APPENDIX D
Cast-in-Place vs. Float-in-Place Letter Report

IHNC LOCK
INNER HARBOR NAVIGATION CANAL
NEW ORLEANS, LOUISIANA

NEW 1200' LOCK
LETTER REPORT OF FLOAT-IN-PLACE VS. CAST-IN-PLACE COMPARISON

July 2007

100% Draft Submission

100% Submission – July 2007

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Executive Summary

The following report is included as an appendix to the Supplemental Environmental Impact Study required for the IHNC Lock project.

The IHNC Lock is located in the southeastern portion of Louisiana within the city limits of New Orleans, in the IHNC, formerly known as the Industrial Canal. This channel provides navigation access across the Mississippi River for significant traffic using the Gulf Intracoastal Water Way. A highly urbanized area surrounds the lock on both sides of the canal. The canal cannot be shut down for long periods of time without major impact to the navigation industry. Also the project must be built in a highly congested urban area with ZERO residential relocation.

In February 2003, the URS design team was given Task Order No. 1, Contract DACW29-02-D-0008. The task order was to provide engineering services for the design of the IHNC replacement lock to the 50% level of design completion. The work consists of advancing work that was accomplished as presented in the evaluation report and its appendices. The evaluation report is to be used as the basis for refining and improving the feasibility design and preparing the final design for construction using the float-in methodology of construction. The recommended ***Float-In-Place Plan*** is for a deep-draft lock, 110 feet wide by 1,200 feet long by 36 feet of draft. The lock construction would use a pre-fabricated, float-in method. Five lock modules of concrete and steel would be built at a remote location and floated to the North-of-Claiborne-Avenue site. Movement of the modules will be facilitated by the 300-foot horizontal clearance of the Port of New Orleans and a U.S. Coast Guard bridge at Florida Avenue completed in 2005. The Float-In-Place modules will be founded on 48” diameter steel pipe piles. A bypass channel will be built to allow navigation to continue during construction. The estimated cost for the new Float-in-Place lock is $846 M at a 1 October 2006 Price Level.

A recent New Orleans District project revealed that uncertainty and risks associated with the float-in methodology of construction could greatly increase a contractor’s bid. With the above mentioned risk associated with float-in construction, it was decided in late 2004 that a cast-in place option would be investigated for the 110-foot by 1200-foot ship lock. The alternative plan was developed to build the new lock using a more traditional ***Cast-In-Place Plan***. Seven lock monoliths founded on 24” Square PPC piles, will be built in the dry within a Cellular Sheet pile Cofferdam. A bypass channel will be built to allow navigation to continue during construction. The bypass channel shall be shifted to the east due to the cofferdam configuration. The estimated cost for the new Cast-in-Place lock is $792 M at a 1 October 2006 Price Level.

This report summarizes the studies to date to determine the best constructible procedure (Float-in-Place vs. Cast-in-Place) for the new IHNC Lock. The report provides background and establishes selection criteria for two construction methodologies. Also included is an evaluation of alternatives, risks and costs. The criteria used for this comparison included impacts to the community (noise, vehicular and construction traffic), cost growth potential, constructability, biddability, operability, maintainability, construction cost, aesthetics, impacts to navigation, design, funding constraints, and life cycle cost.

The end result showed that the ???-In-Place was the preferred method of construction. The Working Cost Estimate is $????????.

Pertinent Data

Table 1 Dimensions of the new IHNC Lock

Description	Data	Reference
Normal Size	110 ft X 1270 ft	
Actual Size (pintle to pintle)	110 ft X 1287’-8” ft	Measurement from CIP drawings “Plan and Wall Profile”
River Side Pool Project Depth (Normal)	50 ft	Depth in Chamber; CIP drawings
Lake Side Pool Project Depth (Normal)	41 ft	Depth in Chamber; CIP drawings
Normal design lift	9 ft	DDR Appendix A
Maximum design lift	22 ft	DDR Appendix A
Type of service gates	Sector Gate (top elevation of RS & LS Gates will be EL 23)	DDR Appendix A, Section 6-1
Type of emergency closure, upstream	Bulkheads with wheels not to Exceed 80 tons	DDR
Type of maintenance closure, downstream	Bulkheads not to Exceed 80 tons	DDR Section 13 Maintenance Bulkhead
Type of filling and emptying system	In-walls longitudinal culvert systems, horizontal porting	
Valves	Vertical Slide	New Orleans CIP drawings Culvert roller gate drawings S-604
Top of Lock Walls	EL 23	DDR Appendix A Para.1-3
Maximum RS pool	EL 18	CIP Dgn S-401
Design RS Pool (normal)	EL 10	DDR Appendix A Para 3
RS upper pool (Minimum Stage of Record)	EL -1.6	CIP Dgn S-401
RS approach channel depth (normal)	47 ft	FIP Dgn. 2-107 section A-A shows River bed is at El. -37
RS cofferdam bottom elevation	EL -125	Cofferdam 100% submittal DWG

Top of RS maintenance bulkhead sill	EL -40	CIP drawing S-401
Protection elevation of maintenance bulkhead	El 20	CIP drawings S-601 & S-602
Top of RS sector gate sill	EL -40	CIP Dgns
Sector Gate Weighs	623 KIP	Binder #6, page 6
Maximum LS pool	EL 13	CIP Dgn S-402
Design LS pool (normal)	EL 1	DDR Appendix A Para 3
LS Lower pool (Minimum)	EL -2	CIP Dgn S-402
Top of LS maintenance bulkhead sill	EL -40	CIP Dgns
Top of LS sector gate sill	EL -40	CIP Dgns
LS approach Channel depth (normal)	38 ft	FIP Dgn. 2-108 section E-E Shows river bed depth at elevation -37
LS cofferdam	El 5	Cofferdam 100% submittal DWG
Chamber Floor	EL -40	CIP Dgns
Top of LS bulkhead Sill	EL -40	CIP Dgns
Upper Approach Wall	Floating	CIP S-102
Length of Upper Approach Wall, feet	1260’ Floating Guidewall	CIP S-102
Top of Upper Approach Wall, feet	El 23	FIP 2-107
Lower Approach Wall	Timber Crib	CIP S-103
Length of Lower Approach Wall, feet	800’ Timber Guide	CIP S-103
Top of Lower Approach Wall, feet	EL 13	CADD Measurement from FIP 2-108
Culvert Size	18.25’H X 15’W	CIP DGN; DDR Append A Para 1-3
Cofferdam Top Elevation	EL 5	Cofferdam 100% submittal DWG
Crane 175 tons at an operating radius of 85'		Section 5.16

Table 2 Dimensions of the Existing IHNC Lock

Size of Chamber		
Normal Size	75 ft x 640 ft	EM 1110-2-1604 May 06
Normal design lift	9 ft	EM 1110-2-1604 May 06
Depth	31.5 ft	Project Fact Sheet on Web

Basic Design Data

The following basic design data were used in the preparation of the design computations and development of the structural evaluation of the lock features.

Unit Weights

Unit weights of materials used in preparation of the structural calculations were as follows:

Water	62.4 pcf
Concrete	145.0 pcf
Select Sand	120.0 Ko=0.50
Semi-Compacted Sand	110.0 Ko=0.80
Silt	117.0
Stone	132.0
Concrete	150.0
Steel	490.0

Load Cases:

Gate Bay Monolith Design Load Cases:
The following load cases were investigated for the design of the sector gate bay monoliths:
Hurricane, Maximum Head Stillwater
R/S El. = 0.0   L/S El. = 13.0
Hurricane, Maximum Head Stillwater + Freeboard
R/S El. = 0.0   L/S El. = 14.0 (1.33% Overstress)
Operation, Maximum Direct Head
R/S El. = 18.0 L/S El. = 0.0
Operation, Maximum Direct Head + Freeboard
R/S El. = 23.0 L/S El. = -2.0 (1.33% Overstress)
Normal Operation
R/S El. = 10.0 L/S El. = 1.0
Operation Reverse Head Navigation Limit
R/S El. = 0.0   L/S El. = 5.0
Usual Maintenance Dewatering
R/S El. = 10.0 L/S El. = 5.0 (1.167% Overstress)
Unusual Maintenance Dewatering
R/S El. = 18.0 L/S El. = 0.0 (1.33% Overstress)
Construction (1.33% Overstress)

Chamber Monolith Design Load Cases:
The following load cases were investigated for the design of the chamber monoliths:
Hurricane, Maximum Head Stillwater

R/S El. = 0.0   L/S El. = 13.0
Hurricane, Maximum Head Stillwater + Freeboard
R/S El. = 0.0   L/S El. = 14.0 (1.33% Overstress)
Operation, Maximum Direct Head
R/S El. = 18.0 L/S El. = 0.0
Operation, Maximum Direct Head + Freeboard
R/S El. = 23.0 L/S El. = -2.0 (1.33% Overstress)
Normal Operation
R/S El. = 10.0 L/S El. = 1.0
Operation Reverse Head Navigation Limit
R/S El. = 0.0   L/S El. = 5.0
Usual Maintenance Dewatering
R/S El. = 10.0 L/S El. = 5.0 (1.167% Overstress)
Construction (1.33% Overstress)

1. Introduction

The IHNC Lock is located in the southeastern portion of Louisiana within the city limits of New Orleans, in the IHNC, formerly known as the Industrial Canal. It connects the Mississippi River, the Gulf Intracoastal Waterway (GIWW), the Mississippi River-Gulf Outlet (MRGO), the Industrial Canal (also known as the Inner Harbor Navigation Canal), and Lake Pontchartrain.

The current lock, placed in service in 1921, is too small to accommodate the existing traffic: 640 feet long, 75 feet wide and 31.5 feet deep. The existing Industrial Canal Lock is a vital link in the nation's inland waterway navigation system. The average delay to navigation is 11 hours but can be as much as 24 to 36 hours on many occasions. A highly urbanized area surrounds the lock on both sides of the canal. The canal cannot be shut down for long periods of time without major impact to the navigation industry. Also the project must be built in a highly congested urban area with ZERO residential relocation.

The federal government (Corps of Engineers and Inland Waterways Trust Fund) is responsible for the inland (shallow-draft) navigation portion of the project. The Port of New Orleans and the federal government are sharing the costs of the deep-draft navigation portion, as described below. The Industrial Canal Lock Replacement Project is authorized by the River and Harbor Act of 1956 (PL 84-455) and the Water Resources Development Acts of 1986 (PL 99-662), which reauthorized the project and established cost-sharing requirements, and 1996 (PL 104-303), which authorized the Community Impact Mitigation Plan.

1.1. Background

The recommended plan is to build a deep-draft lock, 110 feet wide by 1,200 feet long by 36 feet of draft. The lock construction would use a pre-fabricated Float-in-Place (FIP) design and construction technique; five lock modules of concrete and steel would be built at a remote location and floated to the North-of-Claiborne-Avenue site. Movement of the floating modules will be facilitated by the 300-foot horizontal clearance of the Port of New Orleans bridge at Florida Avenue completed in 2005. A bypass channel will be built to allow navigation to continue during construction.

The recent New Orleans District- Harvey Canal Hurricane Floodgate project revealed that uncertainty and risks associated with the float-in methodology of construction could greatly increase a contractor's bid. Project information can be located at the following website: http://www.mvn.usace.army.mil/harvey_photopage.htm
With the above mentioned risk associated with float-in construction, it was decided in late 2004 that a cast-in place option would be investigated for the 110-foot by 1200-foot ship lock. The cast in place study would be completed concurrently with the completion of the URS design team's Task Order No. 1. After completion of the study, a decision will be made on a number of design and construction factors including cost.

An alternative plan was developed to build the new lock using a more traditional Cast-In-Place Plan (CIP). This report summarizes the studies to determine the best constructability procedure (Float-in-Place vs. Cast-in-Place) for the new IHNC Lock.

1.2 Work Plan

In February 2006, the New Orleans District tasked a CELRD team consisting of Pittsburgh and Huntington District team members to review the float-in-place method (FIP) proposed in the recommended plan, and compare it to a cast-in-place method (CIP) which the New Orleans District had previously developed to some detail. The ultimate goal of this task is to produce a decision document that compares the two alternatives in the following areas:

1) Design and Constructability
2) Impacts to Navigation Industry and Local Communities
3) Construction Schedules and Contracting Flexibility
4) Risks
5) Cost Differential ($ Δ)

This document outlines the work effort required to further develop these two alternatives to a commensurate level of detail so that the differences in cost, schedule, impacts, and risk, can be accurately compared.

LRD was tasked with reviewing the cost estimates for the float-in-place (FIP) and cast-in-place (CIP) alternatives, and developing a comparative analysis report for each option. The ultimate goal was to provide a comparative cost estimate between the FIP vs. CIP construction options, specifically investigating those features that contributed to the cost differences between the two plans. The items that were similar for each plan were not designed, investigated or estimated in further detail. The cost estimate does not represent the total project costs for the IHNC Lock. LRD tasked URS to provide additional quantities for the Float-in-Place design so that a detailed "MCACES" level estimate could be completed for the significant items of cost and a comparison could be made with the cast-in-place option.

LRD completed the Cast-in-Place work plan to achieve a feasibility level design within a 20% contingency then compared this design and quantities to the Float-in-Place design and summarized the differences and risks with regard to design and construction. Based on the review of the documentation for both alternatives, a Gap Analysis was performed on both the Float-In Preliminary Design and the Cast-In Place Concept Study to identify project features or critical components of each design that were either missing from the existing reports, or not developed to a sufficient level of detail that will allow for accurate comparisons.

2. References

The following are references used to bring the new lock analysis to the current design level. This list is not intended to be all inclusive.

2.1. Technical Manuals

The structural components shall be designed according to the applicable portions of the Corps of Engineers (COE) Manuals for engineering and design and other reference material.

a. COE Publications

(1) EM 1110-2-2000, Standard Practice for Concrete (Sep 85).
(2) EM 1110-2-2102, Waterstops and Other Joint Materials (May 93).

(3) EM 1110-2-2104, Strength Design for Reinforced - Concrete Hydraulic Structures (June 92).
(4) EM 1110-2-2105, Design of Hydraulic Steel Structures (May 93).
(5) EM 1110-2-2502, Retaining and Floodwalls (Sep 89).
(6) EM 1110-2-2602, Planning and Design of Navigation Locks (Sep 95).
(7) EM 1110-2-2703, Lock Gates and Operating Equipment (Jun 84).
(8) EM 1110-2-2906, Design of Pile Foundations (Jan 91).
(9) EM 1110-2-8152, Planning and Design of Temporary Cofferdams and Braced Excavations (Aug 94).
(10) EM 1110-2-6050, Response Spectra and Seismic Analysis for Hydraulic Structures, 30 June 1999.
(11) ER 1110-2-1806, Earthquake Design and Evaluation for Civil Works Projects (Jul 95).
(12) ETL 1110-2-256, Sliding Stability for Concrete Structures (Jun 81).
(13) ETL 1110-2-307, Flotation Stability Criteria for Concrete Hydraulic Structures (Aug 87).
(14) ETL 1110-2-338, Barge Impact Analysis (April 93).
(15) ETL 1110-2-355, Structural Analysis and Design of U-Frame Lock Monoliths (Dec 93).
(16) ETL 1110-2-562, Navigation Lock Guard Walls, 30 July 2004
(17) ER 1110-2-1806, Earthquake Design and Evaluation for Civil Works Projects, 30 Sep 1995.
(18) ERDC/CHL CHETN-IX-8, General Guard Wall Design Considerations for Tow Entry and Exit, June 2002
(19) TR 02-33, Positioning for Float-in and Lift-in Construction in Inland Waterways, December 2002.
(20) TR 00-2, Assessment of Heavy-Lift Equipment for In-the-Wet Construction of Navigation Structures, November, 2000.
(21) TR 01-24, Underwater Geotechnical Foundations, December 2001.
(22) TR 03-14, Proposed Design Criteria on Thin-Wall Precast Panels for Hydraulic Concrete Structures, August 2003
(23) INP-SL-1, Assessment of Underwater Concrete Technologies for In-the-Wet Construction of Navigation Structures, September 1999
(24) SL-80-4, Strength Report of Reinforced Concrete Hydraulic Structures, Report 3 – T Wall Design (Jan 82) B-3

b. Technical Publications

(1) American Concrete Institute, Building Code Requirements for Reinforced Concrete, (ACI 318R-89).
(2) American Concrete Institute, Guide for the Design and Construction of Fixed Offshore Structures, (ACI 357R-84).
(3) American Concrete Institute, State-of-the-Art Report on Barge-Like Concrete Structures, (ACI 357.2R-88)
(4) American Institute of Steel Construction (AISC), Manual of Steel Construction, 9th Edition, 1989.
(5) American Welding Society, Structural Welding Code, Steel, (AWS-D 1.1-88).

(6) Concrete Reinforcing Steel Institute, CRSI Handbook, (1984).
(7) American Petroleum Institute, Planning, Designing and Constructing Offshore Platforms - Load and Resistance Factor Design, (API RP-2A), 1993.
(8) Precast/Prestressed Concrete Institute, PCI Design Handbook, 4th Edition (1992)
(9) Post-Tensioning Institute, Post Tensioning Manual, 5th Edition, (1990).
(10) American Association of State Highway and Transportation Officials, Standard Specifications for Highway Bridges, 14th Edition (1992).

c. Computer Programs

(1) "Pile Group Analysis (CPGA)", WES Program No. X0080.
(2) "Pile Group Graphics Display (CPGG)", WES Program No. X0081.
(3) "Two Dimensional Analysis of U-Frame and W-Frame Structures (CWFRAM)", WES Program No. X0091.
(4) "C-Frame", WES Program No. X0030.
(5) "CWALSHT", WES Program No. X0031.
(6) "GT STRUDL", Georgia Institute of Technology.
(7) "CGSI", WES Program No. X0061.
(8) "CGFRAG", Wes Program No. X8008.

2.2. Previous IHNC Reports

Mississippi River - Gulf Outlet, New Lock and Connecting Channels Evaluation Report, March1998, http://www.mvn.usace.army.mil/prj/ihnc/EvaluationReport/ihnc_eval.htm

Design Documentation Report No. 3; IHNC Lock Replacement Project; Lock Foundation Report, May 2002

Inner Harbor Canal Lock Replacement Project; East Bank Soil Mixing Test Section Report (July 2004)

Inner Harbor Canal Lock Replacement; Design Documentation Report; Phase I Design, 100% Submittal (August 2006), URS Group, Inc Contract DACW29-02-D-0008; Task Order No. 1.

Inner Harbor Canal Lock Replacement; Cast In Place Cofferdam, Feasibility Level Design, 100% Submittal (October 2006), URS Group, Inc Contract DACW29-02-D-0008; Task Order No. 2.

Inner Harbor Canal Lock Replacement; Cost and Schedule Analysis. Comparison of Cast-In-Place and Float-In-Place construction alternatives. 100% Submittal (January 2007), Project Time & Cost, Inc Contract DACW01237-05-D-0020,

Task Order No. 7.

3. EIS

3.1. Impacts to Community EIS Mitigation Plan

This Inner Harbor Canal Lock Replacement Project was authorized under the following Legislation, which authorized navigational construction of the Mississippi River Gulf Outlet, navigation improvements and construction of the Inner Harbor Navigation Canal and Lock, and authorization to mitigate direct and indirect social and cultural impacts related to these construction efforts.

84th Congress, Public Law 455, Chapter 112, 29 March (1956)
94th Congress, Public Law 587, Section 186, WRDA (1976)
99th Congress, Public Law 622, Section 903(a), WRDA (1986)
104th Congress, Public Law 303, Section 326, WRDA (1996)
U.S. House of Representatives, Report No. 101-536, Committee on Appropriations, Energy and Water Appropriations Bill (1990)

Under Report No. 101-536, the Corps was specifically required to give maximum consideration to lock replacement alternatives which minimize residential and business disruptions while meeting the goals to improve the water born transportation network within the region.

3.2. Revisions to Environmental Impact Statement

In March 1998, the New Orleans District issued a final Environmental Impact Statement (EIS) that was prepared in conjunction with the Inner Harbor Navigation Canal (IHNC) Lock Replacement Feasibility Study. This study justified the need for replacement of the antiquated lock facility, and identified several alternative plans and lock sizes. In December 1998, the District Engineer issued a Record of Decision (ROD) committing to build a new 1200 x 110 foot lock at a location north of the Claiborne Street Bridge on the west bank of the canal near an area know as the Galvez Street Warf. The ROD plan proposed the following:

- Demolition of the Galvez Street Warf, the US Coast Guard Facility, and several other businesses along the industrial canal.
- Excavation of a temporary by-pass channel along the east bank
- Dredging the canal bottom and driving piles for the foundation of the new lock
- Disposing of approximately 3 million cubic yards of dredged sediments into the Mississippi River, at a mitigation site for wetlands creation, and a confined disposal site on the MR-GO, and as backfill during construction of the new lock.
- Off-site construction of five lock modules that will be floated into place and ballasted into place to form the new lock.

The Corps estimated the project will cost more than $800M and take over 12 years to construct.

Unsatisfied with the Corp's response to address concerns related to the dredging and safe disposition of potentially contaminated sediments located within the industrial canal, locally organized groups from the Holy Cross and By Water communities sought injunctive relief to enjoin the Corps from the commencement of work. In 2004, the court issued the Corps of Engineers a motion to stay proceedings so that it could conduct further testing and analysis of the canal sediments. In February 2006, the court lifted the stay, and allowed both parties to file motions for summary judgment.

As a result of vast socio-economic and demographic changes that occurred within southern Louisiana as a result of the Hurricane Katrina storm surge and flooding damage, Judge Eldon E. Fallon of the U.S. District Court, Eastern District of Louisiana rendered a decision on 4 October 2006 requiring the New Orleans District to prepare a new or supplemental Environmental Impact Statement that addressed construction of the IHNC Lock replacement project. The court ruled "In light of Hurricane Katrina, the underlying purpose of NEPA will not be served if the Corps moves forward with the Industrial Canal Project according to a plan devised almost a decade ago. The Court notes that the Corps, at a minimum, must prepare a supplemental EIS addressing the significant new circumstances relevant to environmental concerns that have arisen since Hurricane Katrina." The New Orleans District is currently evaluating the most appropriate alternative for addressing the courts ruling. A copy the court ruling in HOLLY CROSS, ET AL. –vs- U.S. ARMY CORPS OF ENGINEERS; CIVIL ACTION NO 03-370 SECTION "L"(4) is included in this letter report as Appendix I.

3.3. Impacts

The proposed Lock Replacement project as described in the 1998 ROD is located in the midst of a highly developed and densely populated part of the city. In fact, the areas adjacent to the IHNC are among the oldest and most established neighborhoods in New Orleans and include two nationally designated historic districts, Holy Cross and Bywater.

The magnitude of the project and the estimated duration of the implementation phase are such that it is likely to have a significant impact on the neighborhoods, historic resources, residents, and businesses located therein. Construction activity associated with lock and bridge replacements generate both adverse and beneficial impacts to the neighborhoods in the area.

Even with the innovative engineering of a new lock and the development of the tentatively selected plan north of Claiborne Avenue, there will still be significant impacts on the affected communities. While it is virtually impossible to eliminate all impacts associated with the construction of the lock project, it is possible to mitigate their effect on the community and its resources.

