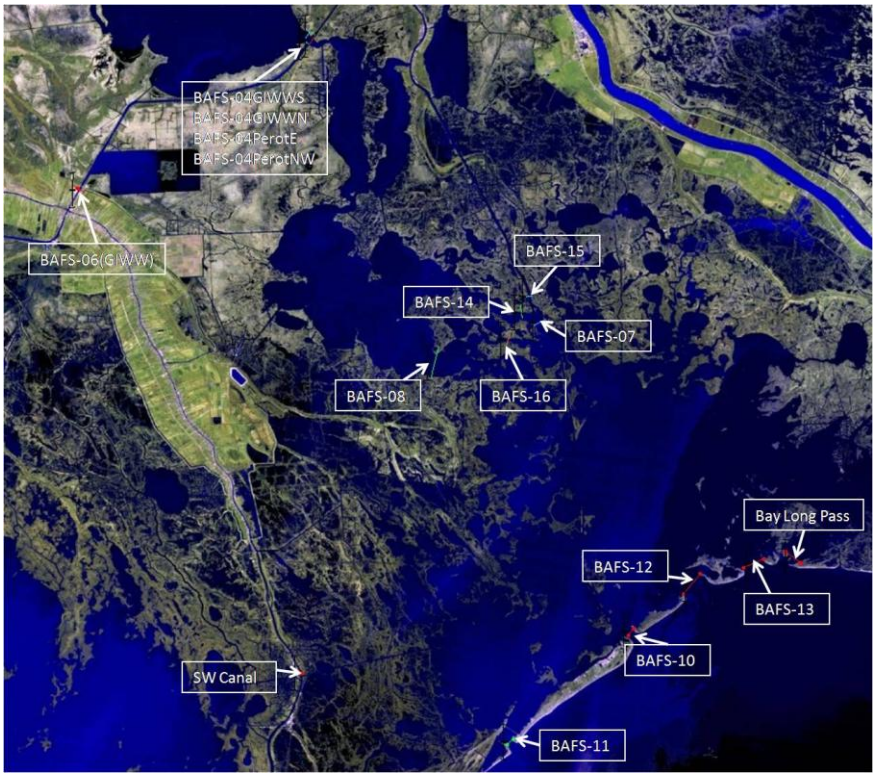


Figure 7-4. Discharge Cross-Sections

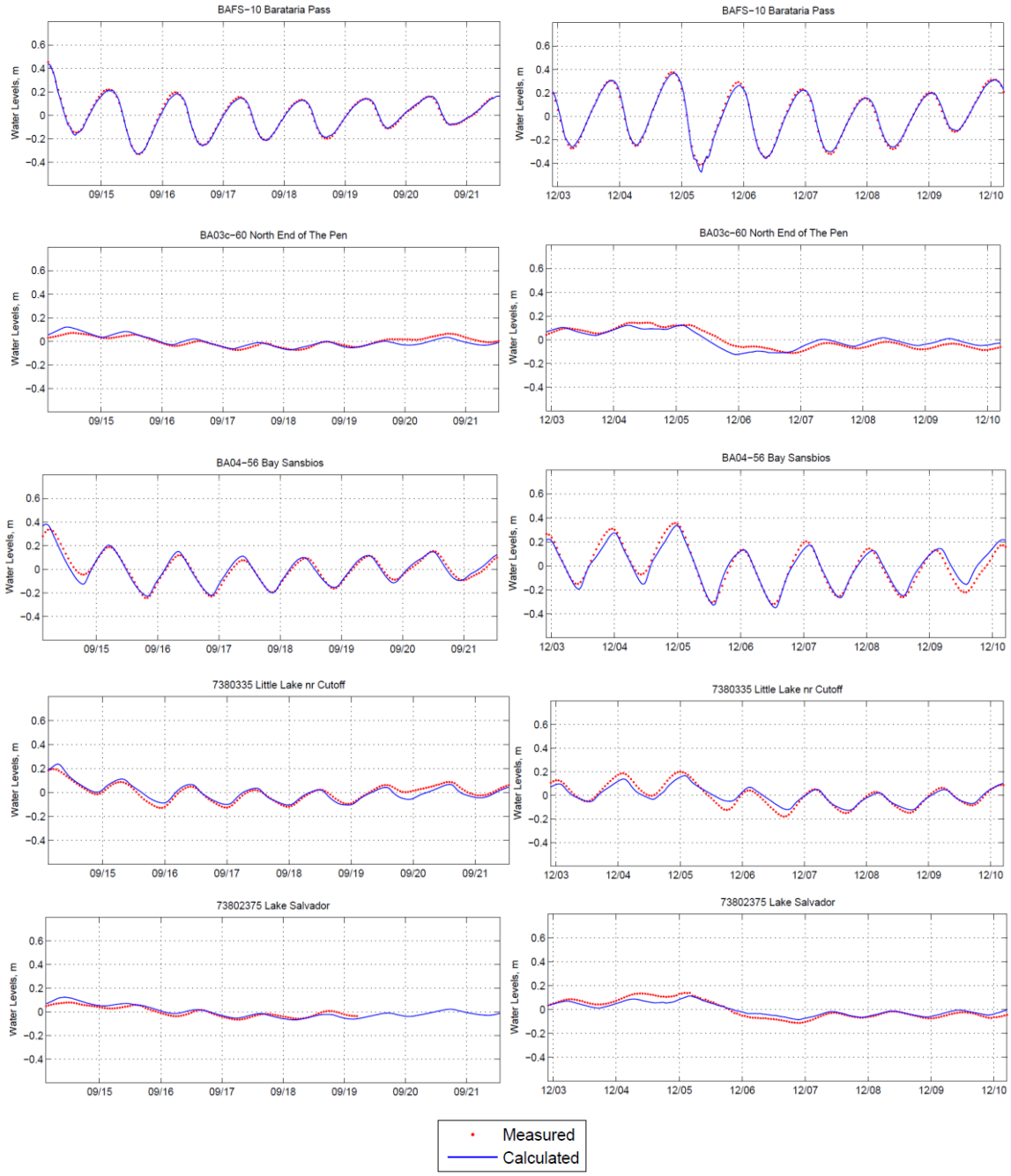


Figure 7-5. Delft Model Water Level Calibration Results

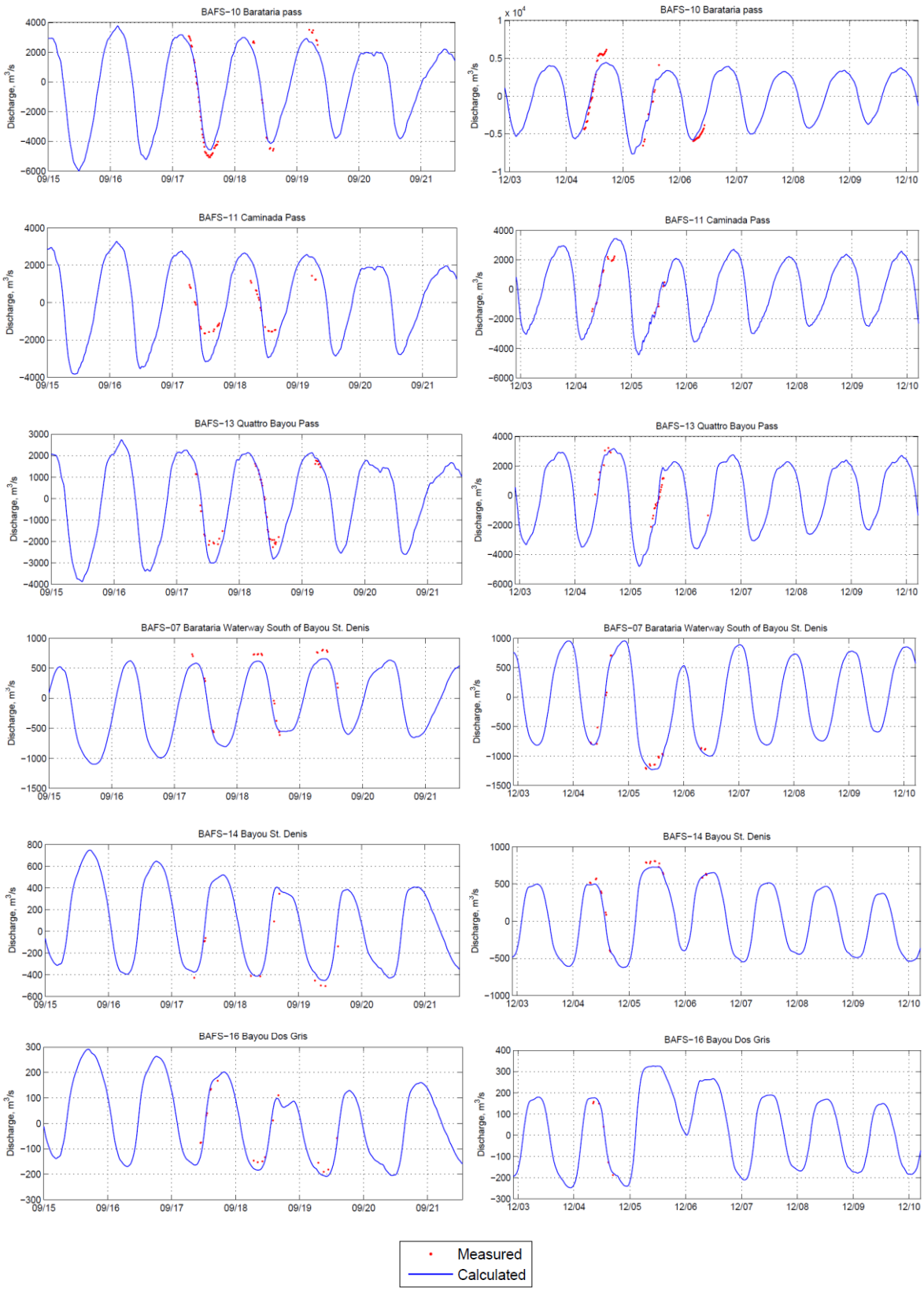


Figure 7-6. Delft Model Discharge Calibration Results

7.4 Sediment Transport and Morphological Modeling

The calibrated hydrodynamic model was on-line coupled with the sediment transport and morphology modules to quantitatively simulate the fate of the diverted sediments from the Myrtle Grove diversion.

Based on the Mississippi River and diversion sediment transport modeling performed by others, three different sediment sizes were included in the Delft simulation: fine sands of 96μm and 64μm, and 32μm silt. To simulate a long term land building process over 20-25 years, the computation time would be 50-63 days if all twelve months in a year were included in the model runs. Additionally, it should be noted that the computation time increases significantly when more sediment fractions are modeled.

Thus, due to the constraints of the project schedule, the model was schematized so that it could be completed in a more timely fashion. The largest proposed diversion flow happens mainly in March, April and May each year when both sand and silt fractions are included. During the remaining nine months, smaller flows with only silt will be diverted. Therefore, to reduce the computation time, these nine month periods were represented by two one-month periods resulting in a total of five months per year actually being numerically modeled. For these two one-month simulations, two morphological time scale factors were used to factor up the sediment deposition such that the five months that were actually modeled can schematically represent an entire year-long real time simulation.

The implementation of the morphological time scale factor in Delft3D is achieved by simply multiplying the erosion and deposition fluxes from the bed to the flow and vice versa by the factor at each computational time step. This allows accelerated bed-level changes to be incorporated dynamically into the hydrodynamic flow calculations.

For the offshore water level boundary conditions, the predicted tide from NOAA station 8761724, Grand Isle, was applied. In order to develop appropriate tide levels for the two one-month periods that were morphologically factored, Figure 7-7 which shows the average seasonal cycle of mean sea level along with each month's 95% confidence interval from the same NOAA station, was reviewed. It was determined that the period from June to November is approximately represented by a one-month period with an average mean sea level close to that which occurs in August, while the period from December to February could be represented by another one-month period with an average mean sea level close to that which occurs in February. Thus, morphological time scale factors of six and three were applied respectively for these two one-month periods in the morphological modeling process. The predicted tides in these two periods were then spliced together with the predicted tides in March, April and May to form a one-year cycle tidal boundary and then repeated each year for the long term land building simulation.