To assure that these impacts are fully captured and addressed in the comparative cost analysis, all significant issues identified in the EIS have been tabulated and organized in Appendix G of this letter report. For the purposes of this report, the impacts were organized by issue, recourse, and commitment. Commitments that could be quantified as a constraint on construction methods or efficiency have been factored into the project cost and schedule estimates so that an accurate comparison between the Cast-In-Place and Float-In alternatives can be formulated.

3.4. Mitigation Plan

Section 844 of the Water Resources Development Act of 1996, PL 104-303, dated October 12, 1996, authorized implementation of the community impact mitigation plan as follows:

> (c) Community Impact Mitigation Plan - Using funds made available under subsection (a), the Secretary shall implement a comprehensive community impact mitigation plan to the maximum extent practicable, provides for mitigation or compensation or both, for the direct and indirect social and cultural impacts that the project described m subsection (a) will have on the affected areas referred to in subsection (b)." This authorization reaffirms Congress' intent to mitigate project impacts on the community.

The community impact mitigation plan recommended as part of the lock project represents a departure from traditional Corps of Engineer environmental analysis and mitigation planning and was developed through a broad-based community participation process in the form of a neighborhood working group. Participants in the process from the community maintained their strong opposition to the project during the discussions, but still provided valuable input toward the formulation of the community impact mitigation plan. The plan insures that communities adjacent to the project remain complete, livable neighborhoods during and after construction of the project. It also minimizes residential and business disruptions while meeting the goals of improving waterborne commerce.

The plan includes direct impact minimization actions that will be taken by the Corps in cooperation with local government, community groups, and residents. It also includes measures to indirectly compensate for those impacts which direct impact minimization cannot properly address.

The plan costs an estimated $33,000,000 to implement. It addresses the impacts relating to noise, transportation, cultural resources, aesthetics, employment, community and regional growth, and community cohesion. It also includes features intended to serve as compensation to the neighborhood for impacts that are not quantifiable. Implementation of the plan will begin prior to construction and will continue throughout the project construction period. The plan includes, in part, job training, business assistance programs, street and house improvements, community facilities, cultural and historical markers and displays, and new roadways.

To adequately implement the plan and to ensure that all of the stakeholders are involved in the implementation process, the New Orleans District proposed that a Partnering Agreement be entered into among all concerned residents, local interests, and officials. The agreement would commit all concerned to work together for the benefit of the community and to determine how the $33 million would be expended. Details of this would be developed through continued discussions with all concerned once the project is approved for construction funding.

4. Project/Site Description

4.1. Site Description

The replacement lock site is located near the east bank of the Mississippi River at mile 92.6 AHP. The lock will be located in the Inner Harbor Navigation Canal (IHNC) at the North of Claiborne site approximately one mile north of the Mississippi River, and about one-half mile north of the existing lock. The replacement lock centerline will be approximately 40 feet west of the centerline of the existing canal.

Construction has been completed for demolition and environmental restoration of the abandoned industrial sites on the east side of the canal adjacent to the future location of the new lock. The $29 million contract let to Washington Group, involved removal of aboveground and underground structures and canal-side obstructions, and also included extensive environmental restoration. The area, visible from the North Claiborne Avenue Bridge, is now green. The work was completed in June 2005. A second construction contract, for demolition of the Galvez Street Wharf, was awarded to Virginia Wrecking Co. in April 2001 and completed in February 2003. After wharf demolition, nine mooring buoys were emplaced to protect the exposed bank line and enhance navigation. The next contract, pending funding, will be for construction of a levee/floodwall along the west side of the canal from St. Claude Avenue to the Mississippi River.

The real estate was purchased from the Port of New Orleans for $16.8 million. The final act of sale took place Dec. 19, 2002.

On July 30, 2005, the Corps began to collect soil, sediment and water samples in the canal to insure the proper management of material that will be dredged later in the lock project.

4.2. Relocations Summary

There are three existing movable bridges located on the Inner Harbor Navigation Canal between the Mississippi River and the Mississippi River Gulf Outlet: Florida Avenue (northernmost), Claiborne Avenue (LA Route 39), and St. Claude Avenue (LA Route 46) (southernmost). The Florida Avenue Bridge is not part of the IHNC project and is operated and maintained by the Port of New Orleans. The existing St. Claude Bridge will be demolished and replaced with a low level, double bascule bridge with a 200' clear horizontal span. The Claiborne Avenue bridge superstructure will be replaced.

4.2.1. Replacement of St. Claude and Claiborne Avenue Bridge Crossings

The existing St. Claude Bridge will be demolished and replaced with a low level, double bascule bridge with a 200 foot clear horizontal span. The replacement bridge is intended to give priority to navigation traffic (no curfew). Vehicular traffic is intended to remain status quo after construction.

The bridge design concept involves constructing the replacement bridge along the same alignment as the existing bridge with traffic being diverted to the Florida Avenue and Claiborne Avenue bridges during the 18 months of construction. However, based on public comment from the 1997 Limited Reevaluation Report (LRR) a four lane detour will be provided 100 feet north of the existing bridge. The detour will include two single leaf bascule bridges that span both the existing lock and the demolition by-pass channel.

The replacement bridge for Claiborne Avenue will be of the same type as the existing bridge which is a mid-level, vertical lift span bridge. The Claiborne Avenue bridge superstructure will be replaced with higher towers and a new movable span. New mechanical and electrical equipment will be installed. In the initial 1997 LRR the plan was also to retrofit the existing piers, however after consultation with A/E's specializing in bridge construction this portion of the project was deleted.

4.2.2. Florida Avenue Bridge

As stated previously, the Florida Avenue Bridge is not included in the Corps IHNC lock replacement project. This bridge is operated and maintained by the Port of New Orleans. The existing Florida Avenue Bridge is a single leaf bascule with two vehicular lanes (one eastbound, one westbound) and two railroad lines.

The Float-In construction method is predicated on removal by local interests of the width restriction of approximately 90' at the existing Florida Avenue Bridge. The construction of the new low level vertical lift bridge was completed in 2005. The new railroad bridge provides two at-grade lanes for vehicular traffic and 156 feet of vertical and 300 feet of horizontal clearance.

In addition, the Sewerage and Water Board siphon structure located adjacent to the existing Florida Avenue Bridge has a 105-foot clear width (without fenders) and 90-foot clear (with fenders). Removal of the siphon is a local interest responsibility in conjunction with replacement of the Florida Avenue Bridge. This restriction would also have to be removed in order for the Float-in construction method to work as designed. Again, the risk associated with the siphon structure not being removed will be evaluated with the Float-in construction method.

The Cast-in-Place construction method appears unaffected by the bridge replacement.

4.3. Digital Terrain Model

LIDAR (Light Imaging Detection and Ranging) survey data was provided by MVN. A digital terrain model was created from this data and utilized for quantities, site layout, design and cost comparison.

5 Items Common to Both FIP and CIP

After completion of the 1997 feasibility report, several features of the project have changed based upon further design and analysis. The lock gates have changed from miter gates to sector gates and the culvert valves have changed from tainter valves to vertical roller valves. The following features were screened to be common for both plans and did not appear to contribute to the cost differential between the plans.

5.1 Levees and Floodwalls

The Mississippi River flood protection levees and floodwalls for this plan must be extended from the existing lock downstream approximately 2500 feet on the east and west banks to tie into the new lock as shown on Exhibit No. COM-5. Reinforced concrete walls will connect the gate bay monolith to the existing protection levee south of the new lock.

5.2 Temporary Bypass Channel

While the new lock is being constructed, a bypass channel will be dredged on the east bank of the canal. The channel will be capable of passing 2 way barge traffic (Elev. -12 and 220 feet wide) and capable of passing one way ship traffic (Elev. -31.5 and 110 feet wide). For the CIP plan the lay out of the cofferdam projects farther into the river. This causes the bypass channel to shift eastward but the excavation will not impact the integrity of the existing floodwall.

5.3 Sources of Concrete and Concrete Aggregates

New Orleans. Louisiana is located entirely within the Gulf Coastal Plain and Mississippi embayment. By nature of its geology, it is an area with poor aggregate potential. The main source of stone mined in-state is a hydrite (a form of gypsum) that is mined in Winn Parish (Autin and John, 1992) about 400 km northwest of New Orleans. In some areas of the state, salt-dome caprock is mined and used for the construction of light duty aggregate surfaced roads. Shell material dredged from the Gulf of Mexico and Atchafalaya Bay (about 120 km southwest of New Orleans) is used as road base. For many years, shell materials were also dredged from Lake Pontchartrain and filled part of the state's aggregate needs. During 1959, approximately 1.5 million metric tons of shell (over half the state's combined production of stone and shell) was dredged from Lake Pontchartrain. However, a ban issued during 1990, and upheld by a state appellate court during 1992, has curtailed dredging from the lake (White and Marsalis, 1994).

The New Orleans Metropolitan Area requires a considerable amount of aggregate to meet the needs of new development, highway construction, and post Katrina reconstruction. The absence of stone that is of reasonable quality for use in concrete and structural fills makes New Orleans dependant on imports from outside the state. Much of the states crushed stone is barged from quarries in Texas, Arkansas, Kentucky, Missouri, and Illinois, and shipped by rail from Kentucky, Arkansas, and other states east of the water routes.

Specific price quotes for rip-rap and aggregates obtained for the development of the CIP and FIP cost estimates were obtained from numerous vendors and material companies located in the southern and gulf coast areas. Quarry locations operated by Vulcan Materials Company are shown below for illustration purposes, and represent one of many materials companies capable of meeting the aggregates and rip-rap specifications for this project.

Figure 5.1

5.4 Stone Slope Protection

Temporary Protection

Due to the close proximity of the by-pass channel to the east bank of the canal, stone slope protection will be required where the existing bank transitions to the proposed bypass channel to protect the channel bank and flood wall from erosion. A graded stone riprap revetment is proposed to minimize bank loss that may occur from vessel generated wave action and secondary wave action generated from the echo effect from the cofferdam. Although the Cast-In-Place alternative shifts the navigation by-pass channel further east and is therefore more likely to impact the canal banks if no protection is installed, both alternatives require erosion control revetments due to the erodible nature of the soils and the close proximity of the East Bank canal floodwall that protects the Lower 9th Ward. With the exception of the reach along the closure section that will receive additional backfill, the graded stone revetment will remain in place after construction.

Permanent Protection
The north and south Lock Approach Channels will require armor stone to protect the channel slopes and toe region from prop wash, scour, and vessel generated wave action. ERDC should be consulted in the next design phase for assistance on sizing and selecting a suitable size and gradation of large armor stone that will resist prop-wash and scour from the barge tows and Ships that utilize the lock. There will be approximately 780,000 tons of riprap stone available for use as bank protection after removal of the cofferdam.

5.5 Hydraulic - Filling and Emptying System

The lock chamber geometry consists of ports in the walls. Flow through the culvert system is controlled by four vertical roller gate valves located in the sector gate monoliths. The 1997 Evaluation Report reported the following filling and emptying times for the 1200' lock with a ship having a hawser limit within the range of 10 tons and 30 tons and utilizing the miter gated lock:

Valve Time (min)	Lift (ft)	Filling Time (min)	Emptying time (min)
4	3	4.14	4.21
6	7	6.31	6.43
7	11	7.70	7.88

The mean stage of the IHNC on the northeast side of the lock is 1.37' NGVD. The maximum stage at the IHNC lock of 10.65' NGVD occurred on September 10, 1965 during Hurricane Betsy, and the lowest stage of -2.00' NGVD occurred on April 12, 1988.

Hydraulic Design Stages.
The following water surface elevations (NGVD) are provided for design case analysis.

Load Case

Operating Cases:	*River Side*	*Lake Side*
Hurricane, Maximum Head Stillwater	0.0	13.0
Hurricane, Maximum Head Stillwater + Freeboard	0.0	14.0
Operating, Maximum Direct Head	17.6	-2.0
Operating, Maximum Direct Head + Freeboard	22.4	-2.0
Operating, Normal	10.0	1.0
Maintenance Condition, Dewatered	10.0	5.0

5.6 Hydraulic Model of Bypass Channel

ERDC has a physical model of IHNC Canal and Lock that has been suspended from service until final design requirements have been determined. An annual facility fee is assessed for storage and maintenance of the model. Upon final decision of the CIP and FIP construction alternative, it is recommended that the model be reactivated, and navigation studies be performed to evaluate potential barge and ship navigation hazards that may arise as a result of the proposed cofferdam alignment and the significant encroachment of the structure onto the shipping lanes. Other ERDC Model studies should evaluate erosion protection requirements in the new lock approach areas.

5.7 Lock Chamber Monoliths

Lock chamber monoliths will enclose the lock between the upper and lower gate bay monoliths. The cast-in–place alternative lock will consist of five chamber monoliths. The proposed float-in place lock chamber is designed to be constructed with a total of three chamber monoliths, that in conjunction with the gate bay monoliths, will provide a chamber 110 feet wide by 1200 feet long (useable length). The lock chamber floor will be set at El. -40.0 (NGVD) with the top of the wall set at El. 23.0 (NGVD). See Exhibit No. FIP- 2 for details. The chamber monoliths will be pile supported reinforced concrete U-frame structures of uniform cross section. Each monolith will be designed independently to support any lateral earth pressure or hydrostatic loads. Hawser loads will be included in the design of the upper part of the lock wall. To prevent concrete damage the lock chamber will be protected with wall armor and corner protection where applicable. See Exhibit No. FIP-3 for detai1s.

5.8 Gate Bay Monoliths

The proposed gate bay monoliths located at each end of the lock will be designed to house the sector gates and the machinery used to actuate the gates. The gate bay floor will be set at El. -40.0 (NGVD), with the top of wall set at El. 23.0 (NGVD). The monolith will allow the gates to be recessed flush with the face of the lock wall when in the open position. Slots will be provided upstream and downstream of the sector gates to allow for emergency and maintenance dewatering, by bulkheads. Each monolith will be designed to distribute the concentrated gate loads as well as any lateral earth pressure or hydrostatic loads. To prevent concrete damage the gate bay monolith will also be protected with wall armor and corner protection where applicable. Protection against seepage under the gate bays will be provided by a steel sheet piling cut-off wall extending across the monolith. See Exhibit No. FIP-3 for details.

5.9 Sector Gates

Subsequent to the original study, CEMVN-OD requested the investigation of a sector gate alternative for the 110-foot by 1200-foot ship lock, which was the primary focus of the Sector Gate Appendix, dated March 2003. The appendix concluded that costs turned out to be roughly comparable and were, therefore, a non-issue. CEMVN-OD elected to pursue the sector gate option for the 110-foot by 1200-foot ship lock.

The existing design will be completed in accordance with EM 1110-2-2105 and EM 1110-2-2703 and applicable industry standards. The sector type gates will be all welded structural steel construction. The gates will have a central angle of approximately 75.25 degrees. The radius to the inside of the skin plate will be 52 feet 6 inches and have an overall height of 63 ft. Loads applied to the gates skin plate will be transmitted to horizontal girders through vertical tees. The girder load will then be transmitted through 3 vertical trusses and four horizontal frames to a hinge and pintle anchorages. Pintle anchorages will be embedded in the base and reinforced as required to transfer thrusts and overturning moments from the pintle directly into the base, rather than through the walls. The hinge anchor rods will transfer the gate thrust into the upper wall near the top of the lock. The anchor rod design will account for any prying action caused by eccentric loading from the hinge thrust. Each gate will be modified by the addition of projections or "ears" that will permit water to flow around the gate and emerge from beyond the pintle, approximately at a right angle with the centerline of the lock. The skin plate thickness will be increased 1/16 inch for corrosion.

Gate Support

The gate frames will be supported at the top by a hinge and at the bottom by a pintle. In order to assure good pintle and hinge alignment, a spherical pin will be used in the hinge to compliment the spherical pintle. Horizontal reactions will be transferred to the lock wall through the bronze bushings. All vertical loads will be transferred to the concrete base through the pintle. Anchor bolts will be used for the hinge anchorage. In order to insure firm contact between the movable and the fixed hinge castings, under all conditions, the anchor bolt nuts will be tightened sufficiently to induce a pre-tensioning stress in the bolts.

5.10 Bulkhead Closure System

The project will require a bulkhead closure system located upstream of the upstream sector gates and downstream of the downstream sector gates. This closure system will consist of steel framed bulkheads in bulkhead slots located in the U-frame lock walls. The bulkhead sill will be integrated in the U-Frame lock. The top elevation of the bulkhead sill is El -40 feet.

5.11 Sills for Sector Gate and Closure Systems

The project will require upstream and downstream sector gate sills. The sector gate sills will be integrated in the U-Frame lock. The elevation of the upstream and downstream sector gate sill is EL -40.

5.12 Lock Appurtenances

The faces of the lock walls are equipped with accessories to facilitate navigation and operations. The layout and design of navigational aids is based on guidance and recommendations contained in ETL 1110-2-2602, Planning and Design of Navigation

Locks, dated 30 September 1995, and the recommendations of project personnel. In order to prevent concrete damage due to the rubbing and scraping action of vessels, vertical runs of T-section wall armor will be provided along faces of the walls within the limits of the lock chamber. Horizontal and vertical corner protection, preformed plates and corner cap castings will also be used at the top edge of the walls and at all exposed corners. Ladders and ladder rungs recessed in the face will also be provided throughout. These ladders will have a resting platform which can be used on long climbs. An equipment surcharge load of 200 psf shall be applied to the applicable structural components. A uniform live load of 150 psf shall be applied to all walkways. A Hawser Loads of 160 kips shall be used for the design of line hooks and check posts.

5.13 Lock Guidewalls and Protection Cells

Fixed timber guidewalls are provided on the lakeside as shown on Exhibit No. COM-3. The lakeside eastern guidewalls will extend 800 feet. Placing the guidewall on the east side avoids obstructing entrance into the turning basin south of Florida Avenue. The riverside (south) guidewall is also located on the east side. The ship lock guidewall is 1200 feet long. The riverside (south) guidewalls were designed as floating guidewalls (See "Inner Harbor Canal Lock Replacement: DDR, Phase I Design, 100% Submittal, URS group 2006"). This will be reviewed in post-feasibility studies to see if a more cost effective alternative exists, such as fixed guidewalls. Sheet piling cells will be used at the ends of the fixed timber guidewalls, ends of guardwalls, at the south end of the bypass channel, and outside the lock construction area. Pile supported, steel sheet pile, concrete filled dolphins will be provided at the end of each timber guide wall.

5.14 Mechanical Design

The machinery and mechanical systems will be identical for either construction method. Therefore these systems were not further evaluated from the 1997 Report.

5.15 Electrical Design

The Electrical systems will be identical for either construction method. Therefore these systems were not further evaluated from the 1997 Report.

5.16 Buildings

The following buildings are anticipated being required for the final lock design.

Control House

Two – (2 20' x 20' and 2 15' x 15') One story reinforced masonry building with a metal roof supported on open web joists. Control House shall include storage area and Restroom facilities. This facility will be designed to withstand hurricane forced winds.

Maintenance and Administration building

Two Story 50’ x 75’ Pre-Fabricated Rigid steel frame on a timber pile founded 8” slab. Wall enclosure shall be insulated 22 gauge Panels. First floor 14’; second floor 8’.

Maintenance building with Emergency Generators
25’ x 45’ clear span machinery building which includes emergency generators. Building shall be reinforced masonry located on a lock wall. Metal Roof supported on open web joists. One Story, 14’ ceiling.

5.17 Demolition of Existing Lock

The existing lock will be demolished after the new lock is in place and in operation. Demolition requires a complete shutdown of the existing lock. The disruption to navigation will be kept to the minimum possible time to complete demolition and debris removal. The south bypass channel will be in place prior to existing lock demolition. The walls and concrete base slab (9-12’ thick) of the existing lock will require removal.

Demolition Plan
After the replacement lock and tie-in levees are in place and the pool is raised, the existing lock will be demolished. The existing lock must be removed in its entirety for completion of the 200-foot bottom width replacement channel to full width. The south bypass channel depth of (-) 12 feet is contained in the (-) 36 foot final channel cross-section.
A demolition expert, DYKON, Inc. of Tulsa, OK, was consulted to find the best demolition method for the existing lock. Methods were considered to demolish the lock using: 1) conventional (non-explosive) demolition methods, and, 2) explosive methods or 3) a combination. Also, the demolition was considered as either "in the wet" or "in the dry". Site preparation costs were determined by constructing a cofferdam and dewatering the lock, as well as the estimated cost of downtime to navigation. Debris removal costs were also developed. Duration of demolition was developed for wet and dry plans.

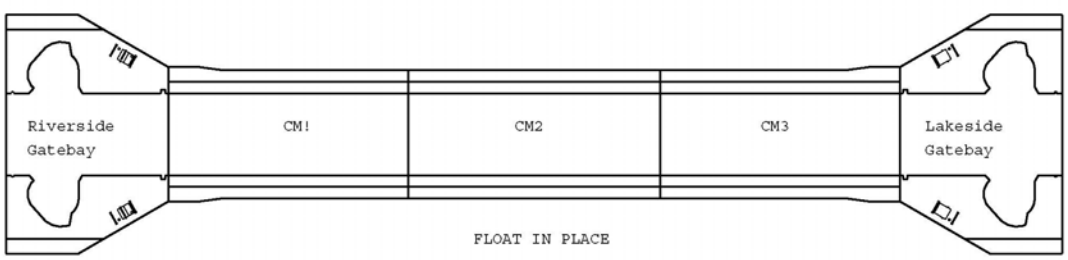
Comparison of "In-the-Dry" and "In-the-Wet" Demolition Plans
“In-the-Dry" Plan - This requires constructing a cofferdam and dewatering of the entire existing lock structure for demolition in dry conditions. This speeds demolition operations since lock features are accessible and construction equipment can work at a higher production rate. Debris removal will be by truck to barge loading areas outside the cofferdam. Navigation must be shut down for the duration of debris removal operations, plus any cofferdam installation and removal time. No navigation bypass channel is provided, thus alternate routes such as Baptiste Collette must be used for navigation. To provide a bypass channel in this option would require major degrading of an MRL mainline levee, thus no bypass channel is practical.

"In-the-Wet" Plan - No cofferdam or dewatering system is required; all demolition operations are done in wet conditions. Demolition operations are slower than "in the dry" due to drilling of holes in locations not visually accessible and thus the

production rate is lower. Debris removal will be by barging to a suitable disposal site. Heavier construction equipment is required for debris removal since pieces may be larger and less accessible. Diver operations will be needed for many features.
A one-way, approximately 85-foot wide bypass channel to elevation (-) 12 NGVD (shallow draft only) will be constructed to allow navigation to continue during most of demolition operations. Navigation is stopped only for a matter of hours during actual detonation. Demolition in segments would proceed probably from the (south) river end.

Summary - Existing Lock Demolition Plan
The "In the Dry" plan offers the lower cost for lock demolition alone. With the additional cost of dewatering and constructing a cofferdam around the old lock, as well as a greater financial loss to navigation due to rerouting makes this the more costly alternative. The "In the Wet" plan will take longer to complete, however, the bypass channel called for by this plan will allow the IHNC to remain open during demolition except for the periods of actual detonation. Thus, the "In the Wet" plan is recommended.
The demolition expert recommends demolition by a combination of explosive and conventional (non-explosive) methods. The above-ground portion of the lock will be demolished using conventional methods such as a hoe-ram and/or wrecking ball. The underwater portion will be demolished with explosives. Debris removal will be using heavy crane equipment to handle the larger pieces of an "In the Wet" operation. The existing lock will be demolished upon completion of the replacement.

6 Float-In-Place Design and Construction, Considerations and Criteria



6.1 Summary of Float-In-Place

The replacement lock will be located north of the existing lock. The structural design will be in accordance with COE guidance and applicable industry standards. The lock design consists of a precast, post-tensioned, float-in concrete lock. The top of the replacement lock wall is elevation 23 feet NGVD. The lock chamber measures 1287.66 feet C-C of the pintles, and 110 feet in width. The lock has sector gates as shown on exhibit COM-9. The filling and emptying system uses a vertically operated roller gate located in the gate bay monolith culverts as shown on exhibits COM-7 and COM-8. The lock culvert is 15 feet wide by 18.25 feet high for the 1200-foot lock. The maintenance bulkheads are as shown on exhibit COM-12. The replacement lock has a sill elevation (-) 40 feet NGVD. The filling and emptying system consists an interior, ported culvert and manifold system as shown on exhibit CIP-3 and CIP-4. The lock structure is pile-founded. Because of the amount of calculated differential settlement the option of soil founding the concrete structure was eliminated. The pile foundation shall be grouted to the concrete base with tremie concrete. The AE firm URS was tasked to prepare a Design Documentation Report (DDR) for the Float-In-Place (FIP) option for IHNC. Significant features of the Float-In-Place include:

- Build offsite graving yard
- Build, transport, set down 5 modules.
- Concurrently drive 48" piling
- Grout walls
- Place equipment

6.2 VE Study

A Value Engineering (VE) study was conducted 16-20 May 2005 on the Float-In construction method for the project. The team was led by Bill Easley from OVEST and consisted of members from various disciplines and districts throughout the USACE. The CELRD team evaluated the study report and proposals and the evaluation is in Appendix D. Although, the recommendations from the VE study are valid, they will not be incorporated into the FIP design. The Float-in-place design is already farther along than the Cast-in-place design; therefore, if additional details for the Float-in design were developed then it would simply increase the "gap" between the two designs. In addition, the Cast-in-place design has not had the opportunity to go through a VE study and possibly achieve some savings through the process. Therefore, after consultation with CEMVN the CELRD team did not include the recommendations from the VE study into the comparison of the two alternatives.

6.3 Geotechnical

Design Documentation Report No. 3, Lock Foundation Report, was prepared by MVN and approved in May 2002. This report contains the results of the subsurface investigations, laboratory testing, bank stability analysis, and pile testing that was performed in support of the IHNC Lock Design. The results of these geotechnical investigations were used as the design basis for the work performed in this Float-in-Place vs. Cast-In-Place comparison letter report.

6.3.1 Subsurface Conditions

In general terms, soil conditions at the project site consist of natural levee deposits underlain by marsh and intradelta deposits. The marsh and intradelta deposits extend to El. -32.0, and consist of very soft medium clays with silt lenses. Interdistributary deposits underlain by prodelta deposits, consisting of very soft to stiff clays, silt lenses, and sand layers, are found between El -32.0 and El. -70.0. Below El. -70 are Pleistocene deposits of stiff clays, silts, and sands.

6.3.2 Soil Properties and Profiles

Subsurface soil profiles and corresponding shear strength parameters as presented in CEMVN Design Documentation Report No. 3, Plates 20 - 23, and 28, were utilized to perform the QA check of the CEMVN slope stability analysis.

6.3.3 Slope Stability

<u>Stability Analysis Criteria and Methodology</u>: In accordance with EM 1110-2-1902, the minimum Factors of Safety for the End of Construction (Undrained) and Long-Term (Drained) Load Cases are:

End of Construction F.S. $_{min}$ = 1.30
Long Term F.S. $_{min}$ = 1.50

<u>End of Construction Load Case:</u> The stability analysis presented in Design Documentation Report No. 3 for the FIP alternative utilized the MVN Method of Planes stability software program, and met the minimum Factor of Safety of 1.3 criteria using the adopted Q strength design parameters. Results of the Stability Analysis of the West Bank are shown in Figure 6.1.

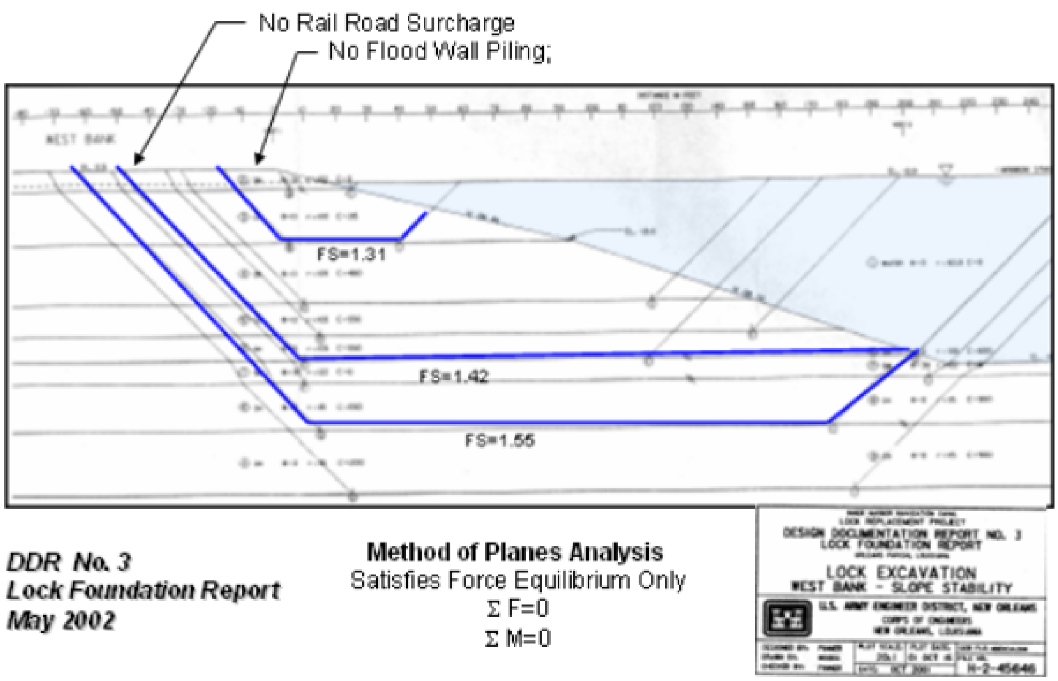


Figure 6.1

As per the scope of work, preliminary slope stability analyses were performed to confirm stability of the proposed west bank canal slopes for both the Float-In-Place and Cast-In-Place alternatives. Soil profiles and material strengths were taken from DDR No. 3.

<u>Model Calibration with MVN Method of Planes</u>. The Slope/W software code by GeoSlope International was used for the slope design effort. Stability model calibration was verified by comparing identical failure surface results from the

Method of Planes analysis in DDR No. 3, with the results from the Slope/W code. The MVN Method of Planes (MoP) satisfies Force Equilibrium only, and therefore it is difficult to directly compare results with other stability programs that solve for both Force and Moment Equilibrium. Janbu's method (which satisfies Force Equilibrium only) was selected as the analysis type which most closely simulated the Method of Planes calculation method.

The Factor of Safety results from the 3 trial surfaces shown in Figure 6.2 were calculated using GeoSlope's Janbu Method, and are higher than the same trial failure surfaces using the MVN Method of Planes. These findings are consistent with recent studies by Wright (University of Texas, 2006) who found that the Method of Planes Analysis tends to result in a more conservative (i.e.: lower) Factor of Safety when compared to other stability methods.

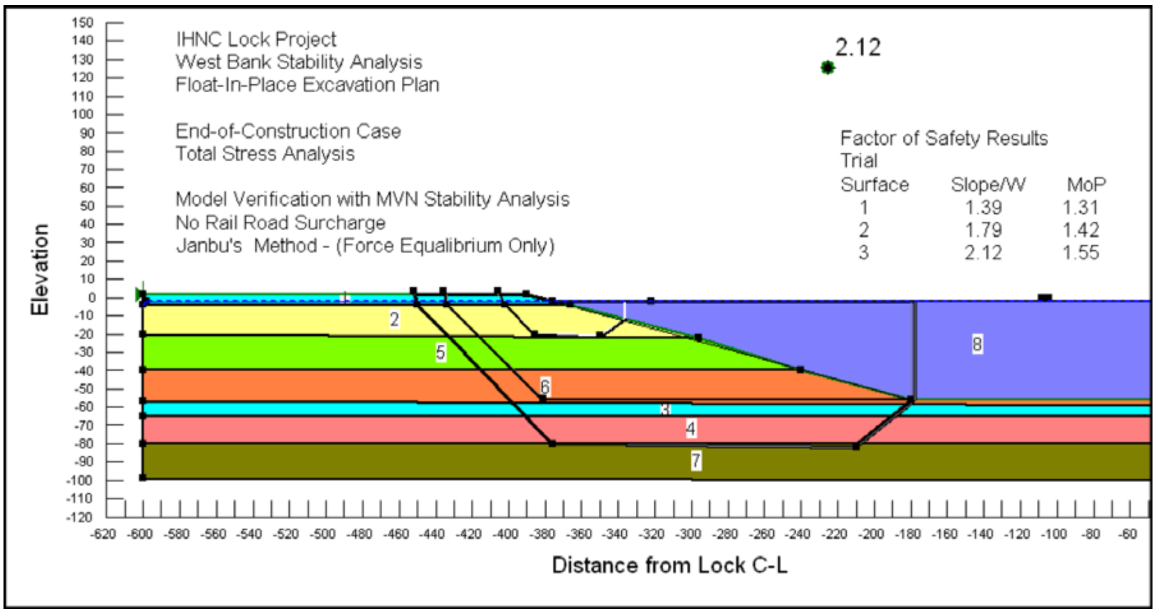


Figure 6.2

The Slope Stability Analysis of the West Bank performed in DDR No. 3 did not include the Railroad Surcharge. Due to the close proximity of multiple railroad tracks and sidings that are located along the top of slope as shown in Figure 6.3, additional stability analyses were performed to model static effects of the railroad surcharge on the stability of the excavated riverbank.

Railroad Surcharge Loading

Railroad surcharges were distributed at the top of bank in accordance with the American Railway Engineering and Maintenance Right-of-Way Association (AREMA) Manual for Railway Engineering, Section 1.3.3, which states that for Live Loads with Four or more tracks, full live load on 2 tracks, ½ live load on 1 track and ¼ live load on the remaining tracks. Based on the aerial photographs at the critical cross-section location A-A adjacent to the West Bank Gate Monolith, 4 tracks meeting the AREAM criteria were modeled in this location. The effects of additional track siding load surcharges should be evaluated in Final Design.

Conrail and Norfolk Southern require Cooper E-80 loading and specify 8.5 feet wide Boussinesq Strip Load of 1880 psf vertical pressure. CSX requires an 8.5 feet wide Boussinesq Load with 1800 psf vertical pressure. For the purposes of this study, the Railroad Surcharges were modeled in accordance with the Norfolk Southern (1880 psf) loading criteria.

<u>Site Conditions and Stability Model Features.</u> The excavation for the Float-In-Place construction alternative consists of a uniformly dredged 1v:3h slope. As shown in Figure 6.3, the most critical stability section is located at Section A-A, where there are active railroad sidings at the top of the cut-slope. Section B-B represents a single track load, and Section C-C represents the same slope geometry as section A-A, however there is no railroad surcharge loading.

Stability analyses were performed using the Slope/W stability modeling code by GeoSlope International. Spencer's Methods was selected for the analysis because it satisfies both Force and Moment Equilibrium, and the side force assumptions are consistent with Corps of Engineers stability analysis procedures.

Figure 6.3

<u>End of Construction Load Case - Stability of Section A-A</u>

As shown in Figure 6.5 below, when the AREMA criteria surcharge loadings are imposed at the top of slope at stability cross-section A-A, the resulting Factor of Safety (FS) = 1.25 and does not meet Factor of Safety criteria as specified in EM1110-2-1902. Because the top and toe of the slopes are constrained by the T-wall and the lock monoliths, flattening the slopes to achieve the minimum stability criteria is not a feasible option. Alternative slope stabilization methods such as retaining walls or ground improvements must be employed to achieve a stable slope while maintaining the constrained slope geometry requirements.

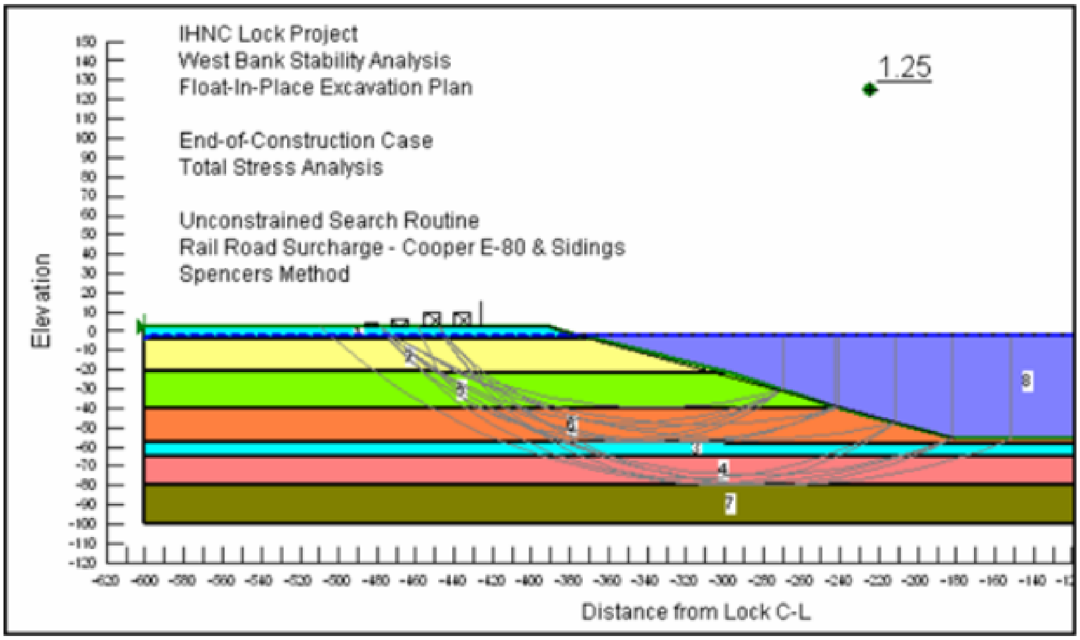


Figure 6.5

Slope Stabilization at Stability Section A-A.

Additional stability analyses were performed at Station A-A in an attempt to increase the Factor of Safety to meet Corps of Engineers design criteria. Because of MVN concerns with utilizing the pile foundation of the T-wall as a structural reinforcing member in the subgrade, soil improvements below the railroad were modeled in the stability analysis.

Stability Analysis results shown in Figure 6.6 and Figure 6.7 illustrate that the resulting Factor of Safety is increased to FS=1.31 if the shear strength of the CH (swamp) soils below the railroad line are increased from their in situ strength of C=215 psf to an improved strength of C=400 psf. Based on this analysis, deep soil mixing appears to be a feasible alternative for achieving Factor of Safety Criteria for this load case.

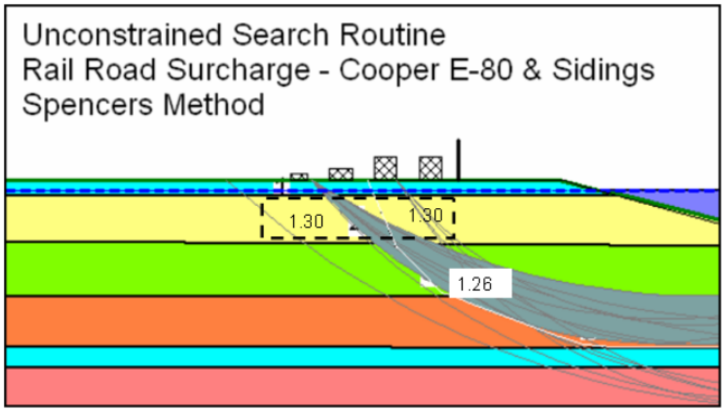


Figure 6.6

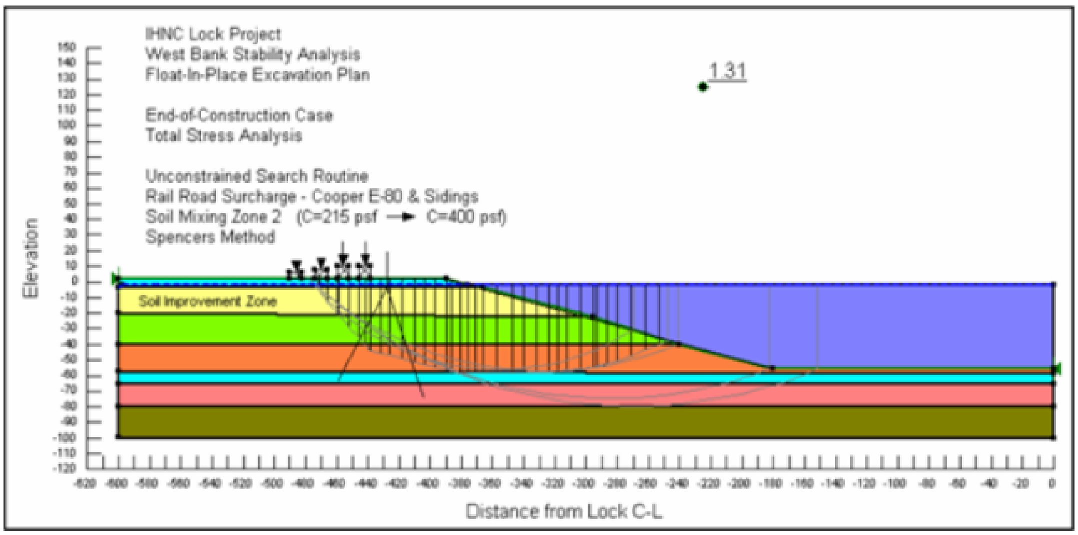


Figure 6.7

Although there may be opportunity in final design to eliminate the deep soil mixing requirement and include the benefits of the T-wall pile foundation into the stability analysis, it was not considered in this report. Therefore, deep soil mixing below the rail tracks is required to meet the Factor of Safety criteria for this load case, and the cost of this requirement has been included in the DELTA cost estimate for the Float-In-Place Alternative.

<u>Long Term (Drained) Load Case - Stability Section A-A.</u> Although this load case may not be fully applicable to the site conditions because it is unlikely that these soils will drain sufficiently to reach the drained strength condition, the unloading of the slope during dredging operations will likely create some negative pore pressures that will eventually dissipate, and this load case is intended to represent the in situ conditions after the soil has had sufficient time to readjust to the changes in stress and pore pressure. Since there is no Drained Shear Strength data in DDR3s, material properties for the West Bank were assumed to be the same as used on the East Bank

by URS Group. The drained shear strengths proposed by URS were based on data collected at other NOLA projects and is believed to be representative of the IHNC canal sediments.

As shown in Figure 6.8, the Factor of Safety for Long Term Stability was found to be 1.666, which exceeds the Corps minimum Factor of Safety Criteria of 1.5.

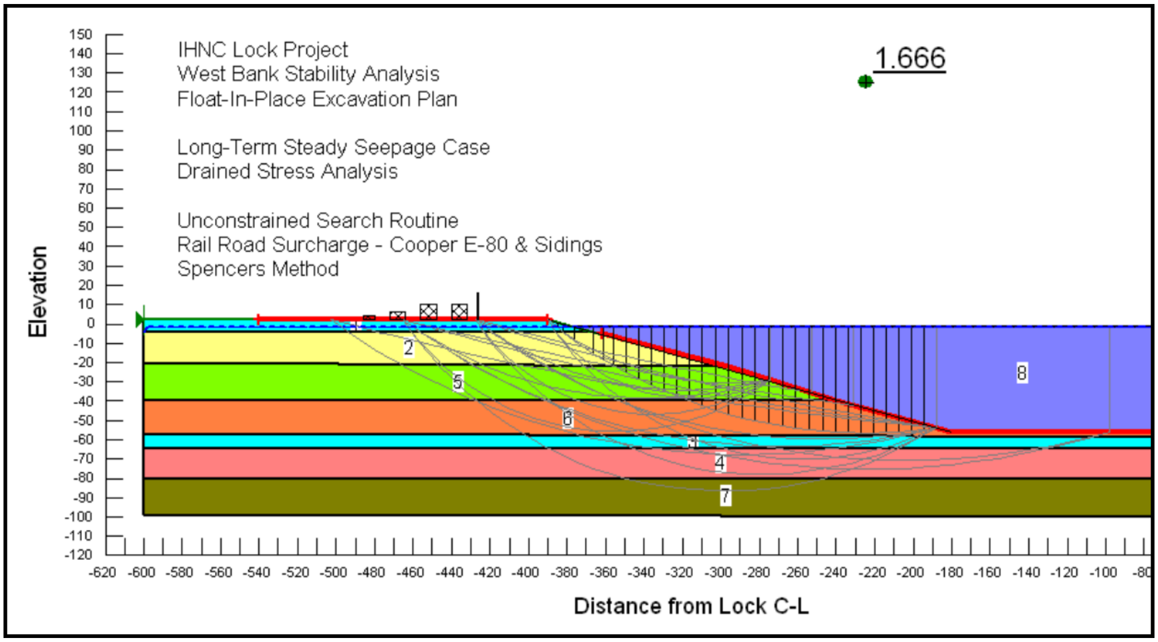


Figure 6.8

End of Construction Load Case - Stability of Section B-B

At stability cross-section B-B, the mainline railroad track pulls away from the top of bank alignment, and there are no railroad sidings at this location. The resulting Factor of Safety with only a single track surcharge loading resulted in a FS = 1.37, which exceeds the 1.3 minimum factor of safety criteria.

End of Construction Load Case - Stability of Section C-C

As shown in Figure 6.9, the critical failure surface at Section C-C where there are no railroad surcharges results in a FS=1.68.