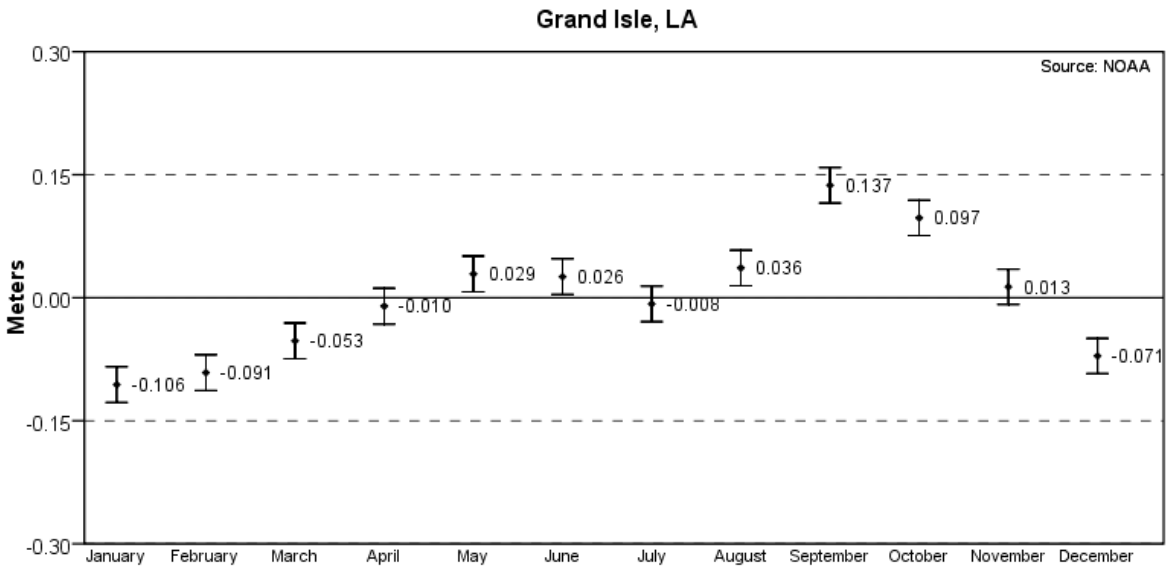


Figure 7-7. Average Seasonal Cycle of MSL at Grand Isle

7.5 Alternative Scenarios

The original USACE channel alignment for the Myrtle Grove diversion consisted of a channel with two bends and having the inflow structure located at River Mile 60.2 AHP. The project team, though, identified alternative alignments that might better capture sediment from the Mississippi River. The first, designated as the "Modified USACE" alignment consisted of a channel with only one bend and having the inflow structure located at River Mile 60.7 AHP. This alignment enters the Barataria Basin at the same location in the back levee as the original USACE channel.

The second alternative, designated "new alignment," also had its inflow structure located at River Mile 60.7 AHP but consisted of a straight channel that entered the Basin slightly south of the other alternatives. The team's opinion was that this new alignment would best capture sediment from the River.

In order to "bracket" the possible results, two alternative scenarios were selected to be modeled. They were the original USACE channel alignment with a maximum 15,000 cfs diversion flow and the new alignment with a maximum 45,000 cfs diversion flow.

A 3-dimensional model of the River, diversion structure and outfall channel was developed by others and run for these two alternatives. A river flow of 700,000 cfs was assumed since this matched with that which occurred when the sediment data collection efforts were undertaken. Based on the conceptual diversion structure, this resulted in an actual peak diversion of 11,369 cfs for the first alternative and 33,735 cfs for the second one. Table 7-1 and Table 7-2 present the sediment load in the two diversion alternatives determined by this numerical modeling done by others.

Table 7-1. Sediment Load in USACE Diversion Alignment (15,000 cfs)

Central Size in Microns	<=2	32	63	96	125	250	>250
Size Range in Microns	<=2	2 -63	63-79	79-113	113-187	187-250	>250
Mean Fractional Sediment Load in River (metric tons / day)	25,805	233,539	10,839	21,816	34,437	23,460	6,564
Water Flow in Diversion (%)	1.62 (River = 700,000 cfs, Diversion = 11,369 cfs)						
Sediment Load in Diversion / River (%)	1.62	1.19	0.96	0.66	0.39	0.01	n/a
Sediment Concentration / River (%)	100	73.64	59.48	40.82	23.76	0.53	n/a
Sediment Load in Diversion (metric tons / day)	418	2,786	104	144	133	2	n/a

Table 7-2. Sediment Load in New Diversion Alignment (45,000 cfs)

Central Size in Microns	<=2	32	63	96	125	250	>250
Size Range in Microns	<=2	2 -63	63-79	79-113	113-187	187-250	>250
Mean Fractional Sediment Load in River (metric tons / day)	25,805	233,539	10,839	21,816	34,437	23,460	6,564
Water Flow in Diversion (%)	4.825 (River = 700,000 cfs, Diversion = 33,735 cfs)						
Sediment Load in Diversion / River (%)	n/a	6.554	6.117	5.639	4.754	0.929	n/a
Sediment Concentration / River (%)	n/a	135.83	126.78	116.87	98.53	19.25	n/a
Sediment Load in Diversion (metric tons / day)	n/a	15,306	663	1,230	1,637	218	n/a

For the DELFT numerical modeling of the land-building potential in the Basin, three sediment sizes were modeled; 32, 63 and 96 microns. The sediment load for the 125 microns sand fraction was added to the load for the 96 micron sand while the clay particles (<2 microns) and coarse sand fractions (>=250 microns) were not included in this analysis.

The assumed hydrograph for the diversion flows consisted of a peak diversion flow for the month of April with all three sediment sizes. For the remaining months, only silt sediments (32 microns) were assumed to be carried with the same concentration as in the peak flow. Table 7-3 and Table 7-4 present the monthly diversion flows, sediment concentrations and sediment loads for the two alternatives modeled.

Table 7-3. Diversion and Sediment Parameters for USACE Alignment (15,000 cfs)

Month	Diversion Flow (cfs)	Sediment Concentration (kg/m^3)			Sediment Load (metric tons/day)		
		32 µm	63 µm	96 µm	32 µm	63 µm	96 µm
January	5,000	0.1	-	-	1,223	-	-
February	5,000	0.1	-	-	1,223	-	-
March	9,000	0.1	-	-	2,202	-	-
April	11,400	0.1	0.0037	0.01	2,789	103	279
May	9,000	0.1	-	-	2,202	-	-
June	5,000	0.1	-	-	1,223	-	-
July	5,000	0.1	-	-	1,223	-	-
August	5,000	0.1	-	-	1,223	-	-
September	5,000	0.1	-	-	1,223	-	-
October	5,000	0.1	-	-	1,223	-	-
November	5,000	0.1	-	-	1,223	-	-
December	5,000	0.1	-	-	1,223	-	-
Total Load: 565,615 metric tons (462,373 cubic yards[1])							

1. Assuming the dry sediment density is 1600 kg/m^3.

Table 7-4. Diversion and Sediment Parameters for New Alignment (45,000 cfs)

Month	Diversion Flow (cfs)	Sediment Concentration (kg/m^3)			Sediment Load (metric tons/day)		
		32 µm	63 µm	96 µm	32 µm	63 µm	96 µm
January	10,000	0.186	-	-	4,551	-	-
February	10,000	0.186	-	-	4,551	-	-
March	18,000	0.186	-	-	8,191	-	-
April	33,735	0.186	0.008	0.0347	15,306	663	2,867
May	18,000	0.186	-	-	8,191	-	-
June	10,000	0.186	-	-	4,551	-	-
July	10,000	0.186	-	-	4,551	-	-
August	10,000	0.186	-	-	4,551	-	-
September	10,000	0.186	-	-	4,551	-	-
October	10,000	0.186	-	-	4,551	-	-
November	10,000	0.186	-	-	4,551	-	-
December	10,000	0.186	-	-	4,551	-	-
Total Load: 2,316,545 metric tons (1,893,704 cubic yards[1])							

1. Assuming the dry sediment density is 1600 kg/m^3.

7.6 Consolidation of Underlying Sediments

An important issue that had to be considered was the consolidation of the existing soils in the Barataria Basin due to the weight of newly diverted sediments. Data for the BA-39 and B-48 Bayou Dupont Marsh Restoration Projects was provided by OCPR. This data was developed by compiling marsh fill settlement estimates for several borings taken for the geotechnical evaluations completed for these projects. It was clear from this data that the underlying material will consolidate significantly due to the weak organic soils near the surface. Where sandy soils are also present, they provide drainage paths for the water thereby increasing the consolidation rate.

An analysis of this data indicated an approximate time varying rate of consolidation as a percentage of the deposition of new material. It was thus assumed that the consolidation would be equal to 19% of the depth of deposition per year in Year 1; 4.75% of the initial depth of deposition per year in Year 2; 1.75% per year in Years 3-5; 0.65% per year in Years 6-10; 0.35% per year in Years 11-15; 0.25% per year in Years 16-25 and 0.1% per year thereafter.

7.7 Results

7.7.1 Without Consolidation

Initially, two runs were performed without considering consolidation of the underlying soils. It is noteworthy that these modeling results also do not include the effects of subsidence, sea level rise or the adaptation of the marsh to such effects.

Figure 7-8 to 7-10 present the modeling results for the total depth of material deposited after 10, 25 and 45 years, respectively for the 15,000 cfs USACE alignment alternative without consideration of underlying soil consolidation. Figure 7-11 presents the initial bed elevation assumed in the model and Figure 7-12 to 7-14 present the modeling results for the resulting bed elevation after 10, 25 and 45 years, respectively. Table 7-5 to 7-7 present the deposition volume for each 0.5 ft deposition depth interval after 10, 25 and 45 years, respectively.

It is also important to note that these modeling results do not account for the decrease in efficiency of the diversion channel as the land on the basin end of the channel builds up; thereby reducing the potential head differential between the basin and the Mississippi River. Maintenance of this area, consisting of excavation / dredging, will be required to achieve the total potential volume of deposition shown in these results. Hence the largest magnitudes of deposition depth and bed elevations shown in these figures will not be reached as this material would be removed and placed elsewhere.

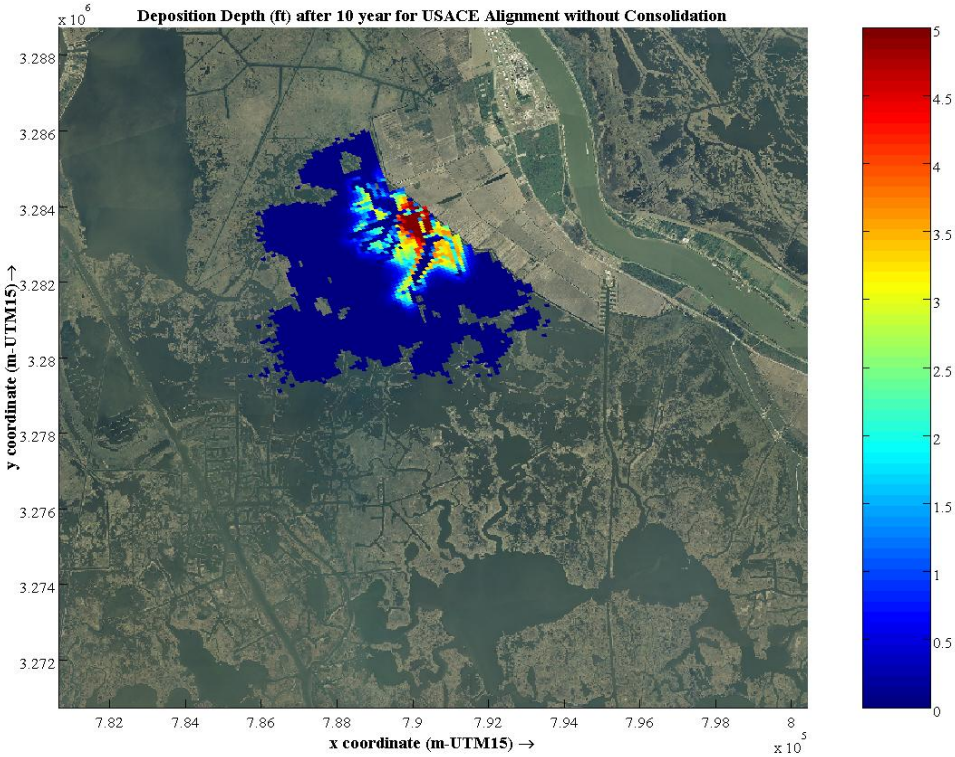


Figure 7-8. Deposition Depth (ft) after 10 years for USACE Alignment (15,000 cfs) without Consolidation