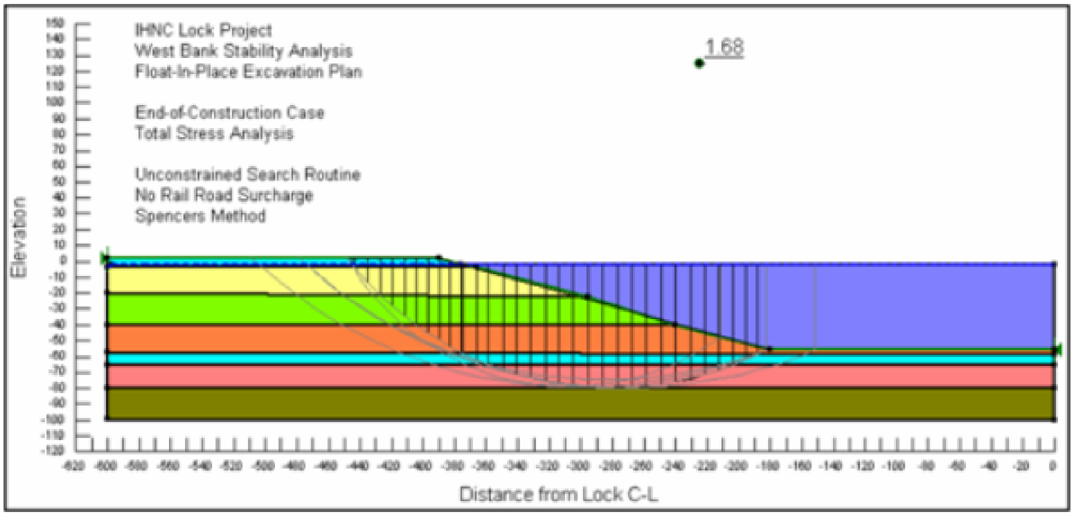


Figure 6.9

East Bank Stability Analysis along Temporary By-Pass Channel

Stability Analysis of the East Bank by-pass channel was performed by URS Group, Inc, under Contract No. DACW29-02-D-0008; Task Order 0002, dated October 2006. Reference Design Report "Cast-In-Place Cofferdam 100% Submittal Feasibility Level Design" for specific analysis results. In summary, the East Bank By-Pass Channel cut-slopes were found to exceed the minimum 1.3 stability criteria using the SLOPE/W stability code and soil parameters provided in DDR No. 3 for the East Bank Soil profile.

6.3.4 Foundations

The 48-inch diameter steel pipe piles were selected by the MVN and URS Design Team based on settlement criteria, layout considerations, geotechnical and structural capacity requirements, and constructability and handling considerations. MVN completed an extensive 48-inch pile testing program in 1999-2000 to determine their load carrying capacity as well as to determine driving characteristics and noise levels caused by different driving methods. 48-inch pile tests were performed by MVN for both the ship lock and barge lock loading cycles.

The data generated from these tests was used as the design basis for determining the pile depth and spacing requirements for the respective river side and lake side, gate bay and chamber monoliths. Separate pile analysis and design effort was performed by URS Group, Inc for the Riverside Gate Bay Module, the Riverside Chamber Module, the Lakeside Chamber Module, and the Lakeside Gate Bay Module.

The foundation plan consists of multiple rows of 120 feet long, 48-inch diameter, tremie concrete filled, steel pipe piles with a wall thickness of 5/8-inch. Pile spacing varies depending on the respective module, and ranges from 14 to 16 feet below the wall and gate monoliths, and 22 feet below the lock chamber slab. Reference

Drawings 4-1 and 4-2 in the URS DDR Phase 1 Report (August 2006) for the specific pile spacing proposed for each respective module.

Each module contains both landing piles and operational piles. The landing piles function as operational bearing piles after the tremie in-filling operations are complete. They are the same diameter and length as the operational bearing piles, however the load on the landing piles is established as a uniform reaction thru the flat-jack assembly at the top of each landing pile, and are arranged in groups to provide leveling capabilities to the module in the transverse direction. While most of the piles act in compression only, four rows of piles in the center of the chamber modules will have tension capacity mechanisms for connecting the piles to the base slab to resist the uplift pressures that are anticipated during maintenance dewatering.

6.3.5 Pile Driving Report

Workers from Boh Brothers Construction Company prepare a reaction frame for the test piles. A hydraulic jack pushes against the reaction frame to apply the test load to the piles. Piles were tested to a load of 1,125 tons (2,250,000 pounds). March 2000.

Boh Brothers Construction Company drives a 48-inch diameter test pile underwater. The pilings for the new lock will be driven underwater in the existing canal. January 2000.

Boh Brothers Construction Company drives a 48-inch diameter steel pipe pile with an IHC 200-S hydraulic hammer. January 2000.

Test Piles
Install 7 casings, 9 test piles and 48 reaction piles. Test for noise and vibration reduction, and load bearing. *Contractor:* Boh Bros. Construction Co., LLC *Contract Amount:* $1.6 million *Start:* December 1999. *Construction:* Completed, May 2000

6.3.6 Graving Yard Issues

The Graving Site was investigated at the preliminary design stage by URS Group under Task Order 1 of Contract DACW29-02-D-0008. Reference Inner Harbor Canal Lock Replacement, DDR Design Phase 1 Design, Final Submittal, Main Report Section Fourteen, dated August 2006 for a complete review of the design effort. The Report was approved in September 2006 following ITR certification.

The proposed graving site, which permits construction of the five float-in shells in the dry, is located in the southeast quadrant of the intersection of the Paris Road Bridge and the Gulf Intracoastal Waterway (GIWW) Channel. This site is a short distance from the lock site on the Inner Harbor Navigation Canal. The graving site was located in the northwest quadrant in the previous Feasibility Study (1997).

Configuration

The basic dimensions of the graving site are defined by the size of the concrete shells to be constructed, the flotation criteria of the shells, the height of the desired flood protection and the geotechnical slope stability analysis. The gate bay shell is 320-feet by 219-feet and the largest chamber shell is 180-feet by 340-feet. The base area is set at 320-feet by 440-feet and has a minimum of 50-feet working space around the casting bed for construction access. The height of the berm around the graving site is set at El. 7.0, which coincides with a 10-year frequency (stillwater level) for a hurricane surge event.

Graving Site for Braddock Dam

6.3.7 Dewatering

As per the URS design, the graving site pit will be unwatered from groundwater El.(+1.0) down to pit bottom El.(-32.5) using a series of pumps and sumps. Following the unwatering, a dewatering effort and pressure relief system will be required at the graving site for the duration of the casting and fabrication activities to address heave and seepage infiltration from 3 separate aquifers.

Well spacing for the dewatering system was estimated at 100 feet C-C for the upper sand and intermediate silt deposits, using 20 gpm pumps with a 50 foot drawdown capacity, and 300 feet C-C for the lower sand deposits, using 50 gpm pumps with a 50 foot drawdown capacity.

6.3.8 Slope Stability

As per the URS design, the graving site is located with the long dimension in the north/south direction based on the slopes needed for the initial earth closure berm and sheet pile walls that isolate the graving site from the GIWW Channel. The graving site is located in the east/west direction by setting the limits of construction a minimum of 110-feet from the Paris Road Bridge piers. More pier foundation information and additional geotechnical analysis is required to verify that this distance is sufficient. Any shifting of the graving site to the east would change the length of the access road but should not greatly impact the graving site excavation. Other features to the east of the graving site are electrical transmission towers, which are about 1,000-feet away, and a property corner to the southeast of the site that may be encroached upon.

The slopes used for the graving site are developed from the geotechnical slope analysis. The inside slopes from El. 0.0 to El. (-) 31.0 are 1 V on 5H. The graving site interior slope for the initial closure plug, adjacent to the GIWW Channel, is 1 V on 6 H. The berm from El. 0.0 to El. 7.0 is set back 40-feet from the top of the excavation for the graving site; it has 1 on 3 slopes with a 10-foot crown.

6.3.9 Module Casting Bed Design

As per the URS design, a casting bed is provided on the floor of the graving site with the top of the bed at El. (-) 31.0 to provide a level, load-bearing surface with minimal expected settlement on which to fabricate the shells. Two casting bed configurations were considered: First, a continuous, pile supported concrete slab and second, a system of pile-supported *grade* beams with granular fill between the grade beams. In discussions with the shell designers, it was decided that the grade beams with granular fill in-between the grade beams is preferable to a continuous, pile-supported slab. Unlike the continuous slab, the granular fill will allow the filling water pressure for flotation of the shells to act directly on the majority of the area of the shell keel slabs, facilitating release from the casting bed.

The grade beam will be supported by 14-inch square precast concrete piles, which was selected based on its combination of capacity and cost. The piles will extend from top El. (-) 33.5 to tip El. (-) 72.0 and will derive approximately 55 kips capacity from both skin friction and end bearing. Pile tips will bear on a dense sand stratum near El. (-)72

Over most of the casting bed area, there is slightly more pile capacity than required by the weight of the shells. Since it is estimated that construction loads on the casting bed are small over any significant area in comparison to shell weight, no additional piles have been added to carry superimposed construction loads. Should a construction contractor elect to locate an extremely heavy piece of equipment on the casting bed, an analysis of the total loads in the local area would be required.

To provide for construction operations, the areas between the grade beams and a 50-foot wide strip around the outside of the casting bed is excavated 18-inches below the floor elevation of the graving site, then covered with a compacted, crushed stone base material.

6.4 Structural

6.4.1 Structural Design Methodology

Designs of the riverside and lakeside gate bay modules and the riverside and lakeside chamber modules were performed by the A-E. The design was separated into two distinct design phases a float-in construction phase and an operational phase. The float-in design was to ensure that the modules were designed to achieve the necessary draft restrictions for transporting the modules from the graving site to the project site and to ensure that the modules can carry the loads to which they are subjected during steps 1-5 (see below). The operational design is to make certain that the module is capable of carrying all applied service loads.

The process that a module will undergo from the time of its fabrication and leaves the graving site to when it is placed in operation can be given by:

1. Transport condition (floating)
2. Ballasting sequence at project site
3. Set-down condition (landing piles only)
4. Tension Pile connections are engaged
5. Tremie concrete is placed under the base of the module (remaining piles are engaged)
6. Operational condition(s)

6.4.2 Design Criteria

The Design Criteria Document (DCD) was prepared by the A-E firm URS to guide the design process. The DCD established and documents the criteria that were utilized to develop the civil, geotechnical, foundation, marine, and structural designs for the project features that were designed and developed. The DCD was prepared and submitted to the Corps in June 2003. The DCD is considered a living document that has been revised and updated as the project design features evolve for the lock replacement. The latest DCD revisions were included in the Design Document Report, Phase 1 Design 95% - Submittal provided to the Corps in September 2005. The following are critical issues considered in the float-in design process:

Material Properties

Structural concrete, cast at graving site:	f'c = 5,000 psi
Structural concrete, infill at project site:	f'c = 3,000 psi
Reinforcing steel, ASTM A615, Grade 60:	fy = 60,000 psi
Unit weight of concrete shell:	155 lb/ft3
Unit weight of concrete infill:	122 lb/ft3
Structural Steel weight:	490 lb/ft3

Uplift Condition

The transverse cut-off walls are located to be in-line with the transverse tie-in levees at both sides of the lock. Full uplift was assumed under the modules to the line of transverse cut-off wall tie-ins. The effects of the sheet pile driven to contain tremie concrete along the module perimeter was not to be considered. A sufficient number of containment wall sheets are to be extracted to assure constant uplift pressure is active under the gate bay and a portion of the chamber modules to the transverse cut-off walls.

Backfill Levels

The Backfill levels have been revised to prevent blowouts of the ground surfaces created by the high riverside uplift pressures. The back fill sections are to have a clean cover. For construction, backfill is to be brought up evenly on opposing walls such that any transverse lateral loading would be within the capacity of the pile layout needed for loadings in the longitudinal direction. A 5-foot differential fill height was determined to be acceptable.

Transport Wave Loading
An original 7-foot transport wave loading was proposed and later eliminated. The wave heights were reduced to be more compatible with the assumed favorable weather conditions for float-out and transport of the modules. The wave height was revised to 2-feet for transport. For second stage construction or a lengthy set down installation procedure, a 4-foot wave is applied along the channel centerline and a 3-foot wave transverse to the channel centerline.

Float-Out Draft
The Module Draft Study considered shell drafts of 25-feet, shallow draft, and 32-feet, deep draft. Based on the Module Draft Study, an approximate design draft of 28-feet was selected for both the chamber and gate bay modules. The 28-foot draft does not include construction tolerances for weight growth. The water depth availability was increased to EL -32.0. This includes a 3-foot clearance over the average channel bottom at EL -35.0.

Transportation of Braddock Dam Module

Negative Buoyancy at Set-Down
The set-down stages are high Canal water surface at EL 3.0 and low water at EL 0.0. The 5% negative buoyancy is designed to occur at a water stage of EL 1.5. A lesser negative buoyancy was permitted at higher set–down water stages, but it must remain a negative buoyancy. The negative buoyancy includes adverse effects of permitted construction tolerances. Set-down piles are designed to resist the compressive loadings when the Canal water stage drops to EL 0.0. The pile factor of safety has been reduced since it is considered an unusual load condition which is permitted in EM 1110-2-2906.

Barge/Ship Impact
A barge impact load of 160 Kips was used for the chamber walls. A ship impact load of 750 Kips applied at a 20 degree angle to the wall was used for the chamber walls of the riverside gate bay module.

6.4.3 U-Frame Lock Analysis

Chamber Module 2 (CM2) and 4 (CM4) – Riverside Chamber Module
There were three primary computer analysis tools:

1) Spreadsheets of the itemized weights of the structure and naval architectural considerations (Microsoft Excel).
2) A global, three-dimensional, finite element, grillage model of the floating chamber module for beam flexural, shear, and axial forces (SAP 2000 version 9)
3) A two dimensional, finite element model for local slab and wall internal forces (SAP2000 version 9).

Lakeside (GB5) and Riverside (GB1) Gate Bay Modules
The primary computer analysis tools:

1) Spreadsheets of the various weights of the structure (Microsoft Excel).
2) A two dimensional, finite element model for riverside, middle, downstream and gate recess design strip analysis, culvert frame analysis, wall frame analysis and wall internal forces (STAAD – III - revision 23.0).
3) A two dimensional analysis was performed during ITR in order to analyze the STAAD analysis results (RISA – 3D – revision 23.0)

Pile Foundation Design
The primary computer analysis tools:

1) Spreadsheets of the various weights of the structure (Microsoft Excel).

A Rigid Base Foundation Analysis was performed using CPGA (X0080) – Case Pile Group Analysis Program

6.5 Construction Layout and Sequence

The construction procedure for the FIP modules was developed by URS, for CEMVN under Task Order No. 1, Contract No. DACW29-02-D-0008, Design Documentation Report Phase 1 Design.

Graving Site Construction
A graving site will be used to construct the lock module base section. The proposed site is located in New Orleans East, approximately six miles from the existing lock, where the Paris Road Bridge (Interstate 510/Louisiana Highway 47) crosses the MRGO. The voided base structure will have 28 feet of draft; 3 feet additional draft is provided to assure lift off. The MRGO channel bottom from the graving site to the staging area is elev. -31 feet NGVD (or deeper), which is sufficient draft for transporting all modules. The graving site and details are shown on Exhibit No. FIP-

4. Note: The graving site furnished is not mandatory. Alternately, the Contractor may select a different graving site; however, each module requires a minimum draft of 26 feet and is designed for inland waterway wave forces only.

The site must be cleared of brush and small trees. A small drainage canal (five feet deep by 15-feet wide) must be rerouted around the proposed graving site.

Initially the closure system for the graving site consists of 30 foot sheet pile cells lining the riverside of the graving site with a natural ground earthen berm between the reaches of the sheet pile cells with a cutoff wall driven thru the earthen berm. The earthen berm has to be removed to elev. -31 NGVD in order to float the first module out. Once the earthen berm (and cutoff wall) is removed then a 30' wide cellular sheet pile diaphragm wall with a stone berm is constructed and removed 4 times for the remaining modules. The initial excavation will be done in the wet, using land-based equipment. Of the 664,000 cubic yards of material excavated, about 112,000 cubic yards will be used to construct the hurricane protection and tie-in levees leaving the volume of excess material to be stockpiled at 552,000 cubic yards. The excess material will be stockpiled adjacent (east of the graving site) to restore the site.

The excavated area will be dewatered using wells and/or a wellpoint dewatering system. The dewatering system will remain in place for a four to five year period. Piezometers will be installed to assure that the water level is maintained at five feet below the work surface. The foundation for the graving site consists of a series of concrete grade beams supported by 14" x 14" square precast concrete pile approximately 40' long. In between grade beams is 14" of compacted gravel base. After the project is completed, the graving site will be backfilled to original grade.

North Bypass Channel Construction

Prior to dredging for the lock foundation, the north bypass channel must be opened. The north bypass channel is for two-way traffic, and is composed of a transit bypass channel and a laying bypass channel. Three 78-foot diameter protection cells will be constructed at the south end of the bypass channel, concurrent with bypass excavation. The channel corner riprap protection will be placed. Prior to opening the bypass, the 1510 linear foot timber guidewall will be installed. The guidewall supports will be 12-inch diameter treated timber piles with 12-inch by 12-inch treated timber fenders.

Tug assistance vessel contracts will be set up to begin when the north bypass channel is opened to navigation. Tug assistance vessels (push boats) will be stationed at each end of the bypass to assist tows through the bypass channel. Two push boats will be required (24 hours per day and 7 days a week) at each end for the duration of lock construction.

Lock Foundation Construction

Once the bypass channel is opened, lock foundation excavation will commence in two phases. Initially the footprint of the lock will be excavated to El -54 for the gatebay modules and El. -52 for the chamber modules. Sheet piles are then driven around the footprint of the lock, 3' offset from the perimeter of the lock structure, to provide the tremie containment wall. The remaining 4.5 feet of material is then excavated inside of the containment wall. All excavation will be by dredging, with a base dredge tolerance of plus or minus 6 inches. It is anticipated that a minimal amount of slope dressing will be required after dredging. The 3.0' thick stone base will be placed prior to pile installation. The three-foot thick stone base will be placed by lowering a hopper box to the bottom and opening a bottom chute. Guide cables and spud piles must be installed to guide a work barge which lowers the hopper. Hoppers are approximately 20-feet by 20-feet. In lieu of the hopper box, a stone tremie tube positioned by a submerged frame may be used.

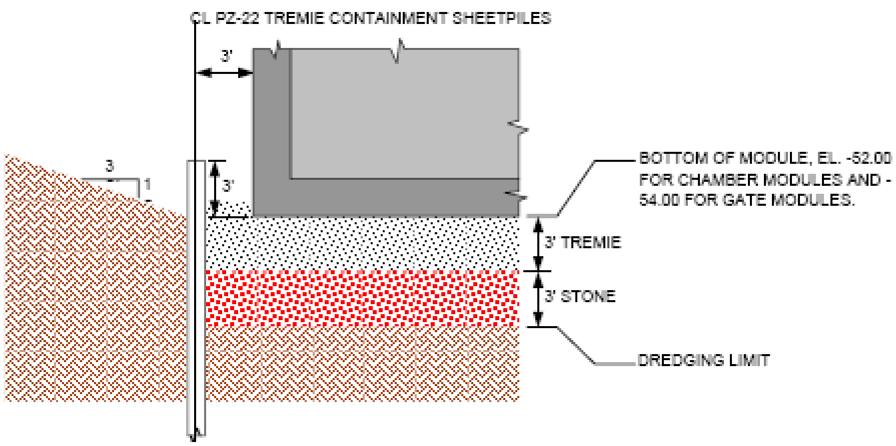


Figure 6.11 Foundation Preparation

Prior to pile driving, the Contractor will complete the eight 78-foot diameter protection cells, located at both ends of the excavation. The lock piles, 48-inch diameter steel pipe piles 120' long, will be continuously installed in two steps. Above the water surface, a vibratory hammer will be used. Below the water surface, a hydro-hammer will be used to bring the pile to grade. The landing piles will be driven to a tolerance of minus one-inch; all other piles will be driven within a tolerance of plus or minus six inches. Flat jacks will be installed by divers; after leveling, the pads will be grouted into place.

The cutoff sheet piling will be driven to a tolerance of plus or minus six inches with the use of a vibratory hammer. The Contractor will install cutoff piling in advance of the setting pads to avoid disturbance.

Two protection cells at the north end of the lock shall be removed to permit entrance of the float-in base sections. After the cell is removed, a 220' corridor is available for module passage. The removable cell must be pulled and redriven each time a new module enters the lock area.

The Contractor will construct a platform on top of the 3-78 foot diameter protection cells for a batch plant and stockpile area. The batch plant must be capable of producing at least 125 cubic yards of concrete per hour. See Figure 6.12 below.

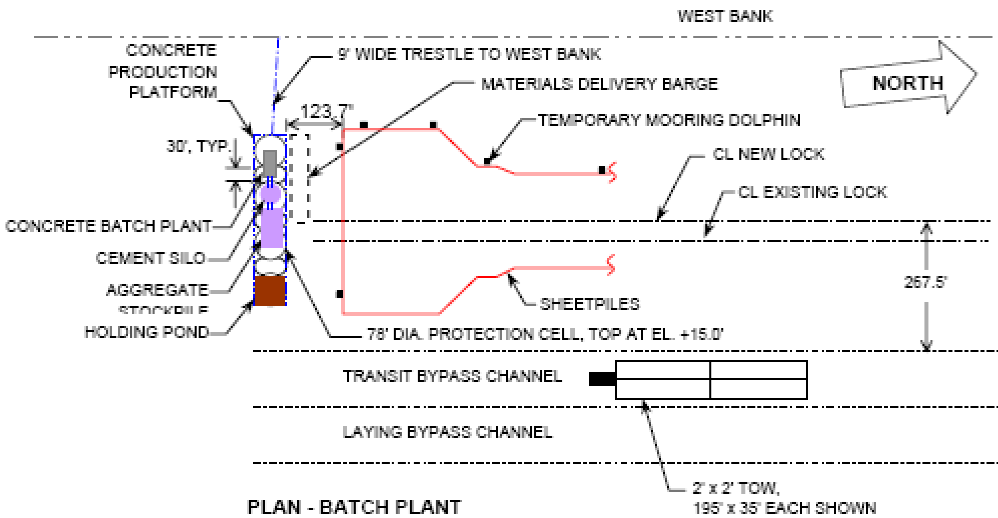


Figure 6.12 South Plan of Lock

Construct Lock Module Base Section

The south (riverside) gatebay module (GB-1) must be constructed first. The entire concrete base section will be constructed from 5,000 psi minimum compressive strength at 28 days concrete. A batch plant or plants will be erected at the site, and have a minimum production rate of 150 cubic yards per hour. Ample right of way exists for batch plant and material stockpiles. Modules CM-2 (chamber riverside), CM-3 (chamber lakeside) and GB-4 (gatebay lakeside) will be constructed in that order.

The embedded metals required for cutoff piling and module joints and waterstops will be positioned during forming. The base section culvert walls will be constructed with slip forms. The main steel reinforcement details are shown on Exhibit No. FIP-10.

Concurrent with lock base module construction, the permanent maintenance bulkheads and temporary transport bulkheads will be fabricated. The permanent maintenance bulkheads can be used for the first module constructed. The maintenance bulkheads will be installed just prior to flooding. Four maintenance bulkheads, each 5 feet high, are required during transport on each end. During set down, nine bulkheads are required at each end. The culvert openings will be sealed with steel bulkheads. The nine bulkheads will always be needed to maintain a dewatered module(s) therefore, temporary transport bulkheads are required. The temporary transport bulkheads consist of a series of vertical support frames spaced across the lock chamber anchored to the floor of the lock. Stiffened plate panels span between the vertical support frames provide the closure. This system is similar to a poiree dam.
Tension struts are needed to counteract the moment induced by the heavy lock walls during transport. W14 struts are attached to the lock face of wall just above the culvert and extend diagonally to the chamber floor. In addition, a horizontal WT4

strut is placed in the chamber wall approximately 8 feet above the culvert and a skin plate is placed on the sloping back chamber wall which extends to elevation 6.0.

Transport Lock Base Modules

Prior to transport of each module, the graving site will be control-flooded and the closure system removed. Closure materials will be stockpiled nearby and reinstalled once the module has been towed out. The graving site will be dewatered again and prepared for the next module.

Tug boats will be needed to pull the module along the six mile route to the lock area. The MRGO will be closed to marine traffic during the one-day haul. To complete transport, each module will be moored to temporary mooring dolphins at the lock site.

Lock Module Installation

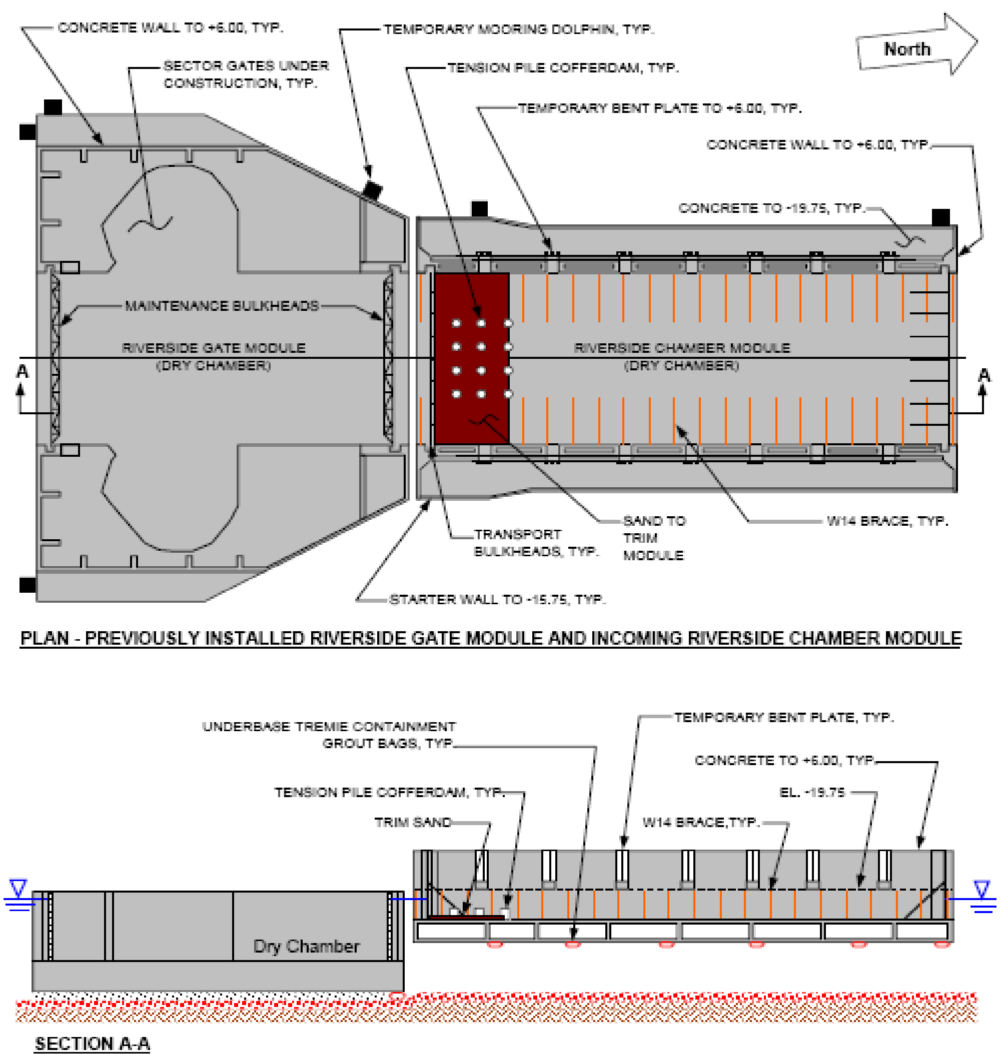


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Figure 6.13 Set Down of Unit

The lock module installation described below is typical.

The module is towed to the new lock site and moored to temporary dolphins or a previously installed module. Layout the underbase tremie containment grout bags. Methods include wrapping the bags around the module while floating at the lock site (recommended) or having divers lay the grout bags out on the canal floor.

Place 4 feet of concrete in-the-dry on the slabs above the culverts to act as starter walls on the perimeter of the lock and concrete walls to elevation +6.00 along the lock chamber act as forms for the concrete. Work from the center of the module towards the ends.

Erect tremie work platform in-the-dry above the culverts. This step may only occur after the concrete placed above the culverts has gained at least 1,000 psi compressive strength. It is assumed that the contractor will elect to install the platform before the concrete above the culverts is submerged. The tremie platform may run the full length of the module or be 60 feet wide and moved with each tremie placement (recommended).

Concurrent with erection of the work platform above the culverts, place sleeves above the culverts. The sleeves extend from elevation -15.75 to +6.00 and allow for placement of underbase tremie concrete and concrete placed in-the-dry in the outer voids adjacent to the culverts.

Place self-leveling concrete in-the-dry in voids below the culverts and lock floor. Voids directly below the culverts must be filled with structural concrete.
A filling sequence that minimizes increases in shears and moments (over the floating without infill case) and limits the depth of concrete placed to ~ 5 feet (for thermal considerations) will be utilized.

Place self-leveling, structural concrete in-the-dry in voids adjacent to culverts. It is expected that during this operation the module will come within 6" of its final location. When this occurs concrete placement must stop and the sand ballasting must start. Placement of concrete in the voids adjacent to the culverts may continue after the underbase tremie has been placed and gained strength.

Place 4-foot diameter cofferdam pipes over tension pile locations in the lock chamber. The pipes allow construction workers to grout rods in the tension piles while the lock chamber is full of sand ballast with or without water ballast. Note that this step may be done within the graving site provided that the maximum draft allowed is not exceeded. However, it is assumed that filling voids with concrete will be a simpler operation without these pipes present.

Place pedestals for the tremie work platform in the lock chamber. Pedestals allow the tremie work platform to be erected when the lock chamber is full of ballast sand and possible ballast water. Pedestals must not interfere with tension pile cofferdams, tremie sleeves, struts, and transport bulkheads.

Add sand ballast to the lock chamber until the module is floating at approximately at its final vertical and horizontal position. Any set down ballast is acceptable provided that: a) the ballast remains where it is placed, and b) the ballast is confined to the lock chamber but not the culverts. It will take approximately 1760 kips or 6 inches of 110 pcf moist sand over the entire lock chamber between the transport bulkheads to increase the module draft by 3 inches. Adjustments to pitch and roll may be made by any or all of the following methods: adding/removing ballast sand, shifting ballast sand, wetting ballast sand. With the module floating at its final location the gap between the flat jacks (pre-inflated 1/2" and manifolded into 3 groups) should be 0". Because they are pre-inflated and manifolded some jacks will extend and others will compress to account for the landing pile tolerance.

Lock off the flat jacks then add additional ballast (sand or water) to the lock chamber to achieve the required floatation factor of safety. Any flotation factor of safety ballast is acceptable provided that: a) the ballast is located in the lock chamber or the chamber/culverts, and b) the ballast is equally distributed. Ballast may not be added to the voids adjacent to the culverts or the voids between the upper walls to minimize residual stresses in the concrete. A 5% flotation factor of safety assuming the IHNC is at elevation +1.50 is required. This corresponds to 8400 kips of ballast or 2.2 feet of 110 pcf moist sand over the entire lock chamber between the transport bulkheads. This sand would be in addition to the sand used to trim and ballast the module to setdown.

If required, use the flat jacks to adjust the final elevation of module then lock off the jacks. Note that the flat jacks are manifolded into three groups to allow for a three-point adjustment of the module.

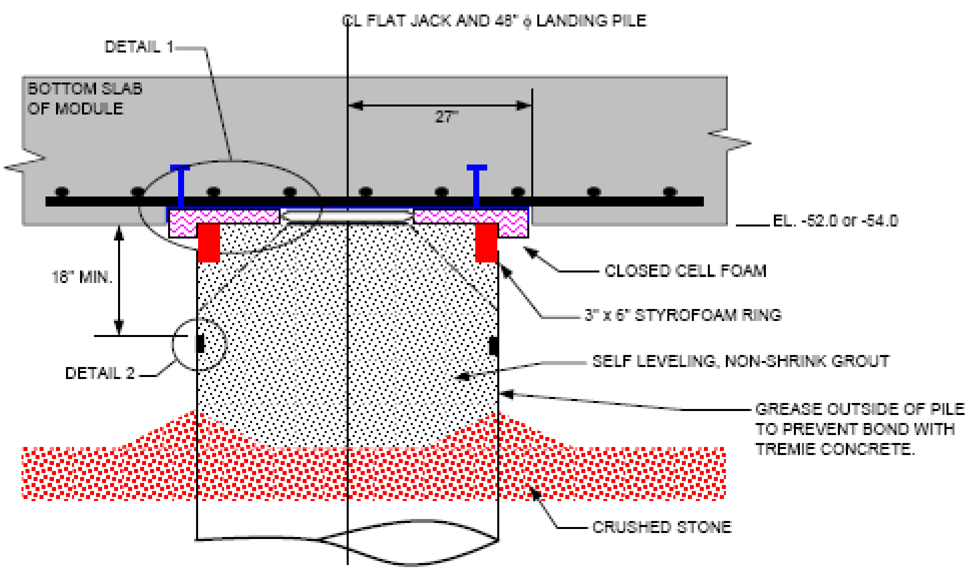


Figure 6.14 Landing Pile Detail

Screw the connector rods and grout rods into the tension piles and grout them into place. Each tension pile has one connector rod and one grout rod. Tension piles cannot be located under the maintenance or transport bulkheads.

Concurrent with the tension pile connection, infill underbase tremie containment grout bags and exterior seal grout bags, erect the tremie work platform on the pedestals previously placed in the lock chamber and drive the cutoff sheet pile that threads into the sheet embedded in the shell.

Start placement of underbase tremie concrete after grout bags gain at least 1,000 psi compressive strength and tension rods have been grouted in the tension piles. The grout bags divide the space to be tremied into five volumes that can each be filled within 24 hours (assuming 50 cy/hr). This division also limits the uplift pressure the tremie concrete exerts on the module to a relatively small area at any given time. It is expected that the pressure from the tremie concrete will relieve some of the load on the landing piles. It is expected that all tremie operations will be complete within 7 days of set down on the landing piles.

After all tremie concrete under the entire module has gained at least 1,000 psi compressive strength release the load, if any, from the flat jacks. Load will be transferred to the bearing and tension piles via the tremie concrete. Landing piles will not carry any load at this time since the landing piles have foam around the flat jacks to prevent tremie concrete intrusion between the top of the landing pile and the bottom of the module.

Transfuse the landing pile flat jacks with grout so their loads match that of adjacent bearing and tension piles (which should have essentially no load). Next, lock the jacks off until the grout gains at least 2,500 psi compressive strength. The flat jacks should not be manifolded for this operation.

If required, prestress the tension pile connector bars. Note that the current tension pile connection detail doesn't allow the bars to be post-tensioned. To allow this, the detail must be modified as follows: a) Replace the tension bar with an equivalent bar that is threaded at the ends only, and b) replace the nut welded to the bearing plate with a seal plate containing an annular wedge seal.

Fill tension pile tubes with concrete. This step must be done regardless of whether or not the tension pile rods are post-tensioned. If the bars are not post-tensioned the tubes may be filled immediately after grouting the tension bars in the tension piles.

Fill the upper wall voids with concrete placed in-the-dry. Upper chamber wall struts may be removed after concrete placed in the void (to within 1 foot of the strut) has gained strength or they may be cast in the voids. Remove the temporary braced skin plates. Note that the upper wall voids were not filled while floating to minimize transverse shears and moments. Likewise, they were not filled to achieve the flotation factor of safety to minimize the residual stresses in the module at the center of lock chamber. Similarly, they were not filled between set down on the landing piles and placement of tremie concrete in order to minimize the number of days that the module is supported only by landing piles.

Concurrent with placing concrete in the upper walls perform the following:
a) remove tremie platforms,
b) remove tension pile cofferdams,
c) remove struts, and
d) remove trim, setdown, and flotation FS ballast.

Flood the module lock chamber to match the IHNC elevation. If the adjacent module is dry at this time, it must also be flooded.

Remove the north end and then the south end transport bulkheads from the module. The transport bulkheads will be taken to the graving site for use with the next module.

Remove the north end maintenance bulkheads from the adjacent module and place them at the north end of the installed module. Use of maintenance and transport bulkheads (instead of maintenance bulkheads only) allows work on the RGM sector gates to take place independent of work on the installed module once it is at the lock site.

Install temporary screw jacks at the module joint at elevation +4.00. Jacks are required to minimize the stresses on the 21-foot cantilevered wall at the south end of the installed module. The wall will subjected to a 15-foot water head once the adjacent module and the installed module are dewatered.

Dewater the adjacent module and the installed module. If the previously infilled exterior seal grout bags are not sealing properly, the space between those two grout bags may be filled with grout to form a third exterior seal.

While the module chambers are dry complete the following:
a) remove the culvert closures from the north end of the adjacent module and the south end of the installed module, and
b) complete the joint between the modules.

Culvert closures are assumed to be designed to be removed by pulling the plate section up to the lock chamber through the 8" joint between the modules when the chamber is dewatered while struts are removed through the ports. The culvert closures may be detached from the shell by construction workers working in-the-dry within the culverts. The removed culvert closures will be taken to the graving site for use on the next module.

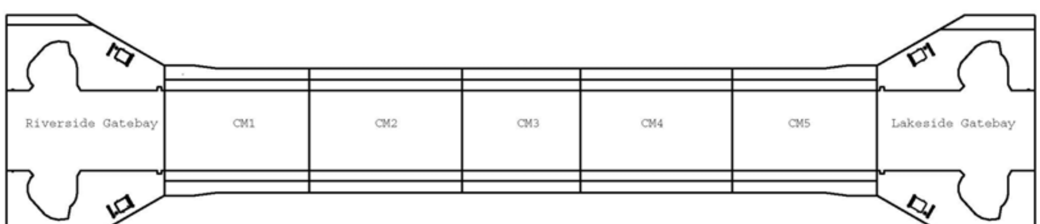
Repeat the above steps for additional modules placed.

Complete module mechanical and electrical installation.

Open Lock as Pass-Through Lock
Test all machinery and flood lock chamber.
Remove the channel protection cells at both ends. Place channel riprap at lock ends.
Close bypass channel and open new lock to marine traffic. (Water stage still controlled by old lock).
Remove the bypass timber fender. The three south end bypass channel protection cells and riprap will remain.
Construct the east side guidewalls at both ends. All work will be done in the wet by barge mounted equipment located behind the traffic channel. End and intermediate piers for the south end floating guidewall will be constructed from within a braced excavation.

Backfill Structure and Levee Tie-Ins
Construct the tie-in levees at both ends. The sand backfill must be barged in and deposited with a clam shell. Fill will be brought up to El. 5.0. The sand backfill will then be placed along the lock wall (fill will be placed uniformly on both sides).
The remaining lock backfill will be dredged material. Sufficient dredging operations required south of the new lock, including the bypass channel at the existing lock, will be delayed so that disposed material is used as lock fill.
The tie-in levee clay crown and I-Wall will be constructed. The I-wall will be overbuilt 6 inches to account for future settlement. Once the tie-in levees are complete the new lock will be operated to control water stage. The old lock will now be demolished.
Complete site work.

7 Cast-In-Place Design and Construction, Considerations and Criteria



7.1 Summary of Cast-in-Place

The replacement lock will be located north of the existing lock. The structural design will be in accordance with COE guidance and applicable industry standards. The structure will utilize standard U-frame construction techniques, including sheet pile cofferdams, dewatering system and cast-in-place concrete. The lock design consists of a cast in place pile founded concrete lock. Because of the amount of calculated differential settlement the option of soil founding the concrete structure was eliminated. The top of the replacement lock wall is elevation 23 feet NGVD. The lock chamber measures 1287.66 feet C-C of the pintles, and 110 feet in width. The lock has sector gates as shown on exhibit COM-9. The filling and emptying system uses a vertical operated roller gate located in the gate bay monolith culverts as shown on exhibits COM-7 and COM-8. The lock culvert is 15 feet wide by 18.25 feet high for the 1200-foot lock. The maintenance bulkheads are as shown on exhibit COM-12. The replacement lock has a sill elevation (-) 40 feet NGVD. The filling and emptying system consists an interior, ported culvert and manifold system as shown on exhibit CIP-3 and CIP-4. The lock structure is pile-founded.

7.2 Geotechnical

Design Documentation Report No. 3, Lock Foundation Report, was prepared by MVN and approved in May 2002. This report contains the results of the subsurface investigations, laboratory testing, bank stability analysis, and pile testing that was performed in support of the IHNC Lock Design, and the results of these geotechnical investigations were used as the Float-In-Place vs. Cast-In-Place letter report.

In general terms, soil conditions at the project site consist of natural levee deposits underlain by marsh and intradelta deposits. The marsh and intradelta deposits extend to El. -32.0, and consist of very soft medium clays with silt lenses. Interdistributary deposits underlain by prodelta deposits, consisting of very soft to stiff clays, silt lenses, and sand layers, are found between El -32.0 and El. -70.0. Below El. -70 are Pleistocene deposits of stiff clays, silts, and sands. Subsurface soil profiles and corresponding shear strength parameters as presented in CEMVN Design Documentation Report No. 3, Plates 20 - 23, and 28, were utilized to perform the QA check of the CEMVN slope stability analysis.

7.2.1 48” Pile Driving Investigation

The layout for the Cast-In-Place Lock pile foundation is shown on Exhibit’s CIP-1 and CIP-2. Foundation utilizes 24” Pre-cast Concrete piles. No pile tests were performed on the 24” pre-cast concrete piles. CPGA was used to determine the arrangement and length of the pile foundation for the sector gate bay monolith. Loadings were input into the CPGA programs using applicable overstress values. The pile length was determined from pile capacity curves supplied by CEMVN-ED-F, using a F.S. of 2.0. Deflections were determined based on an Es value reduced for group effects. The group effect reductions were taken according to the values shown on the pile capacity curve supplied by CEMVN-ED-F.

Piles in CWFRAM were treated as elastic elements that develop resistance proportional to the displacements at the pile head/structure base point of connection. Piles in SAP2000 were modeled as springs. The spring values were provided by CEMVN-ED-F corresponding to the maximum loads generated through a rigid pile foundation design using CPGA. Unfactored spring values were used. The maximum reaction determined through the computer programs was used to determine the pile length from pile capacity curves supplied by CEMVN-ED-F, using a F.S. of 2.0.

The models used in analyzing the float-in-place and the cast-in-place methods utilized finite element programs. Although the final product is very similar there are some loading conditions specific to the float-in-place which cause concerns about initial stresses. The float-in-place construction and placement techniques will likely result in initial stresses being locked into the structure as noted by Dr. Saad Moustafa. One possible effect will be that the piles see the loads differently during operation due to the structure being under some loading when the float-in-place monoliths are set. The cast-in-place construction will not have any initial stresses when casting the flexible base with the concrete piles.

The various monoliths for the cast-in-place method that were analyzed were investigated using a finite element program. The monoliths were cut into sections and analyzed as beams with the loads applied as distributed loads, concentrated loads, and concentrated moments. Piles were modeled as spring constants supporting the flexible base. These modeling techniques and assumptions are appropriate with the structure being analyzed. LRH consulted with Regional Technical Specialist Andy Harkness P.E. of CELRP who confirmed that the modeling assumptions were appropriate for the pile founded flexible base.

7.2.2 Cofferdam Issues

Preliminary design of the CIP cofferdam was performed by URS Group under Task Order 2 of Contract No. DACW29-02-D0008. The report, entitled Cast-In-Place Cofferdam, 100% Submittal, Feasibility Level Design, was approved in October 2006 following ITR certification.

The general alignment, lateral limits, and top height, were determined by New Orleans District in previous studies. The design effort for this comparative analysis required both structural and global stability analysis of the cofferdam and interior berms, in addition to identifying the risks associated with constructing cofferdams on soft soils.

The insitu soils within the canal are fine grained and lack sufficient strength to support granular filled, cellular cofferdam when analyzed for bearing capacity and sliding stability. Alternatives such as supporting the cofferdam on piles or improving the foundation materials using soil mixing or jet grouting techniques were evaluated.

As per the URS design analysis, a jet-grouted foundation below the cofferdam was found to be the most economical stability solution, and was selected for the preliminary design analysis.

The cofferdam consists of 45 cells and 44 arcs at 67.3 feet in diameter, and extend from top El. (+)5 to pile tip El. (-)90. The pile tips are embedded 10 feet into the soilcrete foundation, which extends from El. (-) 80 to the jet grouting vertical limit of El. (-) 125. The zone of jet-grouting is 40 feet deep and extends 20 feet beyond the sheet pile diameter on both the east and west side of the canal. The minimum required strength of the improved soilcrete material was found to be 3500 psf. Inquiries to local contractors in New Orleans found that these strengths are routinely achieved at other projects within the region.

7.2.3 Dewatering and Groundwater Control

Preliminary design of the CIP dewatering requirements were performed by URS Group under Task Order 2 of Contract No. DACW29-02-D0008. The report, entitled Cast-In-Place Cofferdam, 100% Submittal, Feasibility Level Design, was approved in October 2006 following ITR certification.

There are three aquifers - one at -58 to -60, that will be cut off by the cofferdam. There is buried beach sand on the north, but not on the south. The second aquifer is a deep aquifer, -100 to -130, that will need to be dewatered for heave. The third aquifer is a mid aquifer at about -80, but needs to be defined. There is also concern about the radius of influence and settlement if the pumping rates are high.

The groundwater control plan calls for a sheet pile cut-off wall to be installed near the top of the excavated slope, parallel with the existing floodwall, to cut-off seepage from the east bank into the excavation from the upper sand layer at El. (-)60. The jet-grouting work proposed for stabilization of the cofferdam foundation extends thru the sand layer on the west side of the excavation, and will cut-off seepage pathways from the canal. Based on the preliminary design, all cut-offs must extend to El. (-) 75 or deeper. It is anticipated that the sheet pile wall at the top of bank will be tied into the cofferdam and the jet-grouting zones to provide a continuous seepage cut-off around the entire perimeter of the cofferdam.

As per the URS design, the inboard rock fill berms would extend down to the sand stratum and that sumps, wells, or well points would be constructed at the toe of this berm to relieve uplift pressure and lower the groundwater level in the sand stratum. The dewatering specification would require the contractor to lower the groundwater at least 4 ft below planned subgrade (bottom of excavation) in advance of excavation. Unwatering the cofferdam would be permitted without lowering the groundwater level in the El. -60.0 sand strata, since the seepage would be cut-off on all four sides. The drawdown requirement (to 4 ft below subgrade) would become effective after unwatering is complete and before construction of the lock floor and walls.