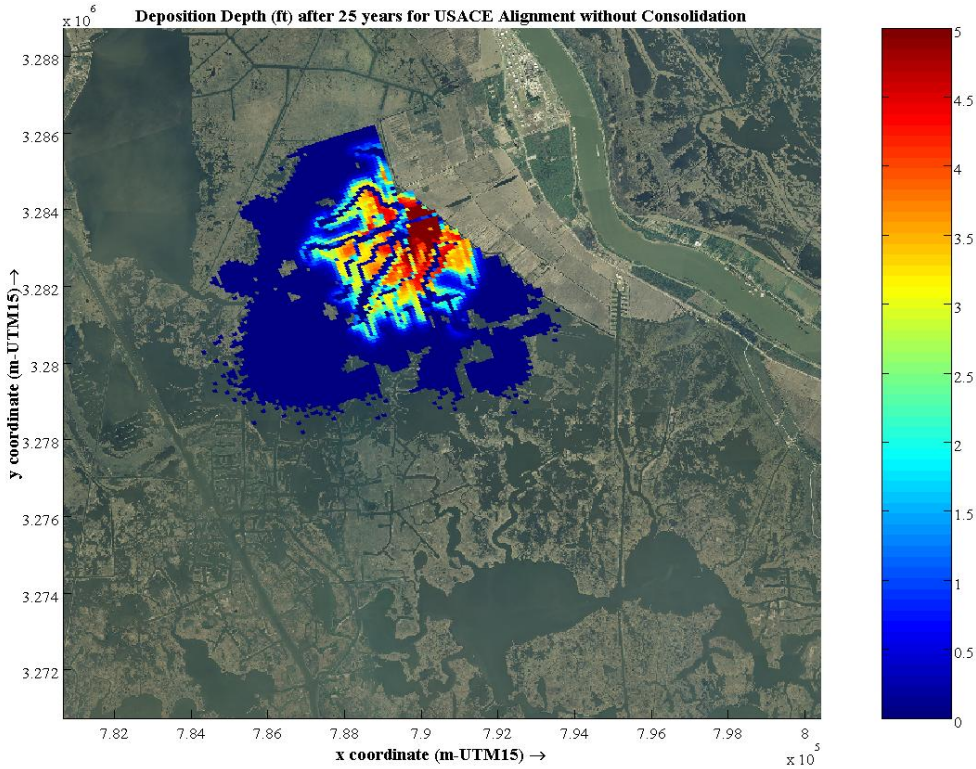


Figure 7-9. Deposition Depth (ft) after 25 years for USACE Alignment (15,000 cfs) without Consolidation

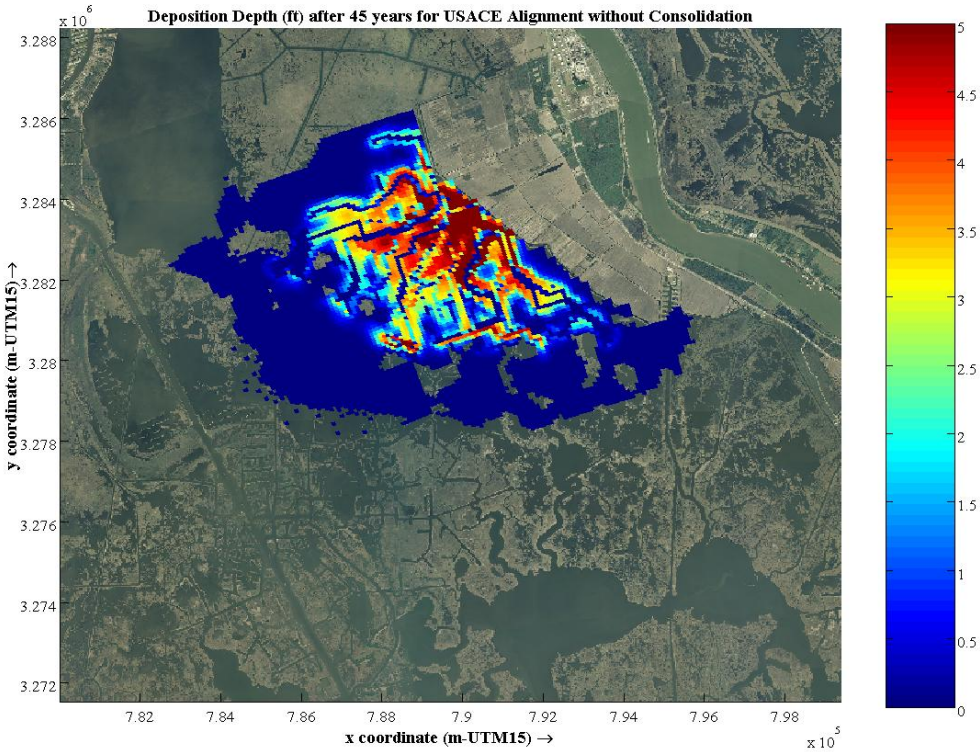


Figure 7-10. Deposition Depth (ft) after 45 years for USACE Alignment (15,000 cfs) without Consolidation

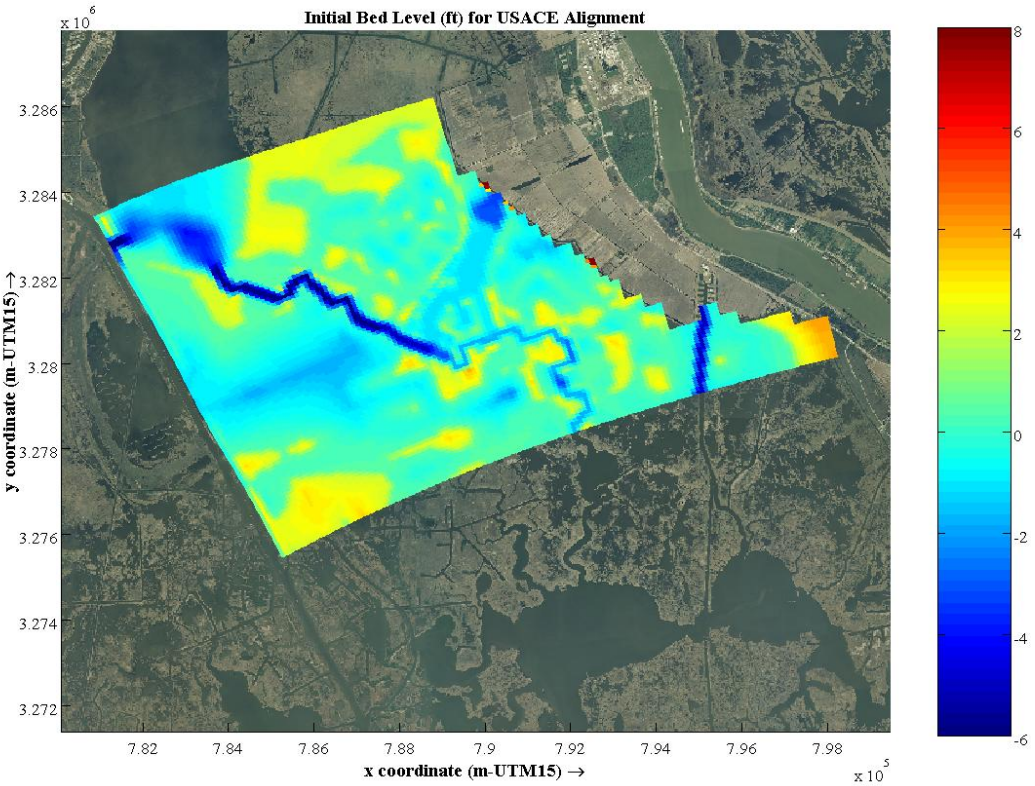


Figure 7-11. Initial Bed Level (ft) for USACE Alignment (15,000 cfs)

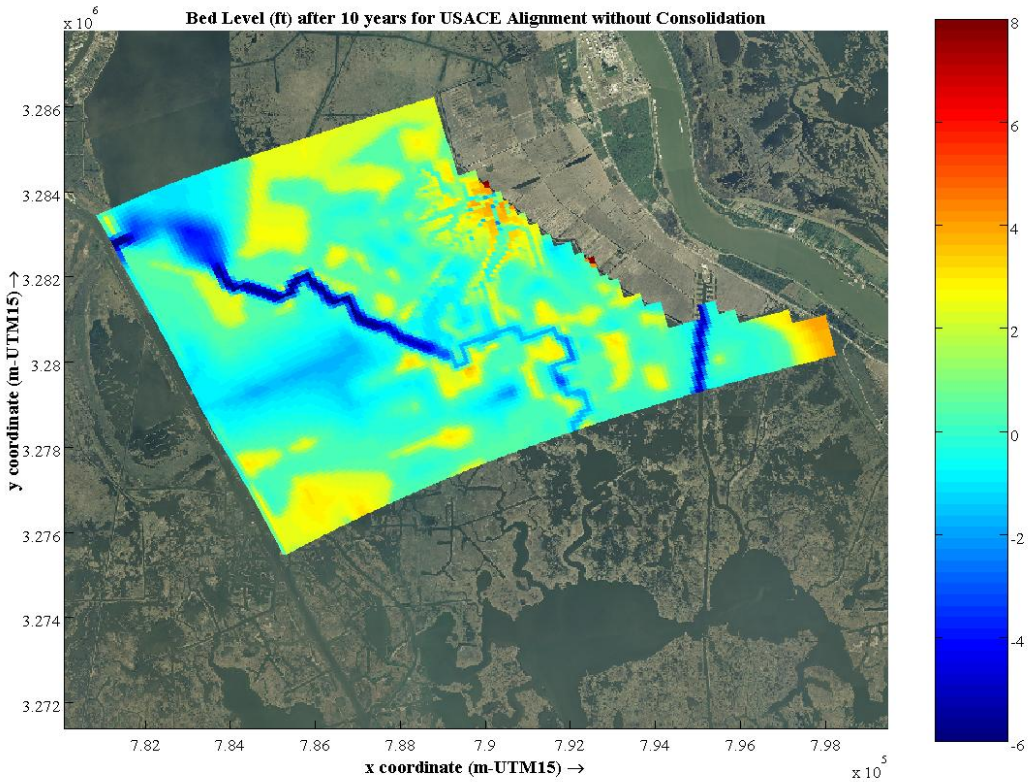


Figure 7-12. Bed Level (ft) after 10 years for USACE Alignment (15,000 cfs) without Consolidation

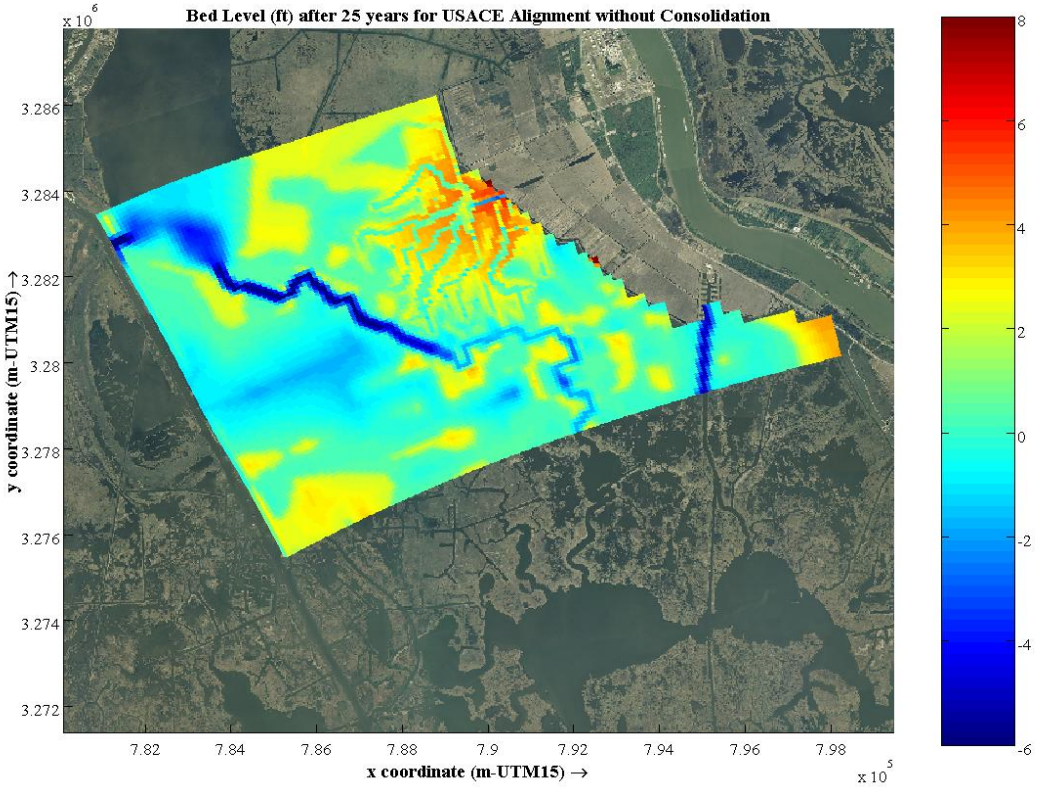


Figure 7-13. Bed Level (ft) after 25 years for USACE Alignment (15,000 cfs) without Consolidation

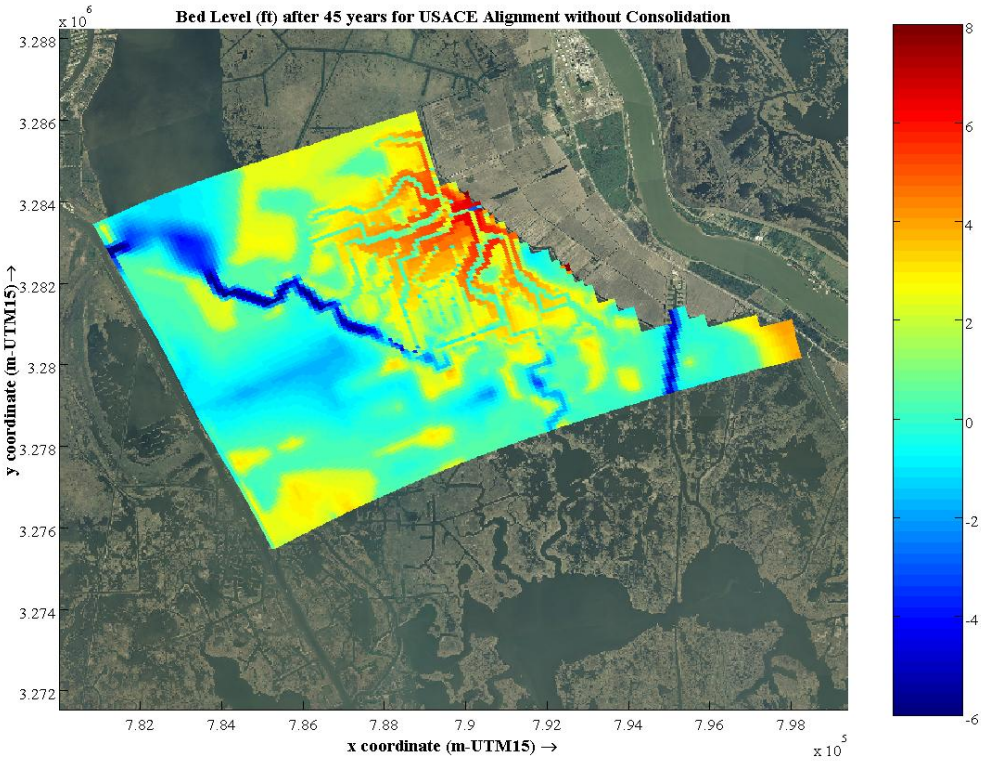


Figure 7-14. Bed Level (ft) after 45 years for USACE Alignment (15,000 cfs) without Consolidation

Table 7-5. Deposition Results after 10 years for USACE Alignment (15,000 cfs) without Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	1,291,647
0.5 – 1.0 ft	869,906
1.0 – 1.5 ft	660,142
1.5 – 2.0 ft	514,944
2.0 – 2.5 ft	386,135
2.5 – 3.0 ft	268,744
3.0 – 3.5 ft	164,482
3.5 – 4.0 ft	101,484
4.0 – 4.5 ft	72,629
4.5 – 5.0 ft	55,479
> 5.0ft	79,087
Total	4,464,680

Table 7-6. Deposition Results after 25 years for USACE Alignment (15,000 cfs) without Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	2,746,189
0.5 – 1.0 ft	2,117,059
1.0 – 1.5 ft	1,704,023
1.5 – 2.0 ft	1,370,322
2.0 – 2.5 ft	1,045,105
2.5 – 3.0 ft	811,491
3.0 – 3.5 ft	579,710
3.5 – 4.0 ft	345,495
4.0 – 4.5 ft	196,238
4.5 – 5.0 ft	112,254
> 5.0ft	289,399
Total	11,317,286

Table 7-7. Deposition Results after 45 years for USACE Alignment (15,000 cfs) without Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	4,671,757
0.5 – 1.0 ft	3,660,220
1.0 – 1.5 ft	3,013,607
1.5 – 2.0 ft	2,486,276
2.0 – 2.5 ft	1,980,827
2.5 – 3.0 ft	1,518,829
3.0 – 3.5 ft	1,083,558
3.5 – 4.0 ft	734,875
4.0 – 4.5 ft	477,780
4.5 – 5.0 ft	293,009
> 5.0ft	591,802
Total	20,512,540

Figure 7-15 to 7-17 present the modeling results for the total depth of material deposited after 10, 25 and 45 years, respectively for the 45,000 cfs new alignment alternative without consideration of underlying soil consolidation. Figure 7-18 to 7-20 present the modeling results for the resulting bed elevation after 10, 25 and 45 years, respectively. Table 7-8 to 7-10 present the deposition volume for each 0.5 ft deposition depth interval after 10, 25 and 45 years, respectively.

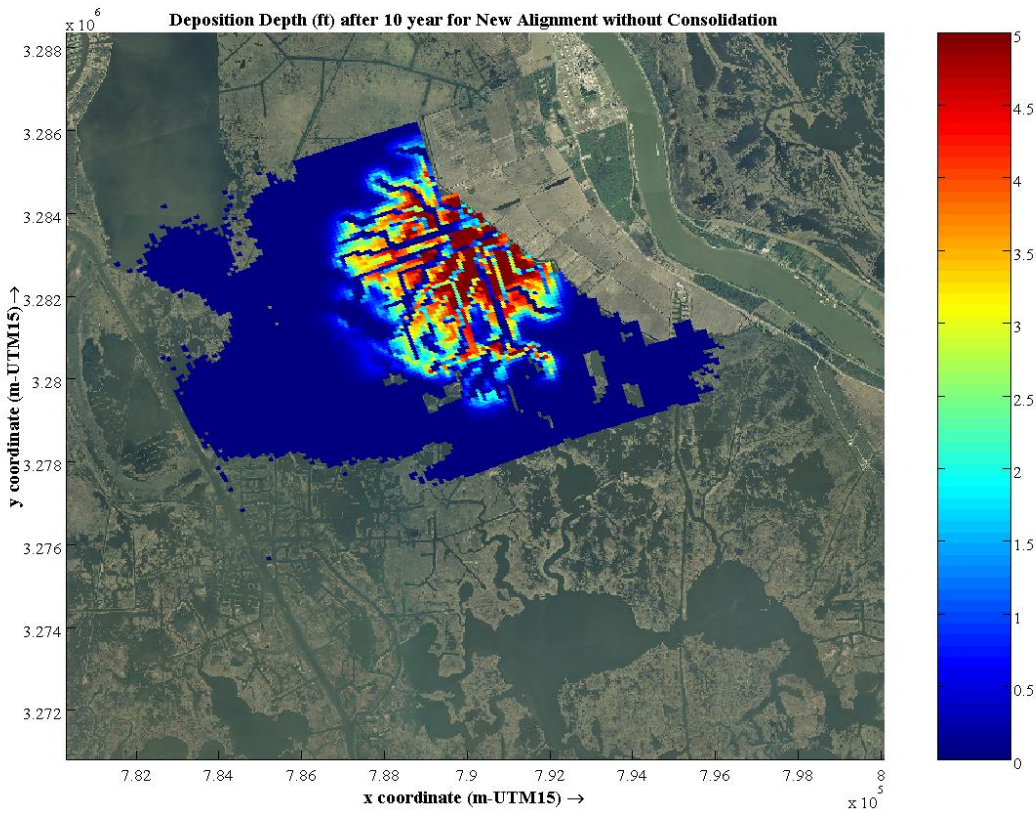


Figure 7-15. Deposition Depth (ft) after 10 years for New Alignment (45,000 cfs) without Consolidation

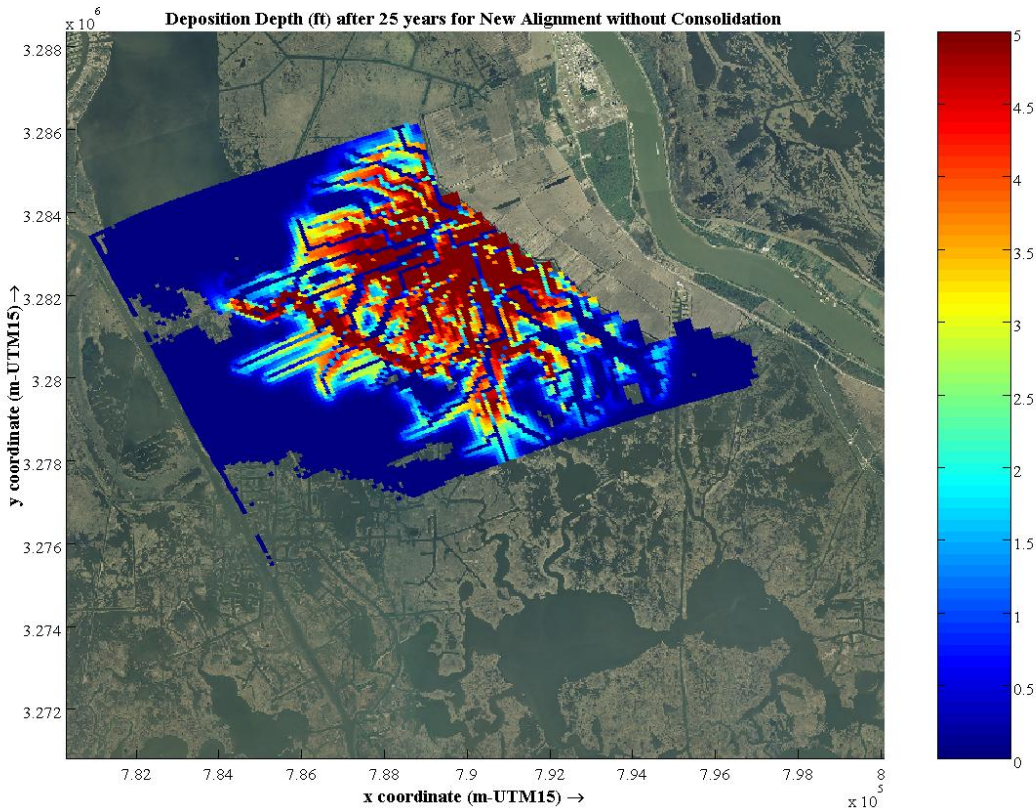


Figure 7-16. Deposition Depth (ft) after 25 years for New Alignment (45,000 cfs) without Consolidation