A performance type dewatering specification will allow the contractor to select the means and methods for the dewatering system. Alternatives include installing a sump and pump system around the perimeter of the excavation, or pre-draining the sand stratum using wells and/or well points. The specification would also require installation of several piezometers to measure the performance of the contractor's system, and assure that the uplift pressures and piezometric levels are within the range of the design assumptions for cofferdam stability.
According to the URS design, it will also be necessary to relieve the pressure in the sand stratum at El. -130.0. The pressure relief wells would be installed on approximately 200 foot centers (about 26 wells) by drilling from the top of the cofferdam and along the top of the landside slope down to about El. -140.0, the

bottom of the next continuous sand layer below the one at El. -60.0 around the perimeter of the excavation.

If the pressure in the sand stratum at about El. -130.0 is not relieved, the factor of safety against heave for a groundwater level at El. 5.0 was estimated to be 1.01, which is inadequate. Therefore, the specifications would require piezometers to measure the performance of the pressure relief well system, which would be specified as a minimum system.

It is anticipated that the total volume of flow would be small from the pressure relief system (estimate less than 200 gpm). The wells would be sealed and pumped with jet eductors to induce vacuum within the casing to increase flow (if necessary) from the deep sand stratum. Alternatively, submersible pumps may be used in the wells and the vacuum in the well casings developed (if necessary) using a vacuum pump.

7.2.4 Slope Stability

Stability Analysis Criteria and Methodology. In accordance with EM 1110-2-1902, the minimum Factors of Safety criteria for the End of Construction (undrained) and Long-Term (Drained) Load Cases are as follows:

End of Construction	F.S. $_{min}$ = 1.30
Long Term	F.S. $_{min}$ = 1.50

For the purpose of this study, both the End of Construction (undrained) load case and the Long Term (drained) Load Cases were analyzed for stability. The need for a Sudden Drawdown Analysis should also be evaluated after the initial and emergency flood-out drawdown rates have been developed.

Stability analyses were performed using the Slope/W stability modeling code by GeoSlope International. Spencer's Methods was selected for the analysis because it satisfies both Force and Moment Equilibrium, and the side force assumptions are consistent with Corps of Engineers stability analysis procedures.

Railroad surcharge loadings used for the Dewatered Cofferdam Excavation in compliance with AREAM, as referenced in Section 6.3.3 for the Float-In-Place Alternative.

Subsurface profiles and material properties are the same as illustrated on Plate 28 of Design Documentation Report No. 3.

Site Conditions and Model Features. The excavation inside the cofferdam consists of a series of sloped grades and horizontal benches as shown on Exhibit No. CIP-16. As shown in Figure 7.1, the most critical stability section is located at stability cross-section A-A, where the slope geometry is constrained due to the excavation limits for the Gat Bay monolith, and there is a T-wall and an active railroad line and sidings at the top of the cut-slope. Section B-B represents a location along the reach where there is a single track load, and Section C-C represents the same slope geometry as section A-A, however there is no railroad surcharge loading.

Figure 7.1

Stability of Section A-A Prior to Unwatering.

The proposed CIP excavation was analyzed to identify potential stability problems if the dredging work is completed prior unwatering. Since the seepage cut-off walls proposed by URS in the cofferdam design will not be installed until after the dredging, the vertical cutslope at the toe was modeled as a temporary construction slope. Installation of the sheet pile wall would occur after the unwatering and prior to the final foundation grading. Due to concerns by MVN with respect to the foundation piling below the existing T-wall, this structural reinforcement feature was not included in the analysis. As shown in Figure 7.2, the minimum Factor of Safety with the railroad loading and prior to unwatering resulted in a Factor of Safety 1.190.

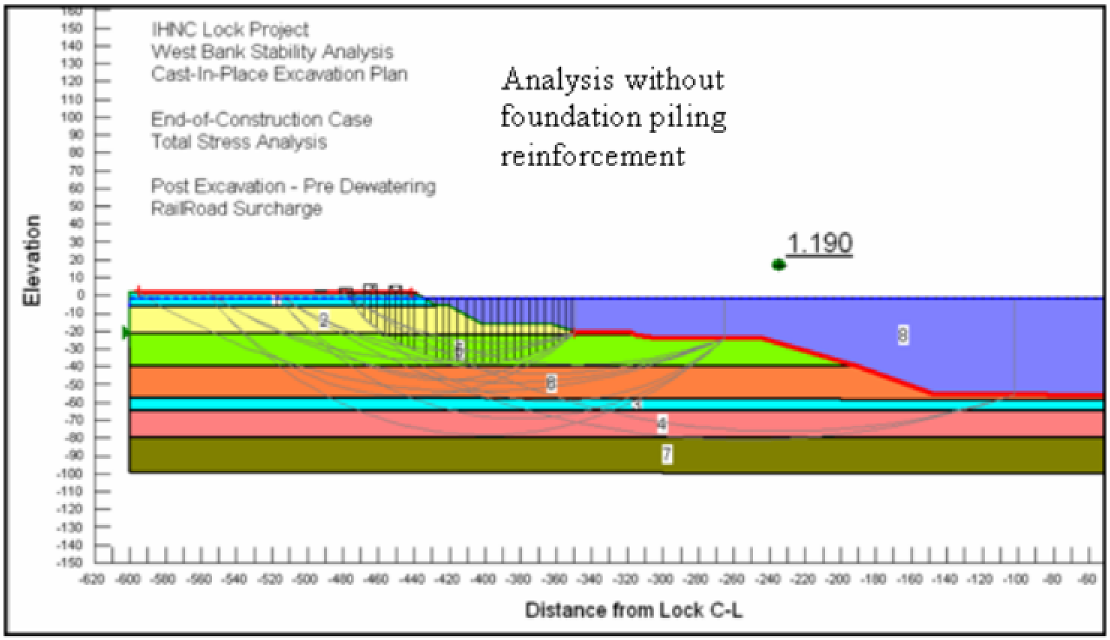


Figure 7.2

Due to the shallow failure plane that results at mid-slope from the railroad loading, a soil improvement zone was added to the model to strengthen the soils under the railroad and raise the Factor of Safety to meet criteria. Soil-Cement mixing is recommended for ground improvement below the rail lines based on the results of field testing performed by MVN, and the relatively shallow depths to which the improvement zone must extend. As shown in Figure 7.3, stability analysis using improved soil strengths of 400 psf resulted in a Factor of Safety of 1.399.

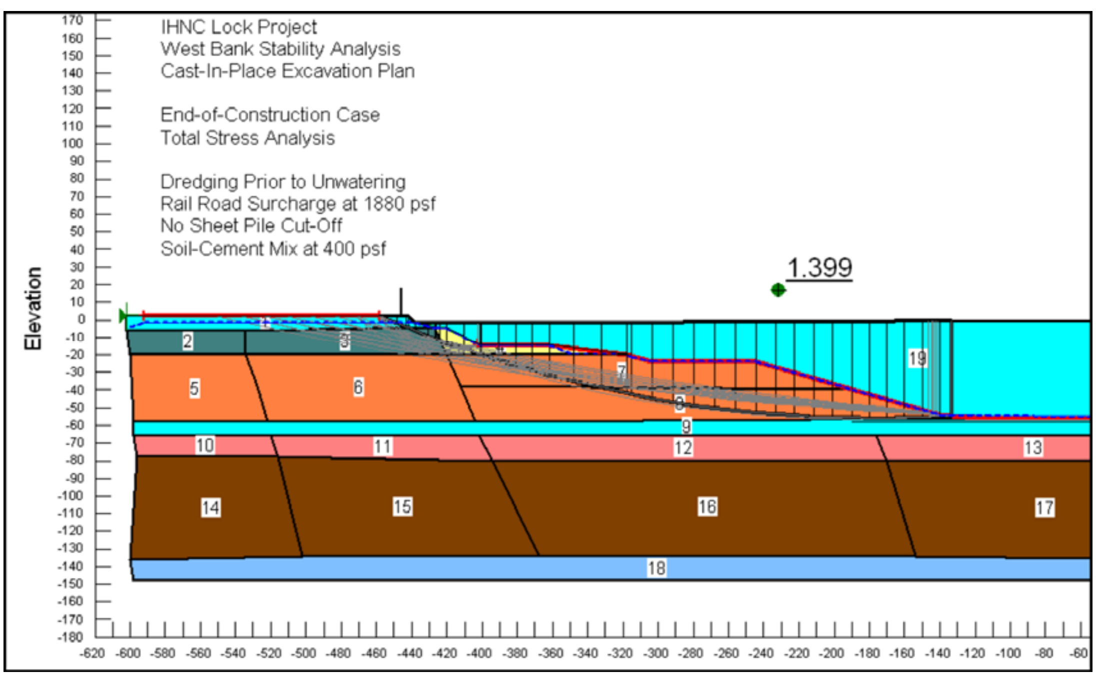


Figure 7.3

Stability of Section A-A After Unwatering.

The slope model was analyzed in a fully unwatered condition without consideration to any structural reinforcement that may be offered by the piling, and without consideration to the soil improvement zone shown in Figure 7.3. Surcharge Loads were imposed at the top of slope, and the unwatering was assumed to take place in a controlled manner to avoid a sudden drawdown condition. As shown in Figure 7.4, the analysis resulted in a Factor of Safety = 0.877.

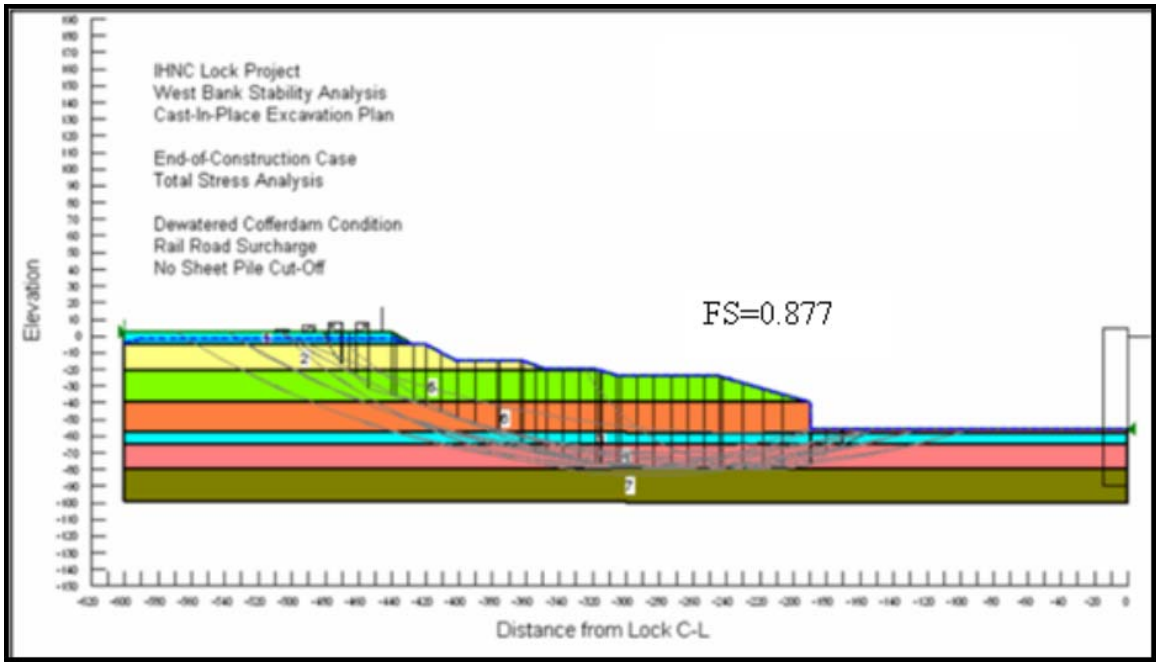


Figure 7.4

The options for improving slope stability become limited when the slope geometry and surcharge loadings are defined and fixed. For the purposes of this preliminary design effort, the jet grouting option was investigated because jet grouting is proposed for stabilization of the soils below the cofferdam, and there is opportunity and economy to stabilize the backslope using the same jet grouting contractor. Jet grouting is also recommended over soil-cement mixing in this area due to the higher depths, pressures, and soil strength requirements.

The soil zone within the neutral block of the potential sliding mass was modeled as a soil zone that was improved by jet grouting operations. Improved soil strengths were assumed to reach 3500 psf, which are consistent with the strengths that URS proposed in the cofferdam design, and are based on the results of past jet grouting projects in the New Orleans region.

Due to the need for jet grouting to improve the soils shear strength and the uncertainties associated with achieving a soil mass of uniform shear strength, the minimum Factor of Safety criteria was increased to FS min = 1.5 when failure planes pass through the treated soil zones. For failure planes that do not pass through the treated soil zones, the standard Corps or Engineers criteria as noted in the first paragraph of this section will apply. This criterion is consistent with what URS proposed for cofferdam stability on the jet grouted soil zone.

As shown in Figure 7.5, the most critical potential failure surface in the unwatered condition results in a FS = 1.55 and exceeds the required minimum for a soil mass that relies on improved soil strengths for stability. Although there may be opportunity to adjust the limits of the jet grouting in future studies to achieve cost saving, the higher minimum Factor of Safety selected for this load case is warranted at the preliminary design stage due to the uncertainty in the existing and improved foundation strengths, the difficulty in achieving uniform strength improvements within the jet grouting zone, and the critical nature of the floodwall that will become distressed if slope movements occur.

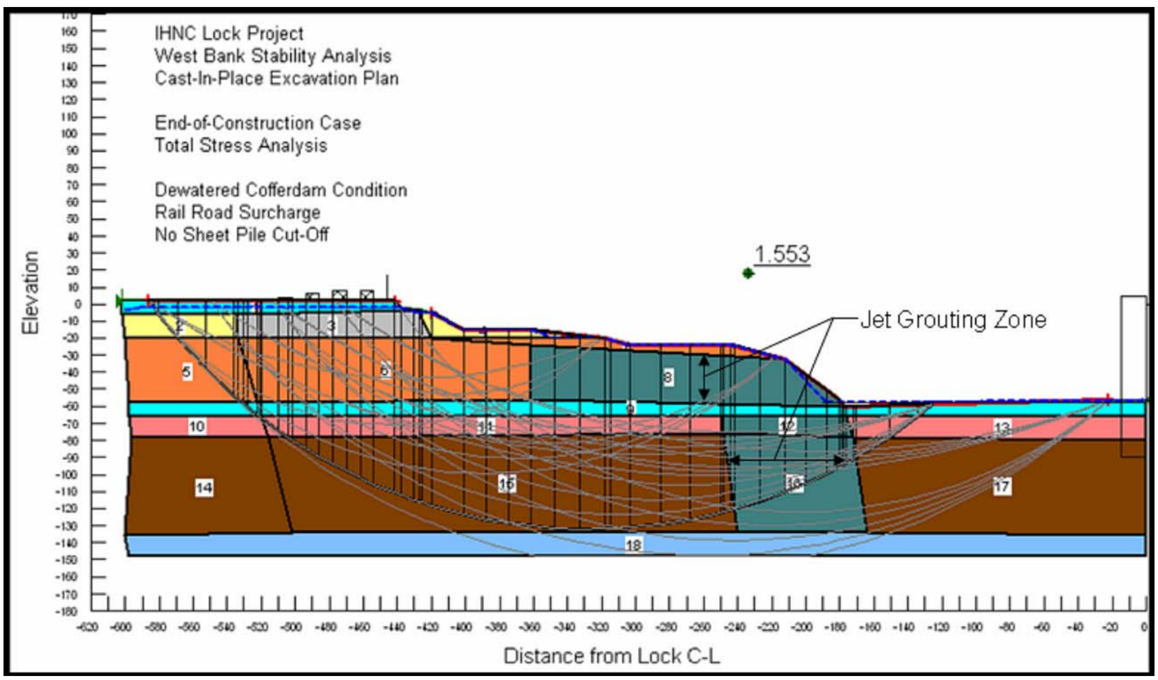


Figure 7.5

Long-Term Load Case: The long term, steady seepage load case was analyzed using the improved soil shear strengths in the jet grout zones, and drained shear strengths in non-jet grouted soil layers. The drained strengths used in the analysis are consistent with the drained strengths used for the long term stability analysis in the FIP alternative, and were taken from the URS stability analysis of the east bank navigation by-pass slope excavation. As shown in Figure 7.6, the stability analysis resulted in a FS=1.93, and exceeds the Corps of Engineers minimum requirement of FS=1.50. These results confirm that the long term load case is not the critical load case.

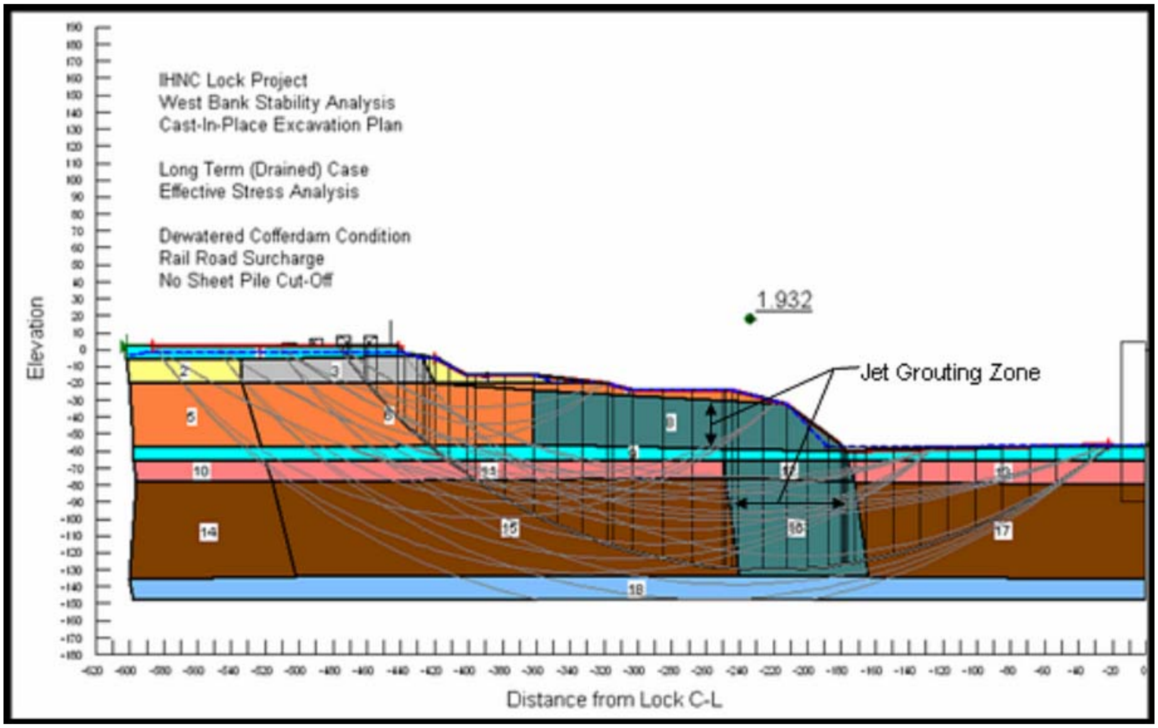


Figure 7.6

Stability of Section B-B After Unwatering

The load case with a single rail line surcharge located at the top of the slope at cross-section B-B was modeled for stability. As shown in Figure 7.7, the critical load case is a shallow, rotational type failure surface that is influences by the railroad, and results in a FS=1.33. Since this failure surface does not pass through the jet grouting zones, the standard Corps of Engineers criteria (FS min=1.3) applies. As shown in Figure 7.7, numerous potential failure surfaces were analyzed, however the deeper failure planes resulted in similar (higher) Factors of Safety, and are consistent with the analysis results at sections A-A and C-C. Because soil-cement mixing is not proposed along this reach (as is proposed below the railroad lines and sidings at Section A-A, the most critical failure surface is a relatively shallow failure plane that is directly influenced by the very soft soils in the upper soil profile, and the single rail line surcharge.

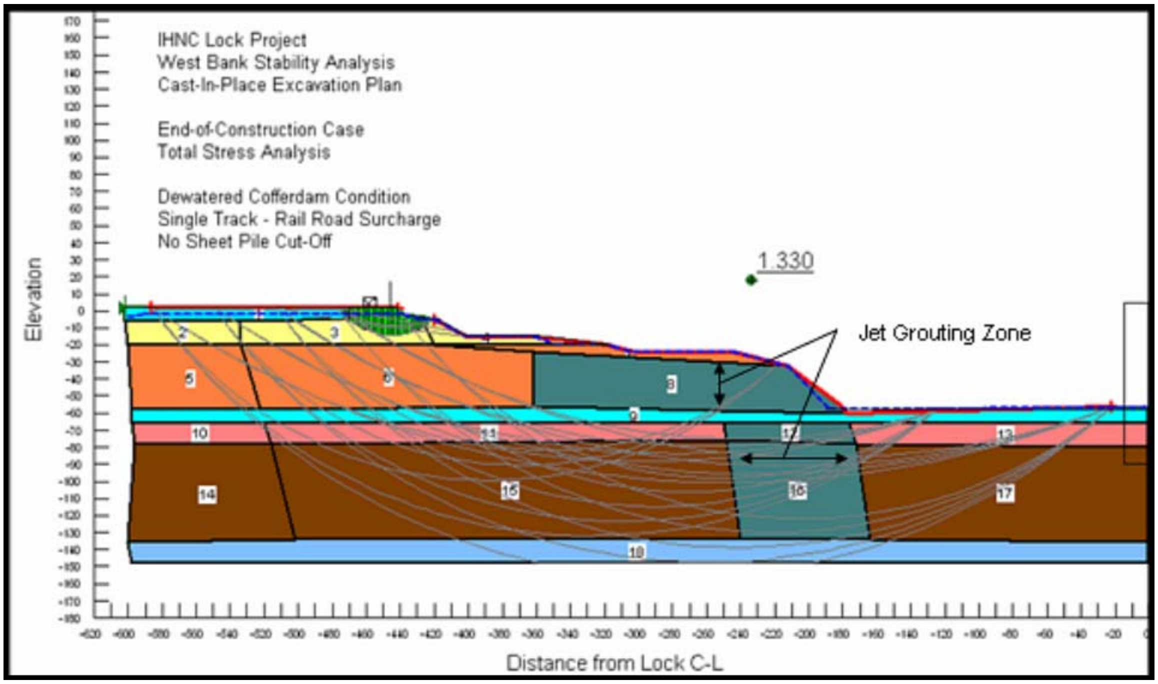


Figure 7.7

Stability of Section C-C After Unwatering

To define the lateral limits of the jet grouting requirements within the cofferdam, additional analyses were performed at Section C-C where there is no railroad surcharge on the slope. Numerous combinations of jet grouted soil zones were analyzed in an attempt to optimize the soil improvement limits; however the failure surface continued to outflank the improved soil zones. Figure 7.8 shows a typical failure surface with limited jet grouting in Soil Zone 12 where the failure surface dropped below the jet grouted zone. Figure 7.9 illustrates the standard soil improvement template used at the other stability sections, and confirms that this template must be used along the entire reach to meet the minimum Factor of Safety. The higher minimum Factor of Safety criteria for this load case is justified due to the uncertainties in the existing soil strengths and the difficulty in achieving uniform strength improvements within the jet grouting zone.