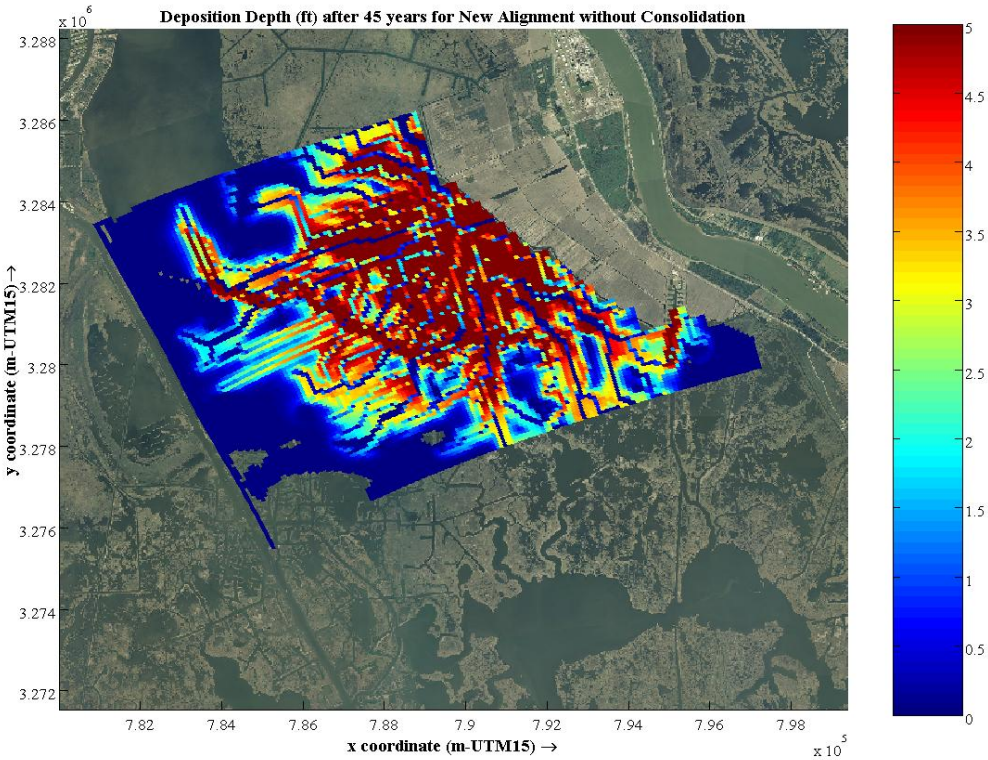


Figure 7-17. Deposition Depth (ft) after 45 years for New Alignment (45,000 cfs) without Consolidation

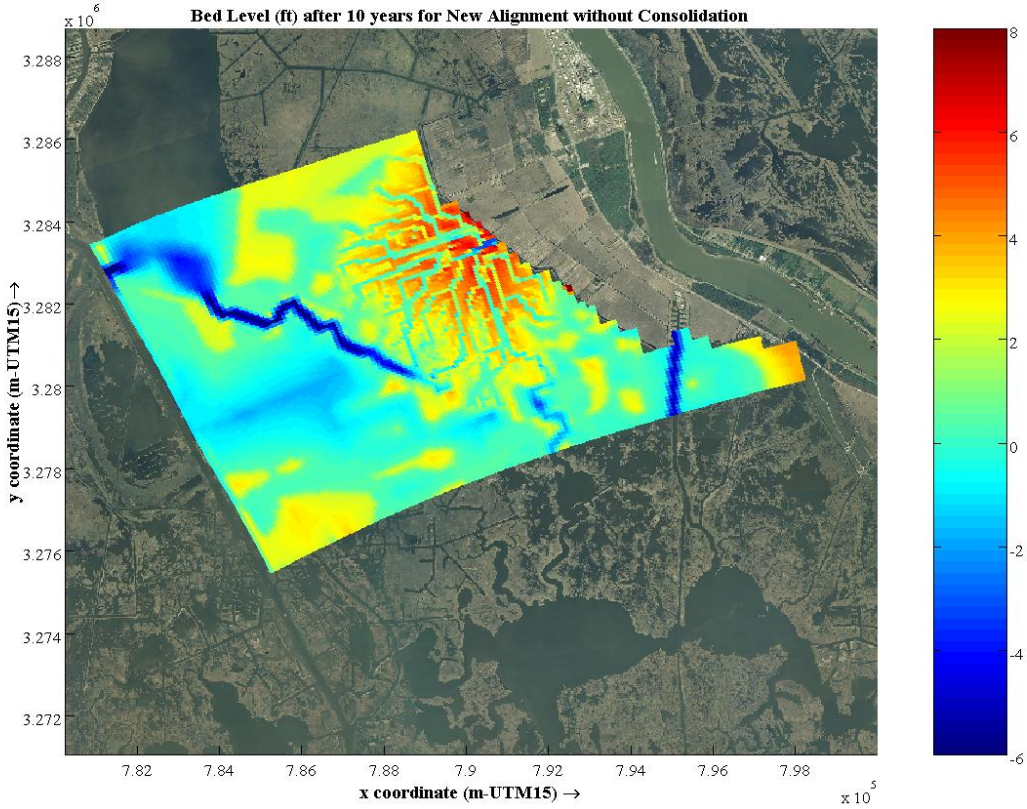


Figure 7-18. Bed Level (ft) after 10 years for New Alignment (45,000 cfs) without Consolidation

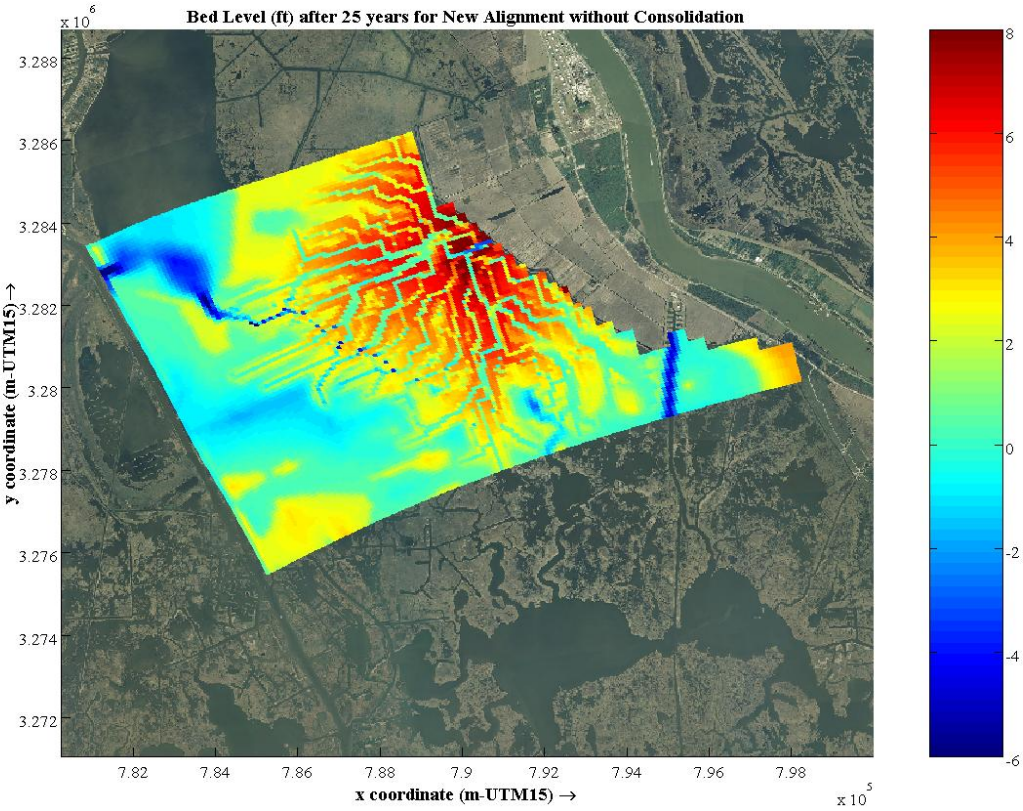


Figure 7-19. Bed Level (ft) after 25 years for New Alignment (45,000 cfs) without Consolidation

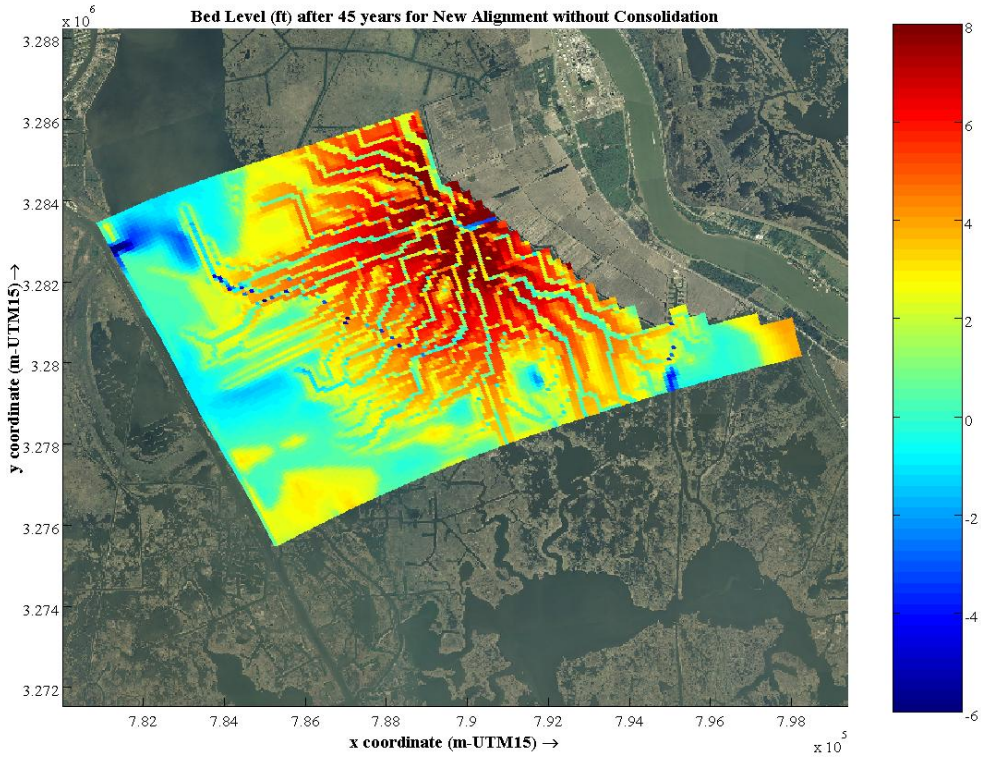


Figure 7-20. Bed Level (ft) after 45 years for New Alignment (45,000 cfs) without Consolidation

Table 7-8. Deposition Results after 10 years for New Alignment (45,000 cfs) without Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	4,431,936
0.5 – 1.0 ft	3,329,867
1.0 – 1.5 ft	2,743,389
1.5 – 2.0 ft	2,245,584
2.0 – 2.5 ft	1,770,123
2.5 – 3.0 ft	1,335,570
3.0 – 3.5 ft	970,826
3.5 – 4.0 ft	666,100
4.0 – 4.5 ft	426,276
4.5 – 5.0 ft	274,039
> 5.0ft	388,628
Total	18,582,337

Table 7-9. Deposition Results after 25 years for New Alignment (45,000 cfs) without Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	8,795,855
0.5 – 1.0 ft	7,179,050
1.0 – 1.5 ft	6,085,195
1.5 – 2.0 ft	5,132,945
2.0 – 2.5 ft	4,322,011
2.5 – 3.0 ft	3,565,413
3.0 – 3.5 ft	2,888,628
3.5 – 4.0 ft	2,323,548
4.0 – 4.5 ft	1,798,140
4.5 – 5.0 ft	1,365,358
> 5.0ft	3,040,195
Total	46,496,339

Table 7-10. Deposition Results after 45 years for New Alignment (45,000 cfs) without Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	12,220,330
0.5 – 1.0 ft	10,788,162
1.0 – 1.5 ft	9,799,366
1.5 – 2.0 ft	8,709,161
2.0 – 2.5 ft	7,533,426
2.5 – 3.0 ft	6,433,712
3.0 – 3.5 ft	5,288,048
3.5 – 4.0 ft	4,286,656
4.0 – 4.5 ft	3,445,633
4.5 – 5.0 ft	2,721,009
> 5.0ft	8,150,007
Total	79,375,509

7.7.2 With Consolidation

Next, the same two runs were performed but accounting for consolidation of the underlying soils. Again, it is noteworthy that these modeling results do not include the effects of subsidence, sea level rise or the adaptation of the marsh to such effects. Also, as previously noted, they do not account for the decrease in efficiency of the diversion channel over time.

Figure 7-21 to 7-23 present the modeling results for the total depth of material deposited after 10, 25 and 45 years, respectively for the 15,000 cfs USACE alignment alternative with consideration of underlying soil consolidation. Figure 7-24 to 7-26 present the modeling results for the resulting bed elevation after 10, 25 and 45 years, respectively. Table 7-11 to 7-13 present the deposition volume for each 0.5 ft deposition depth interval after 10, 25 and 45 years, respectively.

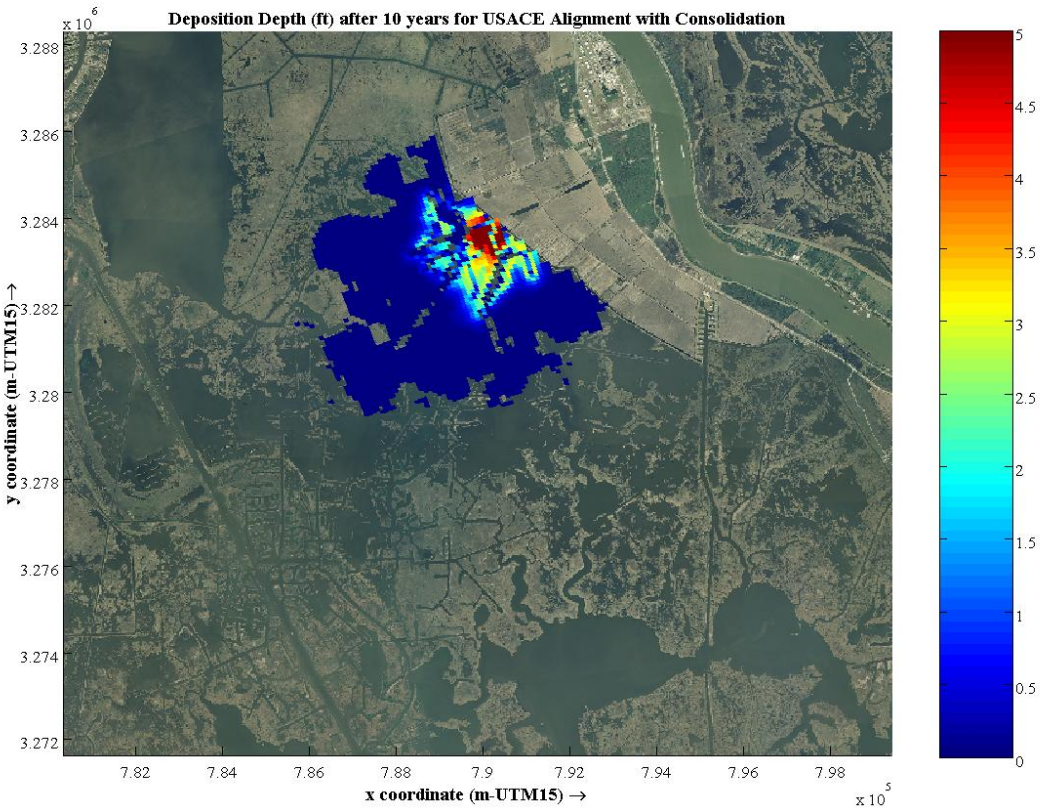


Figure 7-21. Deposition Depth (ft) after 10 years for USACE Alignment (15,000 cfs) with Consolidation

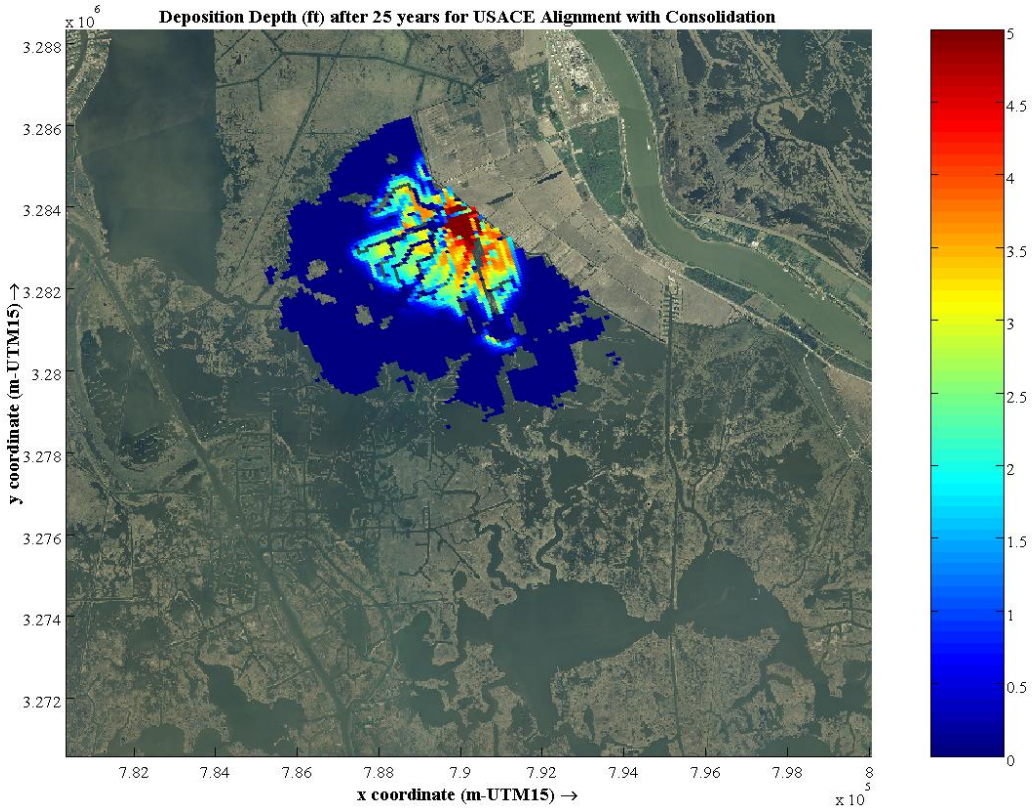


Figure 7-22. Deposition Depth (ft) after 25 years for USACE Alignment (15,000 cfs) with Consolidation

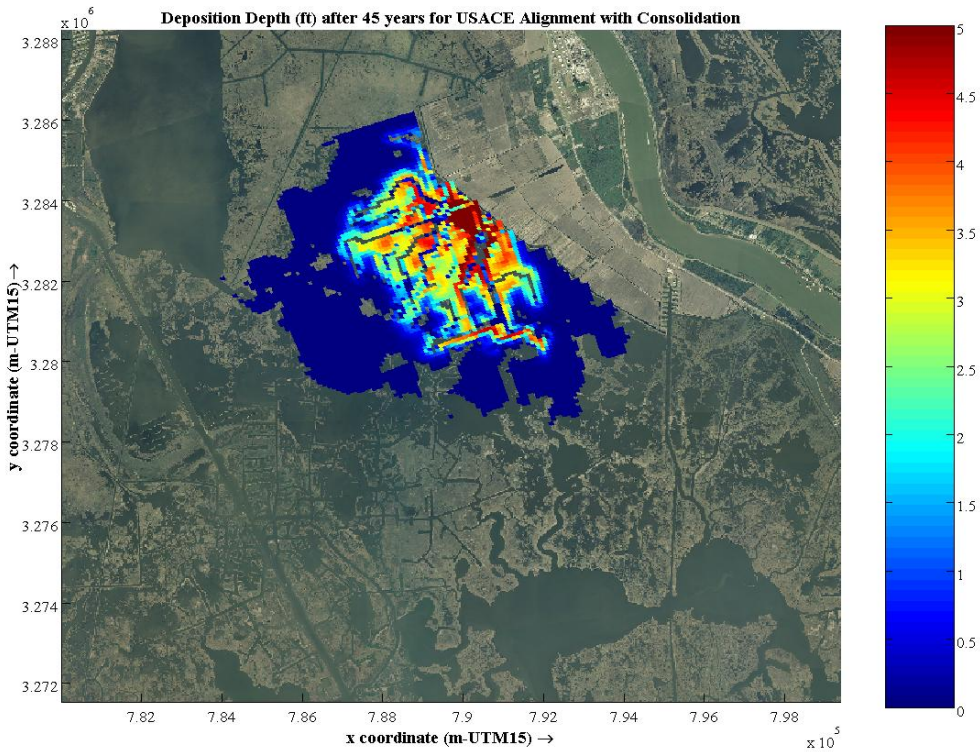


Figure 7-23. Deposition Depth (ft) after 45 years for USACE Alignment (15,000 cfs) with Consolidation

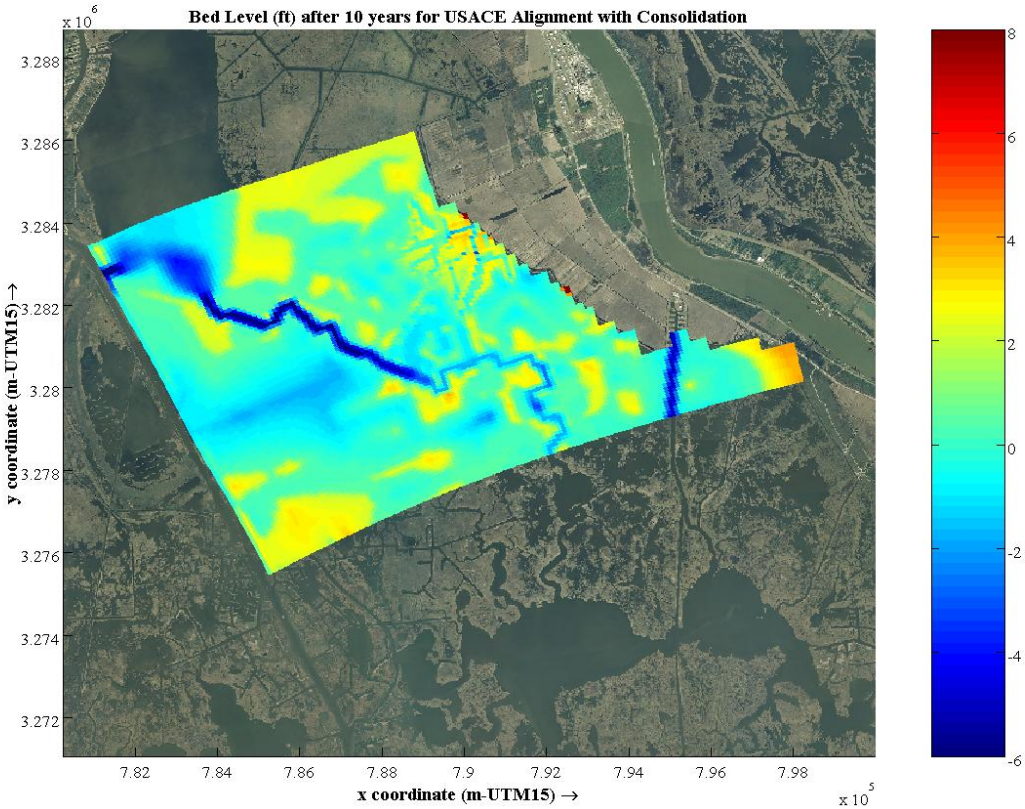


Figure 7-24. Bed Level (ft) after 10 years for USACE Alignment (15,000 cfs) with Consolidation

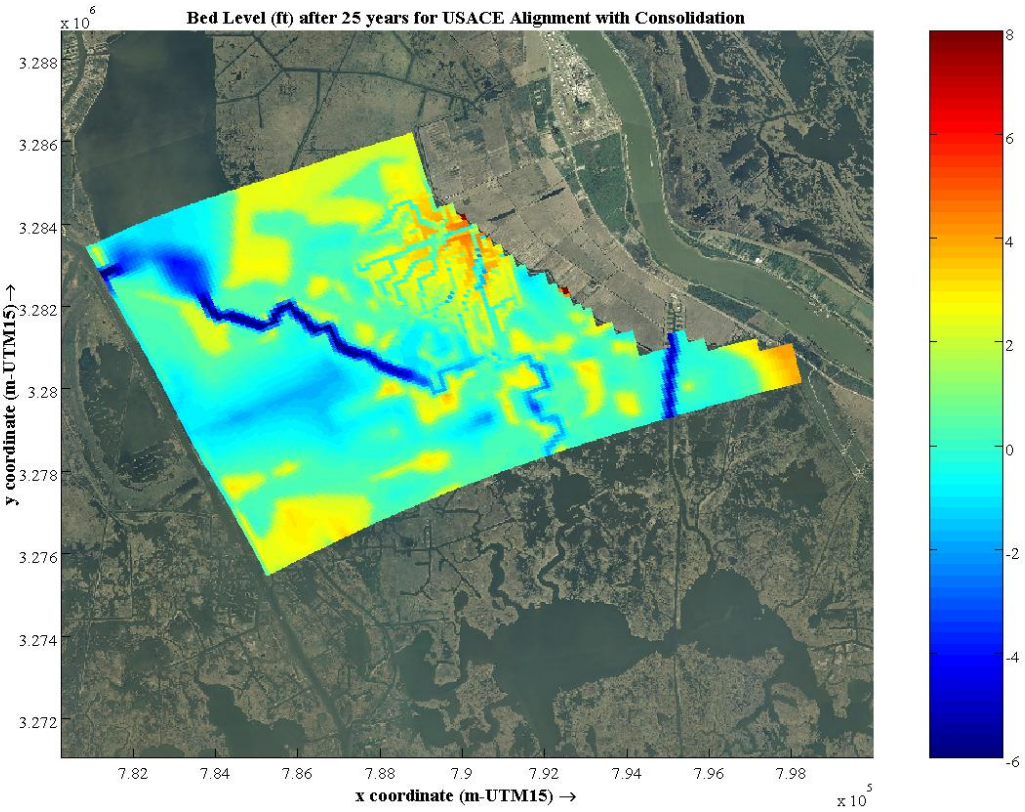


Figure 7-25. Bed Level (ft) after 25 years for USACE Alignment (15,000 cfs) with Consolidation