The results of the analysis at Section C-C clearly illustrate that the improved soil zone must extend along the entire reach of the proposed excavation within the cofferdam, and must penetrate the foundation soils to a depth of EL. -140.

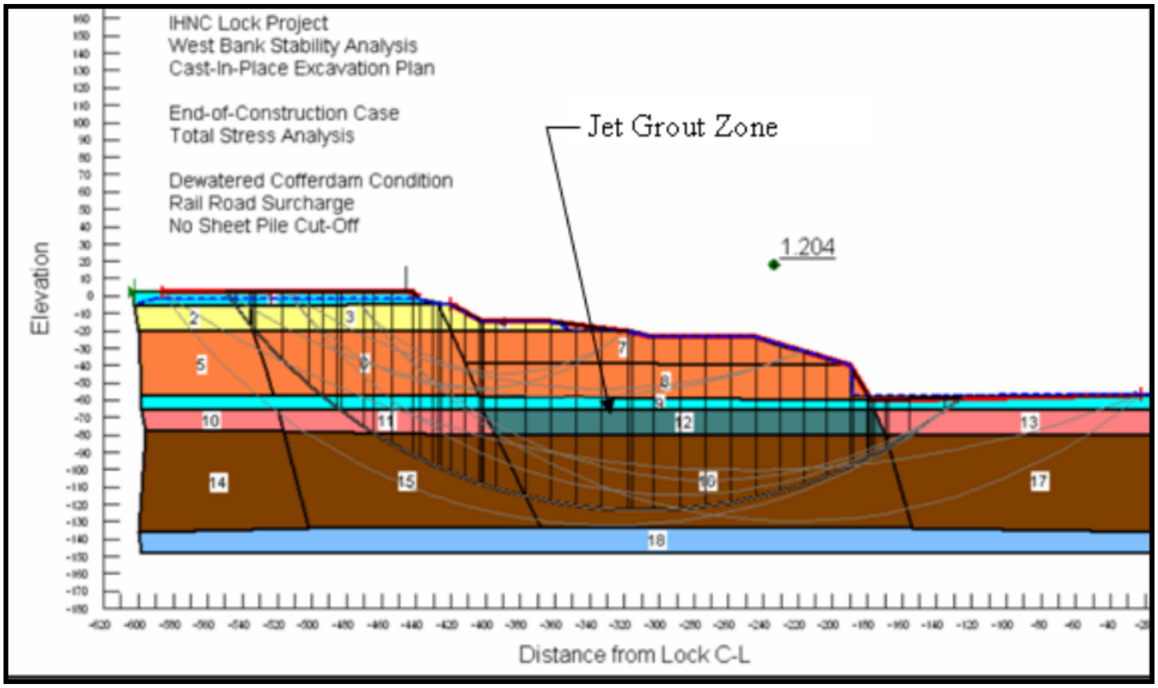


Figure 7.8

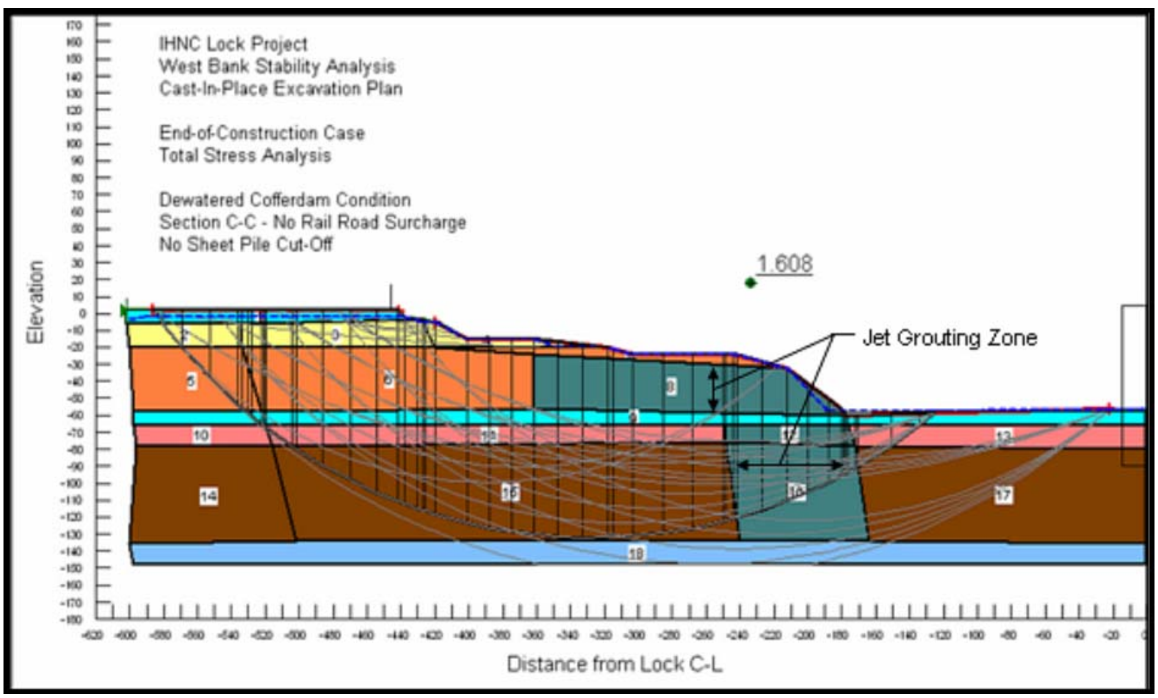


Figure 7.9

7.2.5 Cast-In-Place Lock Foundation Features

For the purposes of this study, the foundation plan for the Cast-In-Place Lock was taken from the Mississippi River - Gulf Outlet, New Lock and Connecting Channels Evaluation Report, dated March 1997. The Lock walls and lock chamber floor will be supported by a 24-inch square Pre-Cast, Pre-Stressed, Concrete (PCC) piles.

A total of 2,607 vertical piles and 808 battered piles will be driven to depths of 131 feet, for a total pile driving length of 447,356 linear feet. The spacing of the gate-bay monolith piles is 8-foot on center, while the spacing of the chamber monolith piles is 10-feet on center. As per EM 1110-2-2906, pile head tolerance will be +/- 3-inches on the horizontal and 1-inch vertical.

Since the 131 linear feet concrete piles will be difficult to cast, transport, and handle at the job site, it is anticipated that the contractor will utilize pile splices in an effort to optimize the pile lengths to meet his equipment capabilities and supplier capabilities.

The design documentation provided in the 1997 report and other internal correspondence made available for the QA check indicated that a preliminary design was performed on the piles using L-Pile from Ensoft. Results from the 48-inch pile load tests were compared to p-y curves generated from L-Pile to validate the estimated load response.

Based on the L-Pile results, the lateral deflection under the normal load cases ranged from 0.3 inches to 0.6 inches, with a maximum deflection under the extreme load case of 1.6 inches. These pile deflections appear to be within the tolerable range of movement for preliminary design, and validate the adequacy of the 24-inch PPC piles.

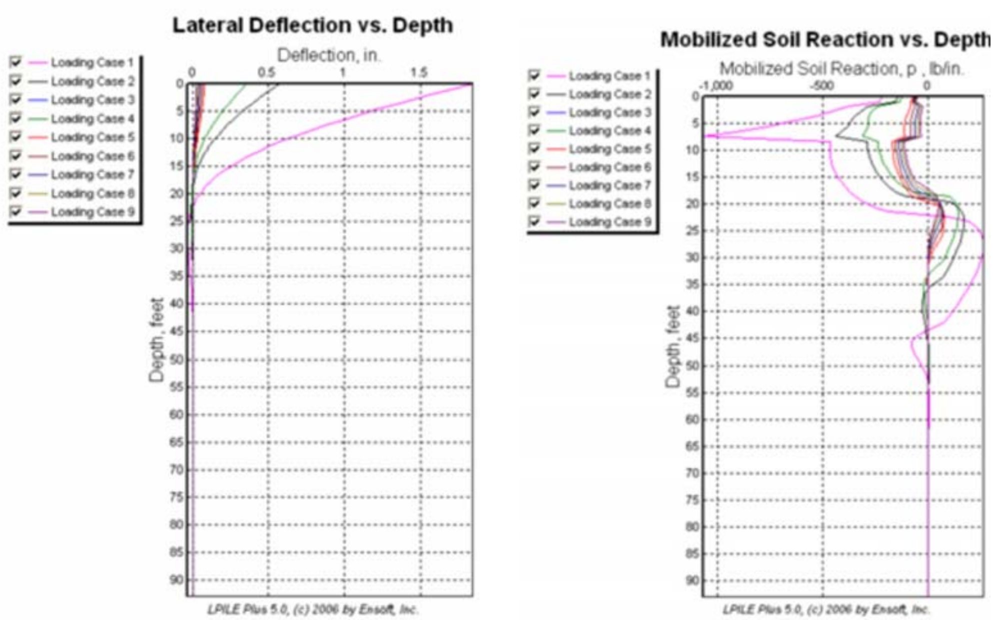


Figure 7.7

Final design of the pile foundation for the Cast-In-Place Lock should include additional pile load analysis using Ensoft's Group software code to optimize the pile diameters and depths, and a Pile Load Testing Program should be conducted to develop load response curves for the selected pile.

7.3. Structural

7.3.1 Structural Investigation Methodology

The LRD team has reviewed the preliminary study documents prepared by MVN for the Cast-In-Place (CIP) alternative. In regards to the CIP structural design, the team believes there is sufficient detail in the existing FIP documentation to utilize the same structural features and lock wall geometry for the CIP method. However, the internal reinforcement and mass concrete requirements of the CIP alternative will be significantly different than the structural panels and tremie infill proposed for the FIP. For the purpose of this study, the team made a general assumption that the 24" pre-stressed precast concrete piles shown in the design plans are adequate for the comparative analysis, and will not perform any additional rigid or flexible base analysis to optimize the current design. A minimal QA effort was undertaken to verify the adequacy of the pile sizes, depths, and spacing based on the lock features and details provided in the existing documentation. Designs of the lakeside and riverside gate bay monoliths and the chamber monoliths were performed by MVN and provided to the team in order to complete the CIP reinforcement quantities. The team investigated the provided analysis in order to understand the design and determine the extent of the design completion. While reviewing the CIP design calculations, the computations were organized and bookmarked electronically in order to aid in the review process and investigations were completed to provide a cursory check of the computations. The LRD team then used the provided information and calculated concrete and reinforcement quantities and prepared reinforcement drawings to be used in the cost comparison.

The analysis completed by MVN used the following load cases and methodology in the design process to calculate the necessary reinforcement for the critical load cases (dewatered chamber etc.) that dictated the design. The critical load cases selected for the analysis are as follows:

1. Dead Load
2. Maximum Operating Water with Gates Open
3. Maximum Operating Water with Gates Closed
4. Maximum Operating Water with Gates Open plus Freeboard
5. Maximum Operating Water with Gates Closed plus Freeboard
6. Normal Water with Gates Open
7. Normal Water with Gates Closed
8. Reverse Head Navigation Limits with Gates Open

9. Reverse Head Navigation Limits with Gates Closed
10. Usual Maintenance Dewatered
11. Unusual Maintenance Dewatered
12. Construction

The methodology used on the design of the Gate bay and Chamber monoliths consisted of the following techniques:

Gate Bay Monolith Design

The design consisted of four two dimensional strips in the transverse and longitudinal directions through critical areas of the monoliths. The longitudinal direction refers to the direction of flow through the structure while the transverse direction is perpendicular to the direction of flow. The strips were analyzed using the structural analysis program STAAD Pro 2004 using a 2D analysis. The slab was modeled as beam members with spring supports. The spring values were provided by MVN ED-F and correspond to the maximum loads generated through a rigid pile foundation design using CPGA. Unfactored spring values were used. The applicable tributary dead and live loads were applied to the beam members. In the transverse direction, a distributed load was applied to each strip to account for the loading effects in the longitudinal direction. The shear and moment output from STAAD Pro was factored by a single load factor of 1.7 and a hydraulic load factor of 1.3. The factored shear and moments were used to size and determine the adequacy of the slab and its reinforcement.

It should be noted that Chapter 3 of EM 1110-2-2104 has changed dated 20 Aug 03. The revised text states that "In particular, the shear reinforcement should be designed for the excess shear, the difference between the hydraulic factored ultimate shear force, V_{uh}, and the shear strength provided by the concrete, ΦV_c, where Φ is the concrete resistance factor for shear design." Also, the revised section requires that "For certain hydraulic structures such as U-frame locks and channels, the live load can have a relieving effect on the factored load combination used to determine the total factored load effects. In this case, the combination of factored dead and live loads with a live load factor of unity

$$Uh = H_f\,(1.4D + 1.0L)$$

should be investigated and reported in the design documents." These requirements should be incorporated in later designs.

The design of the Gate bay monolith walls was accomplished using "Moody diagrams" developed in a Bureau of Reclamation Engineering Monograph document titled "Moments and Reactions for Rectangular Plates" by W.T. Moody reprinted 1970. The vertical walls were modeled as individual plates with free or fixed sides as applicable. Tables of coefficients found in the book were used to determine moments and reactions in such structures with various loading conditions. CFRAME was also used to analyze the loading conditions and identify the maximum loads. The Moody Diagram and CFRAME results along with Excel (or MathCAD) were used to design the reinforcement. Concrete General Flexure Analysis (CGFAG) was utilized on the thrust block portion of the wall using Working Stress Design.

Chamber Monolith Design

The chamber monoliths were designed utilizing two programs, both of which employed two-dimensional analysis. CWFRAM, a two-dimensional analysis program for U-frame structures, was used as the primary design of the chamber monoliths. A two-dimensional SAP2000 model was used to validate the CWFRAM results. The maximum shear and moments output from the two computer programs was factored by a single load factor of 1.7 and a hydraulic load factor of 1.3. The factored shear and moments were used to size and determine the adequacy of the u-frame structure and its required reinforcement.

Foundation Design

The pile foundation is shown on exhibits CIP-1 and CIP-2 and utilizes 24" PPC piles. CPGA was used to determine the arrangement and length of the pile foundation for the sector gate bay monolith. Loadings were input into the CPGA programs using applicable overstress values. The pile length was determined from pile capacity curves supplied by CEMVN-ED-F, using a F.S. of 2.0. Deflections were determined based on an Es value reduced for group effects. The group effect reductions were taken according to the values shown on the pile capacity curve supplied by CEMVN-ED-F.

7.3.2 U-Frame Lock Analysis

Chamber Monoliths

1. The design of the riverside chamber monolith base slab was accomplished using Excel (or MathCAD) to design the reinforcement, and finite element programs were used to analyze the loading conditions and identify the maximum loads for various section with in the structures. CWFRAME, STAAD.Pro and SAP2000 were all utilized in the design of the structure with the most conservative numbers used in the design of the reinforcement.
2. The Pile Group Analysis Program – CPGA was utilized to investigate the design and loading of the pile foundation.

Lakeside and Riverside Gate Bay Monoliths

1. The design of the walls was accomplished using Excel (or MathCAD) to design the reinforcement, CFRAME to analyze the loading conditions and identify the maximum loads, and Concrete General Flexure Analysis (CGFAG) was utilized on the thrust block portion of the wall using Working Stress Design.
2. The longitudinal strips of the gate bays were designed using Excel (or MathCAD) to design the reinforcement, and STAAD.Pro to analyze the loading conditions and identify the maximum loads for each section.
3. The transverse strips of the gate bays were designed using Excel (or MathCAD) to design the reinforcement, and STAAD.Pro to analyze the loading conditions and identify the maximum loads for each section.

Pile Foundation Design

The cast-in-place design utilized the pile foundation design used in the float-in-place design analysis. The programs utilized in that design included Microsoft Excel, and the Case Pile Group Analysis Program (CPGA – X0008).

7.3.3 U-Frame Lock Analysis Investigations

Investigation of the Gate Monolith Analysis

The Huntington District performed a check on one of the finite element analyses performed for the gate bay design. The analysis of the submitted design included finite element analysis utilizing the STAAD.Pro program. The bulkhead and the sector gate reactions assumed to be resisted by the strength of this section indicated that the overall monolith stability was dependent on this section of the monolith. A partial review of the calculations and input file was performed to check for any conflicting or incorrect information. It was assumed that all of the drawings and dimensions included in the design calculations were correct.

In reviewing the calculations and the input file it appeared as though there were several errors or omissions. The differing loads were all added, removed, or changed as necessary in order to run the model to investigate whether the changes had any significant effect on the results of the finite element analysis. The analysis showed that there were no significant changes in the results; the most significant change was the maximum positive moment in the slab but the change in moment would not significantly affect the quantities of steel and concrete that are required at this stage of the design. In addition to performing a general check of the calculations and the input file, the results, assumptions, and manner in which the section was analyzed was investigated. Overall it appeared as though the analysis was performed in a conservative manner; changing some of the assumptions may provide a more realistic model as well as a better design and cost for the CIP alternative.

Chamber Monolith Alternative Analysis Check

The chamber monolith transverse slab calculations were also performed utilizing the STAAD.Pro program. The load case that was run by LRH and compared with the graphical results of MVN was the dewatered load case. In order to check the accuracy of the model as well as the validity of the load assumptions an alternative model was created utilizing beams for each section of the chamber monolith. The loads were constructed based upon information gathered from a drawing within the design calculations of a chamber monolith done by MVN showing vertical loading. The results of the alternative analysis performed by LRH indicated that there may be a larger moment to design for in the area that the chamber wall connects to the base slab than the results given by MVN in the SAP2000 results. The localized larger moment calculated in the STAAD.Pro model is less then the CWFRAM moment that was used in the design according to the summary of the results and therefore should not affect the reinforcement calculations.

See Appendix I for further discussion on the checks performed by LRH on the finite element analysis portion of the design as briefly discussed above.

7.4 Construction Layout and Sequence

North Bypass Channel Construction

The first step in the construction sequence for the CIP alternative is to open the north bypass channel. As with the FIP alternative the north bypass channel is for two-way traffic, and is composed of a transit bypass channel and a laying bypass channel. Three 78-foot diameter protection cells will be constructed at the south end of the bypass channel, concurrent with bypass excavation. The channel corner riprap protection will be placed. Prior to opening the bypass, 4-62.8 foot diameter protection cells will be placed on the west side of the by-pass channel. These protection cells will guide traffic into the bypass channel and protect the future cofferdam. Total excavation required for the by-pass channel is approximately 840,000 cubic yards. The material will be dredged with a hydraulic dredge on a barge and transported via dredge pipe to the disposal area along the MRGO. See Exhibits CIP-23 and CIP-24 for dredge and disposal location and sequence.

Tug assistance vessel contracts will be set up to begin when the north bypass channel is opened to navigation. Tug assistance vessels (push boats) will be stationed at each end of the bypass to assist tows through the bypass channel. Two push boats will be required (24 hours per day and 7 days a week) at each end through the duration of lock construction.

Lock Excavation

Once traffic is re-routed to the by-pass channel, lock excavation can commence. Excavated material will be dredged and placed in the disposal area on the east bank near the MRGO. In addition to the lock excavation required for the footprint of the lock, pre-excavation down to El. (-) 60 will be required along the perimeter of the footprint. This pre-excavation is required for cofferdam cell installation. Again, this material will be dredged and placed in the disposal area. Total excavation required for the lock and the necessary pre-excavation for the cofferdam cells is approximately 2,150,000 cubic yards.

Normally, lock excavation would be done in the dry once the cofferdam is complete and off-road trucks would be utilized to haul material. If the material was to stay on-site adjacent to new lock this would be the method for lock excavation. However, since the material is being disposed of in a off-site location the more economical solution is to dredge the material in the wet from a barge and utilize dredge pipes to transport the material to the disposal site.

Cofferdam Installation

Prior to setting sheet pile for the cofferdam the foundation must be improved. Based on URS's design under Task Order No. 2, Contract No. DACW29-02-D-0008 soil improvement below the cofferdam is necessary. Therefore, the jet grouting activities from a barge will have to be completed. Once the soil improvement zone is complete then sheet pile installation for the cofferdam can commence.

The cofferdam consists of 45 – 67.3 foot diameter cells and arcs. Sheet pile for cofferdam cells extend from El. +5.0 to El. (-) 90.0 and will be driven with a vibratory hammer from a barge mounted crane. Cells will be in-filled with sand by barge mounted crane with a clam shell from El. (-) 60.0 to El. +3.5. A stone cap will be placed on the top 1.5 feet of the cell. Soil improvement will lead sheet pile activities.

Once cofferdam cells are in place, a large rock berm on the land side of the cells is required for global stability. The rock berm material will be brought to the site by barge and placed by barge mounted crane and clam shell prior to dewatering activities. See Exhibit No. CIP-16 below for completed cofferdam.

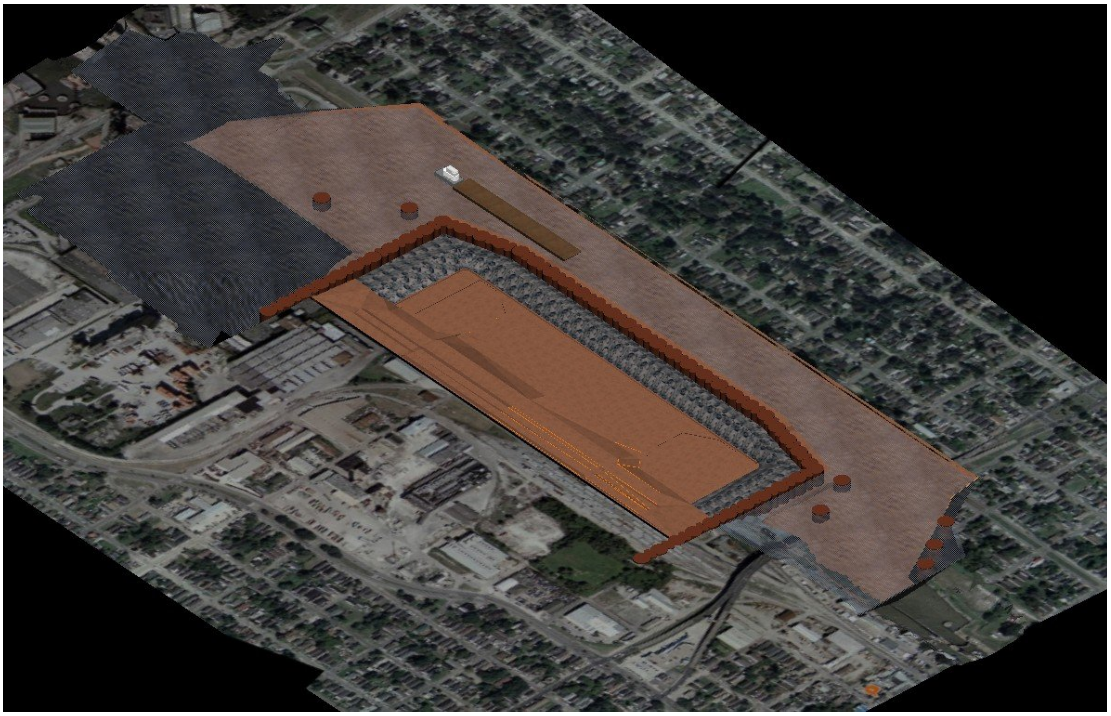


Figure 7.8 Completed Cofferdam

Dewatering and Slope Stability

Once the cofferdam is in place, but prior to dewatering some measures for slope stability have to be constructed. Soil improvement is necessary on the west bank at the top of the excavated slope. Land based equipment will be utilized for jet grouting to improve the soil on the west excavated slope.