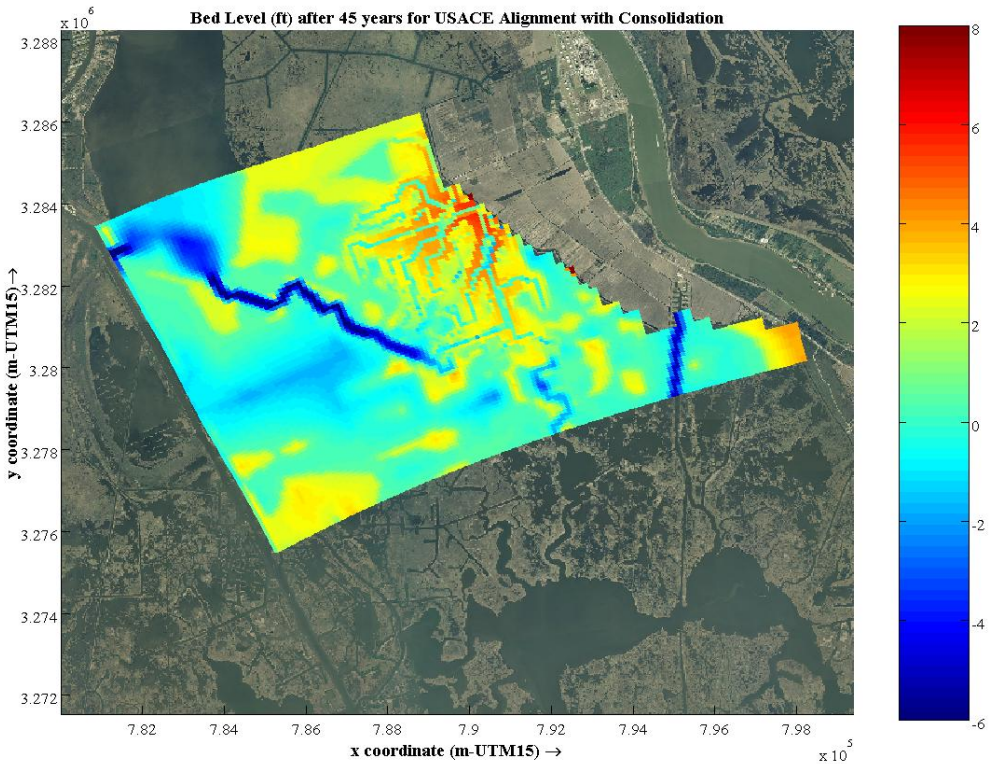


Figure 7-26. Bed Level (ft) after 45 years for USACE Alignment (15,000 cfs) with Consolidation

Table 7-11. Deposition Results after 10 years for USACE Alignment (15,000 cfs) with Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	1,066,671
0.5 – 1.0 ft	692,479
1.0 – 1.5 ft	509,981
1.5 – 2.0 ft	384,412
2.0 – 2.5 ft	251,917
2.5 – 3.0 ft	167,690
3.0 – 3.5 ft	109,319
3.5 – 4.0 ft	74,634
4.0 – 4.5 ft	51,190
4.5 – 5.0 ft	40,473
> 5.0ft	51,771
Total	3,400,537

Table 7-12. Deposition Results after 25 years for USACE Alignment (15,000 cfs) with Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	2,116,982
0.5 – 1.0 ft	1,625,885
1.0 – 1.5 ft	1,262,929
1.5 – 2.0 ft	944,012
2.0 – 2.5 ft	682,811
2.5 – 3.0 ft	451,930
3.0 – 3.5 ft	285,910
3.5 – 4.0 ft	151,910
4.0 – 4.5 ft	101,662
4.5 – 5.0 ft	72,585
> 5.0ft	160,736
Total	7,857,352

Table 7-13. Deposition Results after 45 years for USACE Alignment (15,000 cfs) with Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	3,327,969
0.5 – 1.0 ft	2,653,784
1.0 – 1.5 ft	2,163,814
1.5 – 2.0 ft	1,712,050
2.0 – 2.5 ft	1,318,932
2.5 – 3.0 ft	932,771
3.0 – 3.5 ft	582,574
3.5 – 4.0 ft	301,876
4.0 – 4.5 ft	168,732
4.5 – 5.0 ft	108,870
> 5.0ft	278,055
Total	13,549,428

Figure 7-27 to 7-29 present the modeling results for the total depth of material deposited after 10, 25 and 45 years, respectively for the 45,000 cfs new alignment alternative with consideration of underlying soil consolidation. Figure 7-30 to 7-32 present the modeling results for the resulting bed elevation after 10, 25 and 45 years, respectively. Table 7-14 to 7-16 present the deposition volume for each 0.5 ft deposition depth interval after 10, 25 and 45 years, respectively.

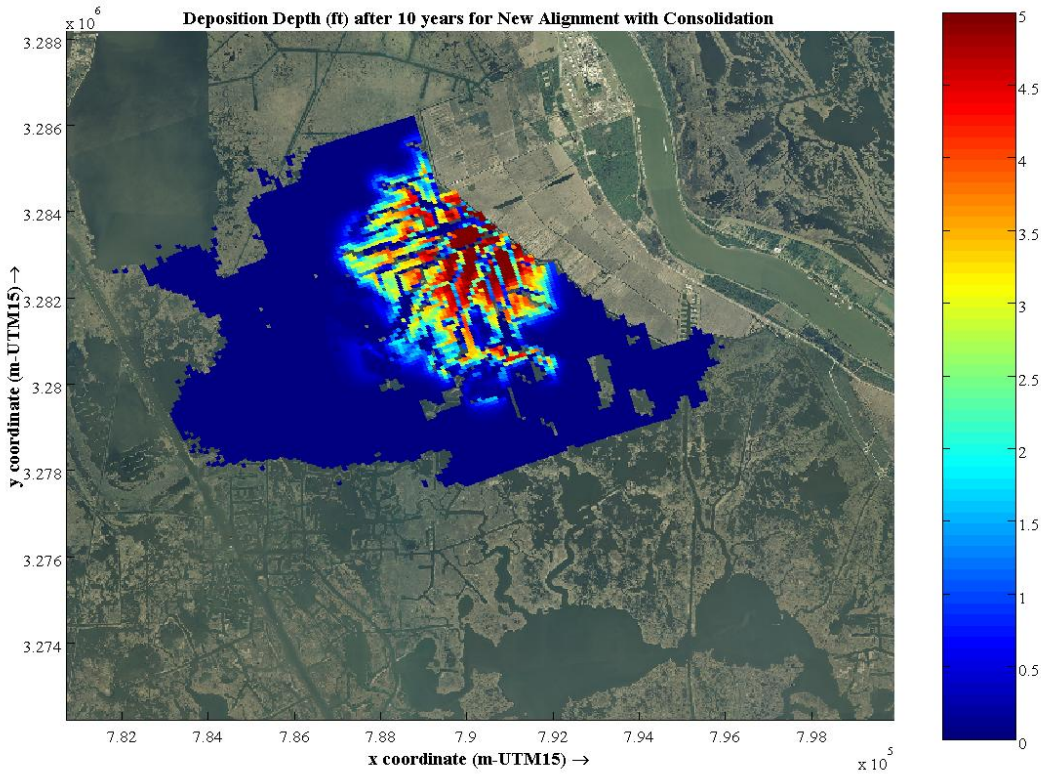


Figure 7-27. Deposition Depth (ft) after 10 years for New Alignment (45,000 cfs) with Consolidation

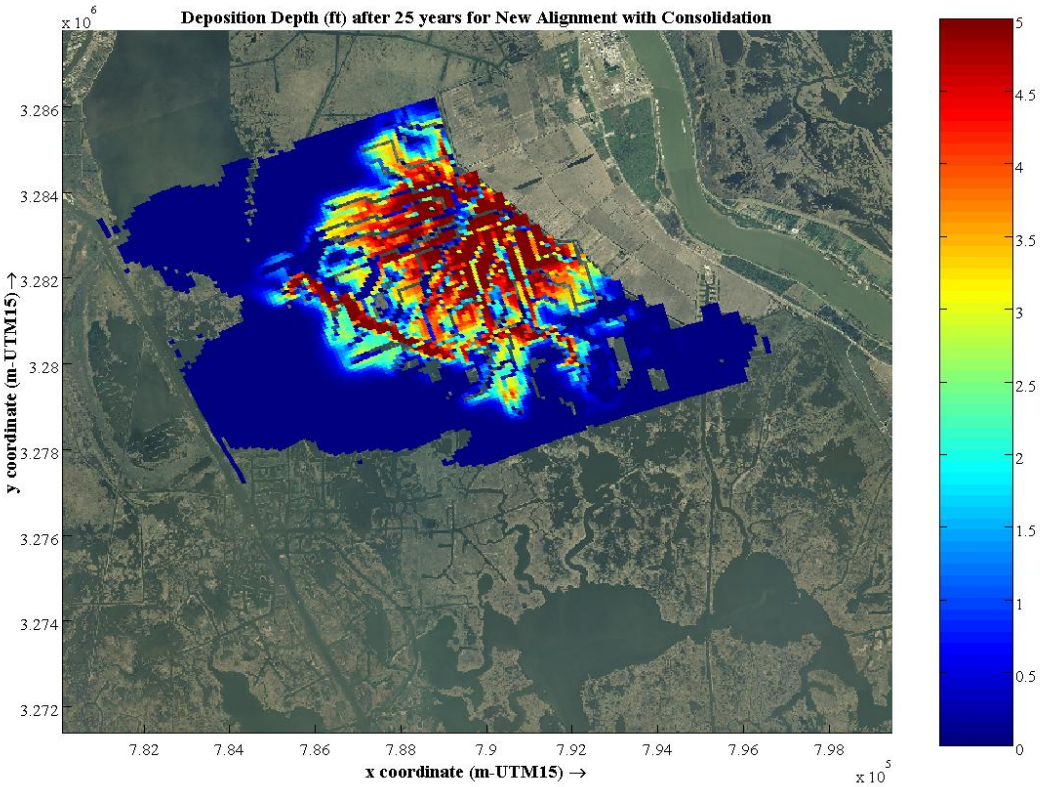


Figure 7-28. Deposition Depth (ft) after 25 years for New Alignment (45,000 cfs) with Consolidation

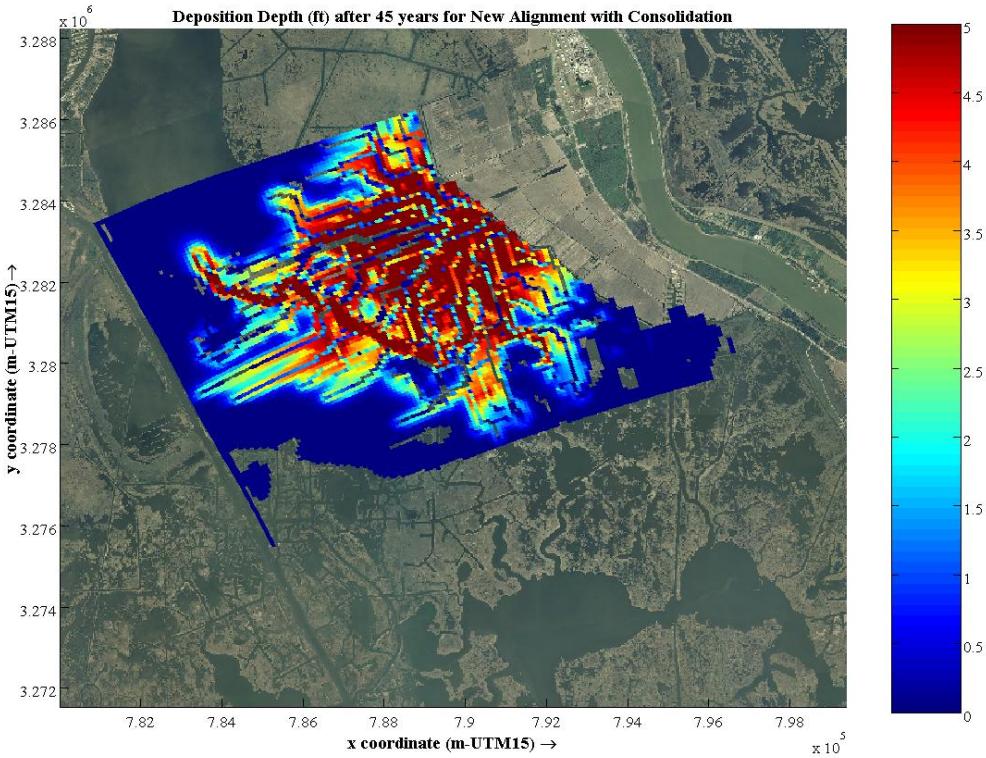


Figure 7-29. Deposition Depth (ft) after 45 years for New Alignment (45,000 cfs) with Consolidation

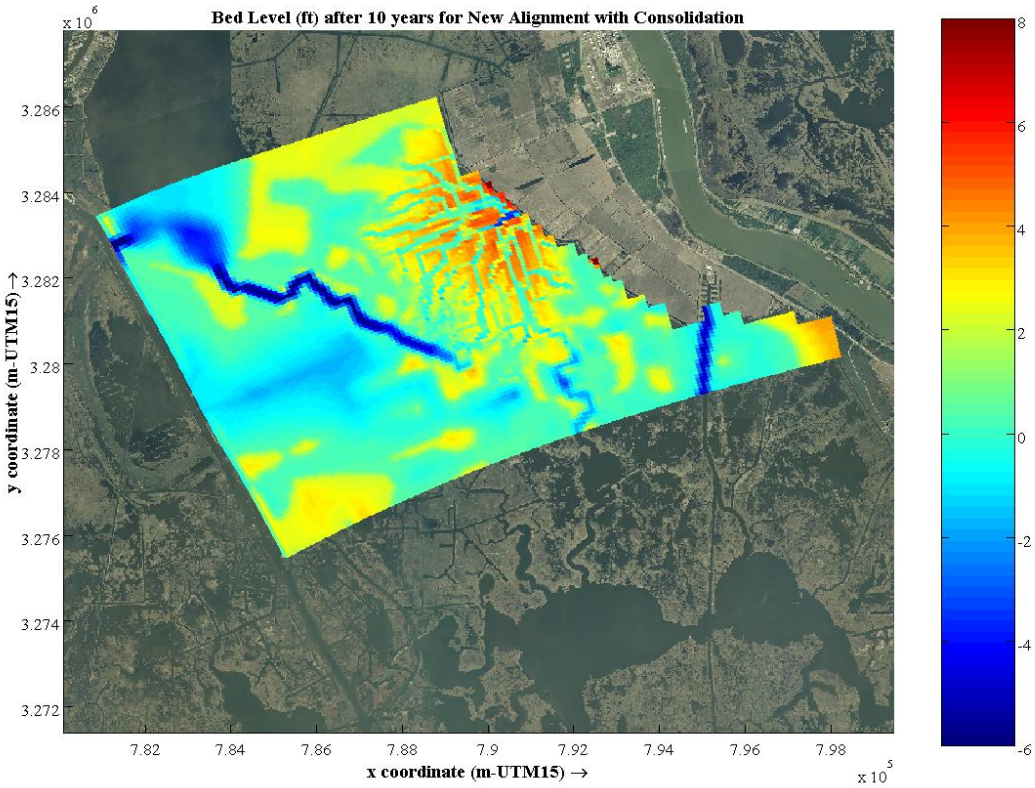


Figure 7-30. Bed Level (ft) after 10 years for New Alignment (45,000 cfs) with Consolidation

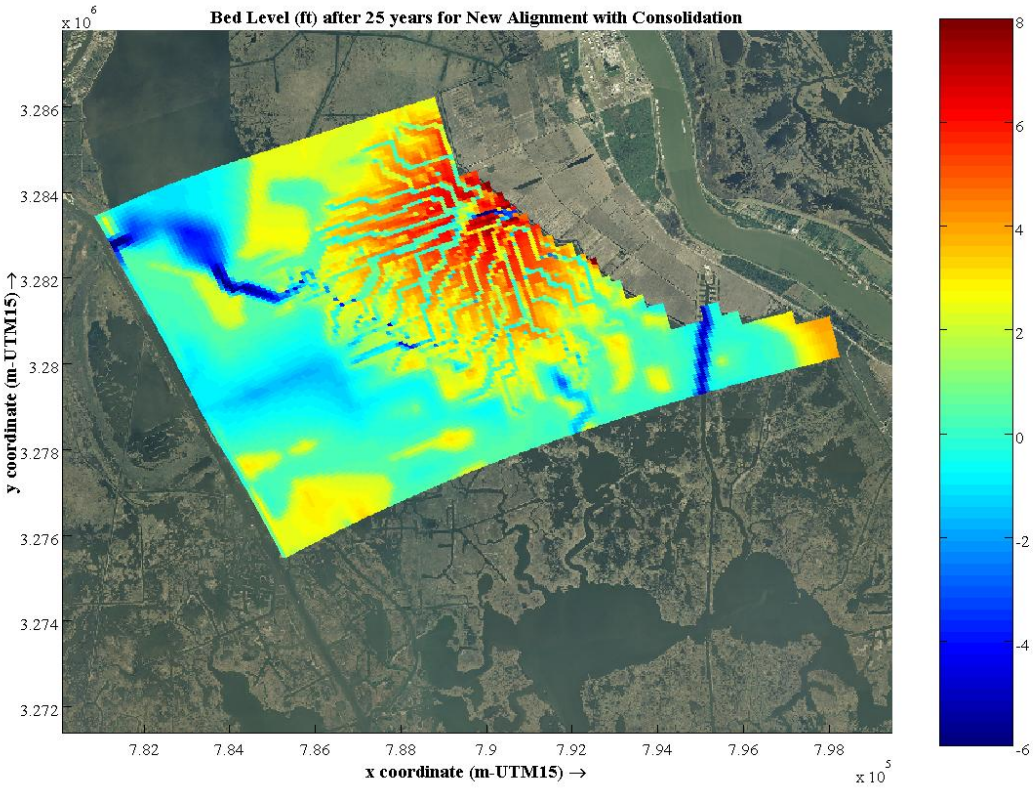


Figure 7-31. Bed Level (ft) after 25 years for New Alignment (45,000 cfs) with Consolidation

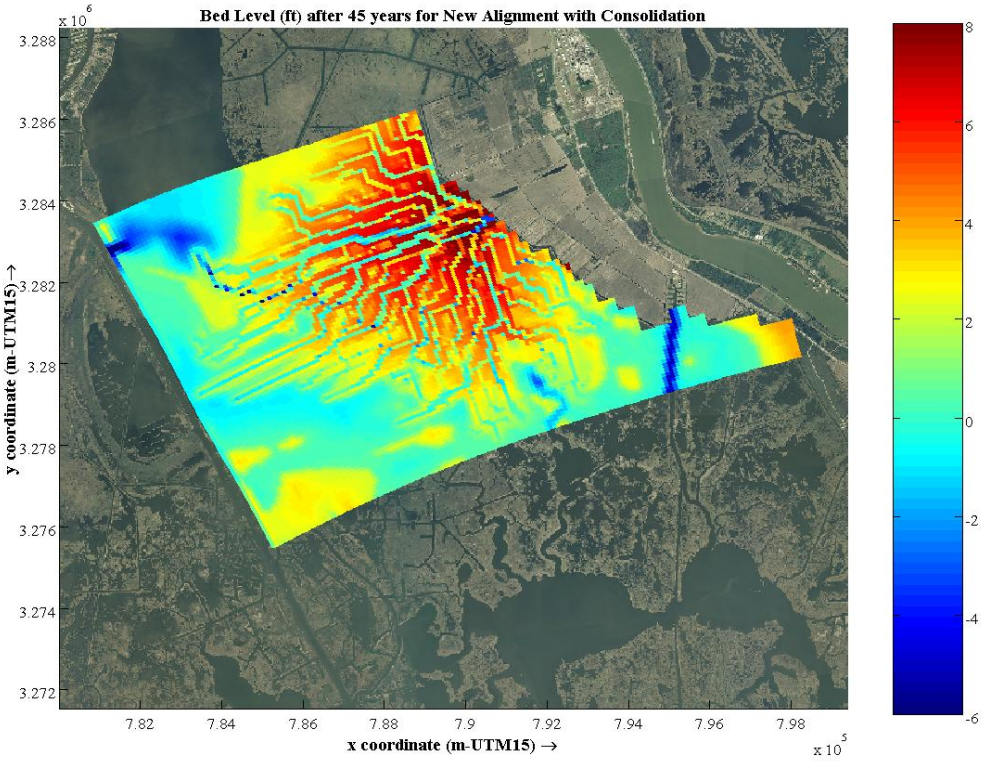


Figure 7-32. Bed Level (ft) after 45 years for New Alignment (45,000 cfs) with Consolidation

Table 7-14. Deposition Results after 10 years for New Alignment (45,000 cfs) with Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	3,657,197
0.5 – 1.0 ft	2,649,417
1.0 – 1.5 ft	2,127,551
1.5 – 2.0 ft	1,680,675
2.0 – 2.5 ft	1,248,275
2.5 – 3.0 ft	920,737
3.0 – 3.5 ft	614,417
3.5 – 4.0 ft	377,099
4.0 – 4.5 ft	234,166
4.5 – 5.0 ft	134,219
> 5.0ft	185,103
Total	13,828,856

Table 7-15. Deposition Results after 25 years for New Alignment (45,000 cfs) with Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	6,499,347
0.5 – 1.0 ft	5,238,749
1.0 – 1.5 ft	4,450,614
1.5 – 2.0 ft	3,703,207
2.0 – 2.5 ft	3,045,390
2.5 – 3.0 ft	2,424,538
3.0 – 3.5 ft	1,962,655
3.5 – 4.0 ft	1,547,820
4.0 – 4.5 ft	1,136,521
4.5 – 5.0 ft	773,719
> 5.0ft	1,227,927
Total	32,010,488

Table 7-16. Deposition Results after 45 years for New Alignment (45,000 cfs) with Consolidation

Deposition Depth	Deposition Volume (yd^3)
< 0.5 ft	9,845,054
0.5 – 1.0 ft	8,269,320
1.0 – 1.5 ft	7,068,785
1.5 – 2.0 ft	6,051,881
2.0 – 2.5 ft	5,166,077
2.5 – 3.0 ft	4,239,346
3.0 – 3.5 ft	3,414,589
3.5 – 4.0 ft	2,743,833
4.0 – 4.5 ft	2,202,083
4.5 – 5.0 ft	1,663,561
> 5.0ft	3,877,226
Total	54,541,756

7.8 Conclusions

Table 7-17 presents a summary of the total depositional volumes after 45 years for the two alternatives for both the without and with consolidation scenarios.

Table 7-17: Summary of Deposition Results after 45 years

Alignment	Discharge	Consolidation	Deposition
USACE	15,000 cfs	No	~20.5 million cy
USACE	15,000 cfs	Yes	~13.5 million cy
New	45,000 cfs	No	~79.4 million cy
New	45,000 cfs	Yes	~54.5 million cy

It indicates that the new alignment is much more efficient than the original one. After accounting for consolidation, while the discharge is increased by 3 times, the volume deposited increases by over 4 times.

Additionally, the graphical results indicate that, over 45 years, a classic "delta" will form and the bed level will be increased significantly within the area up to about 3 miles from where the discharge channel enters the basin. However, the effects further away quickly diminish with little impact observed at the Barataria Waterway.

This model, though, could not be "calibrated" to a similar project in this area, so the spatial results are mainly qualitative, but the total volumes are accurate as they are based on the results of the riverine modeling. Furthermore, it is evident that significant deposition of the coarser materials will occur at the channel / basin interface and higher resolution modeling (both numerical and physical) of this area will be required to determine how best to manage this area to ensure the operational efficiency of the diversion.