In addition, a sheet pile cutoff wall is necessary at the top of the excavated slope parallel to the floodwall to prevent seepage from entering the excavation and for slope stability. Sheet pile will be driven with a vibratory hammer suspended from a land based crane.

Based on URS's preliminary design, a series of pumps/sumps/wells would need to be installed to dewater the excavation. One method is to place a series of wells around the perimeter of the excavation with land based equipment to dewater.

Once dewatering is complete, sumps, wells or well points would be installed at the inboard rock fill berm adjacent to the cofferdam to relieve uplift pressure and lower the groundwater level. Again, land based equipment could be used.

Also, pressure relief wells would be installed on approximately 200 foot centers by drilling from the top of the cofferdam and along the top of the west bank slope with land based equipment. In addition, instrumentation would also be installed to monitor the west excavated slope and the cofferdam.

Foundation Preparation
Once the lock excavation is dry and all associated instrumentation and dewatering system is in place, foundation piling will be installed. Foundation piles consist of 24" X 24" precast concrete pile 120 feet long spaced on approximately 10' centers with tighter spacing under the walls. A vibratory or impact hammer could be utilized to drive the piling. The land based crane will most likely be on mats for stability. Piling for the gate modules would be done first because gate module concrete would need to be completed so machinery and sector gate work could start. Also, the sheet pile cutoff walls which are transverse to the lock would be completed at this time.

Subsequent to pile installation a working slab of concrete would be placed on the piles as a starter for module concrete work. The gate modules would be completed first. In addition, piling operations would have to be at least 100' away from the gate modules in order not to induce vibrations on freshly placed concrete.

Lock Structure
Module concrete would start at the gates and progress inward in the chamber. Placements would be staggered to maximize distance of freshly placed concrete from pile driving operations. Traditional cantilever forms would be utilized for concrete. A number of land based cranes on mats would be required for movement of forms, placement of resteel and embedded items. Installation of water and air lines would be necessary for cleanup of concrete placements and curing concrete. In addition, lights would be required if a second shift was necessary due to temperature restrictions on placement of concrete.

Once gate modules are complete, then installation of sector gates, machinery, culvert valves, electrical and mechanical systems can commence while chamber modules are being completed.

An on-site batch plant (or one in close proximity) would be required that would produce at least 150 cubic yards an hour. Appropriate aggregate stockpiles and conveying system would be required. Conveying concrete to the placement could either be by bucket or a conveying system.

Backfill West Side of Structure and Levee Tie-In
After lock concrete is complete the lock excavation will be re-watered and the north and south ends of the cofferdam will be taken out. The sand fill in the cofferdam cells will be used for backfill on the west side. The sand backfill must be barged from the cofferdam and deposited with a clam shell. Fill will be brought up to El. 5.0. The sand backfill will then be placed along the lock wall. Also, the rock berm on the inboard side of the cells will be used as random fill.

The remaining backfill for the west side will be dredged material. Sufficient dredging operations required south of the new lock, including a portion of the south bypass channel at the existing lock, will be delayed so that disposed material is used as lock fill.

There are two reasons for completing the west side backfill prior to opening up the lock as a pass-though. The first reason is the maintenance and administration building would need to be complete to tie-in controls for the new lock. Therefore, backfilling the west side early is important so the building can be constructed. The second reason is that once traffic is diverted through the new lock then it would be difficult to backfill with equipment on barges and not impede traffic.

Once random and granular backfill is complete then the levee tie-ins can be constructed.

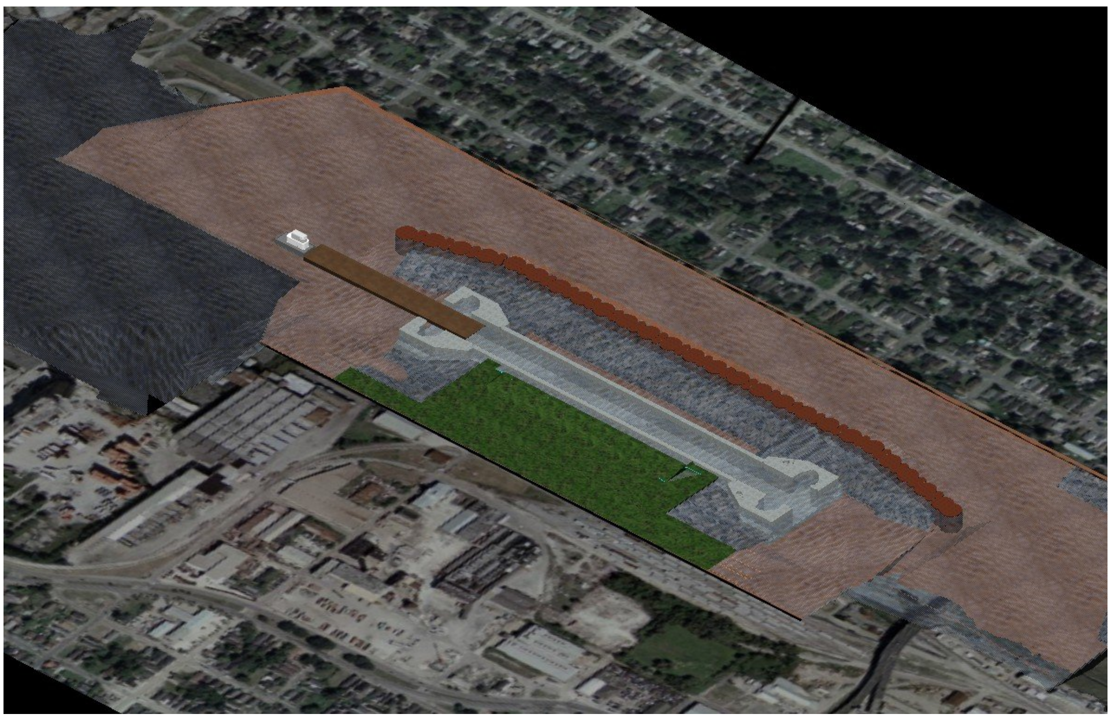


Figure 7.9 Open as Pass - Through Lock

Open Lock as Pass-Through Lock
In order to backfill the east side of the new lock traffic will have to be diverted. At this point water stage will still be controlled by the old lock.

Backfill East Side and Complete Site Work
Backfilling the east side will be performed by removing the sand from the cofferdam cells that run north – south parallel to the locks. Enough granular material exists to place against the wall with a barge mounted crane and clam shell. The rock berm will remain in place and be considered random fill. The balance of random material will be brought back from the disposal area. Once random fill placement is complete then the levee tie-ins can be completed. When the site work is complete the lock is ready for operation.

7.5 Cofferdam Design

A feasibility level design of the cofferdam for traditional lock construction in the dry was completed by URS. The AE's design was to build on the preliminary layout of the cofferdam system performed by CEMVN. The design was to refine and improve upon the cell layout, depth of excavation required for installing the cofferdam, cell and berm fill and prepare a general plan for maintaining the excavated area dry. The A-E was also to provide recommendations and risks for constructing cofferdams on soft soils. The current cofferdam design will leave a 220 foot wide bypass channel at a minimum bottom elevation of El. -12.0 NAVD88 to allow for two way barge traffic at the east bank side of the cofferdam. This alternative was accomplished by optimizing the cell diameter and modifying the east bank of the channel to the greatest extent possible without affecting the integrity of the existing T-wall. The cofferdam cells were analyzed for three vessel impact loading conditions. The conditions consisted of no vessel impact loading, end cells receiving a 600 kip vessel loading and side cells with 160 kip vessel loading. A more detailed discussion on the cofferdam design can be found in Appendix C.

8. Cost Comparisons

Project Time & Cost, Inc. (PT&C) was retained by the USACE to develop cost estimates and schedules for each option. Due to environmental commitments identified in the Environmental Impact Statement (EIS), each option includes both an unconstrained and constrained case. While the unconstrained cases optimize schedule and production, the constrained versions consider the impacts to the environment and community as outlined in the EIS. Appendix A details the environmental commitments considered in the estimate. The commitments that contribute most directly to cost include those associated with asphalt repair, noise control, and limited work hours (which results in longer schedules and, consequently, more inflation and extended overhead)

The intent of this report is not to provide Total Project Cost (TPC); rather, this report is to provide comparative cost estimates for the CIP and FIP alternatives. MVN requested that the cost estimate comparison (and thereby the contents of this report) be limited in scope to those items that would contribute to the difference in cost between the CIP and the FIP alternatives. In other words, major items of work have been intentionally excluded from the cost estimates. While these items would have no bearing on the delta between the cost of CIP and FIP, they would have to be considered in order to estimate TPC. The cost estimates in this report primarily reflect only the construction of the lock itself for all relative construction elements between the CIP and FIP methods, in order to compare and highlight the differences in cost and schedule between CIP and FIP. The following items have been excluded from the cost estimates:

- St. Claude / St. Charles Bridge construction and light rail line;
- Upstream and downstream approach walls;

- Existing lock demolition, demolition of structures;
- New road to link St. Bernard Highway and West Judge Perez Boulevard;
- Detour road in St. Bernard Parish;
- Any levees, floodwalls, or floodgates not associated with the new lock project;
- Real estate, relocations, engineering & design and contract supervision & administration (feature accounts 01, 02, 30, and 31);
- Prior expenditures or sunk dollars; and final dredging of the South bypass channel.

8.1 Schedule Comparisons

As part of this report, PT&C developed a detailed construction schedule for both sub-options for both the CIP and FIP lock options. Table 8.1 lists the project duration in months for each of the four options.

Estimate Type	CIP	FIP	Variance
Unconstrained	97	137	40
Constrained	131	140	9
Variance	34	3	

Table 8.1 Project Construction Duration (Months)

8.1.1 Unconstrained Schedules

The Unconstrained Schedules assume that no outside event, other than typical weather, will impact the construction of the lock. Both the CIP and FIP options assume that the work week will be 6 – 10 hour days. In most cases it also assumes that the contractor will use two shifts to drive piling, excavate/dredge material and construct the lock structures. During the construction of the lock structures, it is assumed that the contractor will perform forming and reinforcing during the day shift and pour concrete during the night shift. The 40-month delta between the CIP and FIP schedules is the result of the construction of the FIP modules in the graving site. Since only one module can be built at a time in the graving site, it will take longer to construct the lock structure.

8.1.2 Constrained Schedules

The Constrained Schedules assume that, besides weather, the construction will be constrained by the items found in the Environmental Commitments list Appendix G. The restrictions that affect the constrained option schedules are: vehicle transportation and noise. The vehicle restrictions include operating heavy vehicles only during 10 hours of daylight. The main noise restriction is the piling operation. Piling can only be performed during 10 hours of daylight. The nine month delta between the CIP and FIP options is the result of the prolonged construction of the FIP lock modules at the graving site.

8.1.3 Unconstrained vs. Constrained Schedules

The shortest duration for construction of the replacement lock is the unconstrained CIP option (97 months total time). The 34 months difference between the unconstrained and constrained CIP options is due to limiting the work schedule at the lock site to one, 10-hour shift per day for piling, heavy vehicle operation, and dredging. The three months difference between the FIP constrained and unconstrained schedules is due to limiting the work schedule at the lock site to one, 10-hour shift per day for dredging. The remaining FIP lock construction work is not affected by any limits to the work schedule at the lock site because the lock module construction will be done at in the graving site and it is on the critical path for both FIP plans.

8.2 Quantities

The CELRD team was asked to prepare quantities for the cast-in-place lock alternative that would be used in the report cost comparison. The quantity list included concrete volume, reinforcement and foundation piling. The New Orleans District provided ten binders of reinforcement calculations and a folio of CIP drawings from which CELRD prepared the quantities.

8.2.1 Concrete Quantities

LRD reviewed the drawing folio of the IHNC Lock prepared by MVN. With dimensions taken from the drawing folio, LRH constructed a three dimensional model of the lock in MicroStation. Concrete quantities for the gate bay monoliths were calculated utilizing MicroStation's "measure volume" command. The chamber quantities were calculated by finding the area from a cross section of the U-shaped lock chamber and multiplying it by the length of the individual monolith to calculate the volume of concrete. Comparison sheet of the FIP and CIP is included in Appendix A "Project Time and Cost". Detailed quantity sheets for concrete can be found in Appendix E.

8.2.2 Reinforcement Quantities

The reinforcement used in calculating the quantity of steel for the cast-in-place reinforced concrete structure has been taken from the ten binders submitted to LRH containing the calculations performed by MVN for the IHNC Lock Replacement Project, Attached as Appendix E. In any case where there appeared to be multiple computations for the same section the most conservative reinforcement was used in the quantity calculations. All exterior or interior surfaces where calculations were not identified were assumed to be #9 @ 12 based upon EM 1110-2-2104, paragraph 2-8 "Temperature and Shrinkage Reinforcement". The calculations for the Gate bay monolith reinforcement were pulled from binders 6 (Transverse Gate bay 2D Slab), 7 (Longitudinal Gate bay 2D Slab), and 10 (Gate bay Walls). The calculations for the chamber monolith reinforcement were pulled from binders 2 (Ship Impact and Reinforcement), 4 (Chamber 2D Transverse Slab), 5 (Chamber 2D Slab), 8 (Chamber Monoliths), and 9 (RS Chamber Monolith). Appendix E goes into further detail concerning what information was used in designing the reinforcement based upon the given calculations.

8.3 Cost Deltas

Table 8.2 shows the major cost deltas for the constrained options of the CIP and FIP. The line item costs shown represent the anticipated total cost for each line item; however, since certain items have been excluded from the estimate, Table 8.2 **does not show the total construction costs for either the CIP or the FIP option.**

Item	CIP	FIP	Delta
Graving Site	0	69,401,000	69,401,000
Lock Site Cofferdam	310,187,000	0	(310,187,000)
Excavating/Dredging Material Handing	65,273,000	51,058,000	(14,215,000)
Lock Foundation	95,095,000	157,002,000	61,907,000
Lock Structure	197,632,000	417,742,000	220,110,000
Site Work	10,584,000	10,348,000	(236,000)
Mob/De-Mob	20,273,000	32,833,000	12,560,000
Mechanical	85,837,000	101,020,000	15,183,000
Electrical	7,160,000	6,945,000	(215,000)
Totals	792,041,000	846,349,000	54,308,000

Table 8.2 Delta Costs between the CIP and FIP Constrained Options ($)

The costs for each option include all indirect costs and contingencies associated with the variances. A breakdown of the costs can be found in Appendix A (CIP & FIP Comparison Costs).

8.3.1 Graving Site

The Graving Site is only applicable to the FIP option. The Graving Site is used to construct the lock models before they are floated to the lock site. This feature consists of relocating a levee around the graving pit to maintain the current flood protection, excavating the graving pit, constructing a casting bed, removing and reconstructing the closure plug, dewatering/re-watering and maintaining the dewatered state. The removal and reconstruction of the closure plug is a major cost driver because it has to be repeated for each of the five modules.

8.3.2 Lock Site Cofferdam

The Lock Site Cofferdam is only applicable to the CIP option. The Cofferdam will enclose the entire lock construction site so that the chamber can be cast in dry conditions. This feature of the CIP is highly dependent on the price of material. As such, the current material shortages that MVN is experiencing have had a drastic effect on the cost associated with this project feature. Additionally, the local soil conditions require jet grouting to be performed to establish a sound foundation for the coffer cells.

8.3.3 Excavating/Dredging Material Handling

The Excavating/Dredging Material Handling costs vary between the CIP and FIP options. The cost delta is based on the difference in length of the by-pass channel and the amount of excavation required for each option. The CIP option requires a significantly greater amount of excavation to provide space for the cofferdam while maintaining canal navigation. The re-use of the coffercell granular fill as random backfill helps to reduce the costs of the larger backfill requirements associated with the CIP option.

8.3.4 Lock Foundation

The Lock Foundation delta is based on the difference in foundation pile placement and the amount of tremie concrete used in the two options. The CIP option allows the pre-cast pre-stressed concrete (PPC) piling to be installed in the dry, where the FIP option requires steel pipe piles to be driven in the wet from barges. The material costs for the steel pipe piles are significantly greater than the PPC piles. Tremie concrete is required only in the FIP option. The area between the bottom of each FIP module and the channel floor requires the tremie concrete to be pumped in the wet. Another major contributor to the delta in the FIP option is the temporary timber protection barrier required to protect the construction area from canal traffic in the bypass channel.

8.3.5 Lock Structure

The delta between the CIP and FIP Lock Construction is due to the type of construction techniques used to build the lock. Even though the total mass of the lock remains approximately the same for both the CIP and FIP options, the cost to build the lock structure will be higher in the FIP option. The FIP lock structure is built in a graving site and then floated to the new lock site and set in place over the piling. Since only one FIP module can be built at a time, the construction is longer than the CIP option. The CIP lock structure is built in the dry inside the cofferdam. The cost per cubic yard of concrete in the module shells for the FIP method are substantially higher than the cost per cubic yard for mass concrete places for the CIP method. The

CIP option is a more traditional method of construction and will take less time and less specialized construction techniques would be required.

8.3.6 Mechanical

The mechanical systems in both the CIP and FIP options are the same. The only delta between the two options is the requirement for more bulkheads in the FIP option. These additional bulkheads are used to seal the modules during their transportation to the lock site.

8.3.7 Electrical

There is essentially no difference between the CIP and FIP options. The same electrical system was assumed to be used for both options and installed in methods with no significant differences.

9. Risk Assessment

For the purposes of this study, the Risk Assessment will be based on the design team's assessment of individual project requirements such as design and construction challenges, quality control verification, long term operability and navigation, and impacts to the community. Risk for the two construction methods was evaluated by the team as shown below.

ENGINEERING & DESIGN		
Technical Approach	Cast-In-Place	Float-In-Place
Design Quality	• C-I-P utilizes conventional design procedures and standard load cases. • Guide Specs, EM's, ETL's, readily available for C-I-P Structures • Lessons Learned from numerous COE Projects	• F-I-P requires specialized Marine Design Experience to assure transport stability, weight control and drafting, and set-down. • Guide Specs, EM's, ETL's, must be adapted from C-I-P Structures. Some criteria available from Braddock and Olmstead. • Lessons Learned from limited number of F-I-P Dam Projects.
Design Execution	• Addition Subsurface Investigations, Sampling, and Lab Testing will be required for the development of geotechnical design parameters. • Groundwater studies, instrumentation, and pump tests required for design of unwatering and dewatering systems. • Pile Load Testing Program Required for design and analysis of 24-Inch PCC Piles • Level 2 NISA Study required for mass concrete structures. Data may be available from other projects in MVN. (1 Year WES Study) • Deep Soil Mixing Lab and Field Testing completed in 2004. • WES modeling	• Plan as developed is feasible, however project of this scale has never been done before. Final design effort may identify additional design and construction issues that have not yet been addressed, leading to cost increases or schedule delays. • Pile Load Tests on 48-inch Pipe piles completed in 2000. No pile design issues identified at 50% submittal. • Flexible Base Analysis, Temporary Chamber Struts, and Tension Pile Connection details, need additional investigation. • Level 3 NISA Study required due to complex and unprecedented design. Existing data not available. (2 year WES Study)

	required to confirm cofferdam does not constrain navigation.	
Work Areas and Logistics	• Current site plans require management of 1 Work area. • Work area size is very limited and constrained. • Contractor likely to pursue use of additional work areas thru Leases on existing industrial tracts.	• Current site plans require management of 2 Work areas. • Logistical inefficiencies with the movement of manpower and materials between sites, • 2 batch plants required. • Lock site lay down area is constrained. Contractor likely to pursue use of additional work areas thru Leases on existing industrial tracts.
Environmental Compliance	• NPDES Permit for Batch plant. • C-I-P is not addressed in EIS. • Additional public and NEPA coordination required in SEIS	• Graving Site requires mitigation of wetland and environmental impacts. • 2 NPDES Permits required for 2 Batch Plant Point Discharges. • F-I-P Lock Plan complies with current EIS and NEPA requirements.

CONSTRUCTION		
Technical Approach	Cast-In-Place	Float-In-Place
Construction Execution	• Work Sequence is more linear and provides less opportunity for concurrent work efforts. • Large pool of local and national contractors having experience with conventional piling, dredging, forming, and concrete placement. • Specialty contractors required for Dewatering, Jet Grouting and Deep Soil Mixing. • Cofferdam overtopping due to storm surges could cause significant damage to work areas and delay progress.	• Work sequence allows for concurrent activities at Graving Site and Lock Site. • Specialty contractors required for installation of deep foundation piles. • Transportation of floating units, alignment, set down requires marine specialists on staff. • Module transportation is both weather and seasonally dependant. • Tidal effects on water surface elevation.
Contractor	• Traditional	• Availability of specialty

Expertise	construction methods, large pool of local and national contractors capable of performing work. • Reliance on specialty contractors within linear construction can constrain execution of work.	contractors is limited and can impact schedule and costs. • Concurrent construction schedules allow for some critical path adjustments if Specialty contractors are delayed or unavailable.
Construction Safety	• Standard Safety Concerns dealing with worker safety, equipment. • Monitoring of cofferdam will require significant effort. • Stability of temporary construction slopes within cofferdam pose major risk to integrity of Hurricane Protection Wall. • Slope movements triggered by Railroad surcharge or long tem soil creep could distress floodwalls and compromise protection of west side communities. • By-Pass channel alignment requires dredging which significantly encroaches on East Bank slopes. Slope movement triggered by loss of support or prop wash erosion could distress floodwall and compromise protection of East Side communities.	• Same as C-I-P, plus significant diving requirements at depths in excess of 60 feet, which requires additional diver certifications and dive safety procedures. Zero Visibility below 10 feet depth. • Divers must perform difficult tasks for construction of temporary bulkheads for underbase grouting, flat jack placement, welding and burning, tension pile connections. • Confined space issues with inspection of module components, underbase grouting. • Use of Robotics to supplement underwater inspection is feasible but also has limitations and very costly.
Quality Control	• Standard QA/QC requirements for most components and features. • Visual inspection (surveys, GPS, etc) can be performed on most items.	• More intense QA/QC requirements. Special inspection procedures for weight control, rebar specs, concrete mix, additives and high tolerance underwater construction. • Quality control of foundation and structural connections is

		limited to soundings and diving inspections. Visual observation and measurements are restricted to tell-tales and templates. • Many uncertainties in regards to final elevations, placements, and disposition of critical components including foundation to module interaction. • QA/QC Methodologies must be formulated in the field. • Extensive involvement required from Designer of Record and Design Team.
Biddability	• Multiple, sequential contracts of 3-5 years duration each. • Conventional construction requirements with large pool of national and local contractors. Some reliance on specialty contractors.	• Multiple, sequential contracts of 3-5 years duration each. Some conventional construction requirements with heavy reliance on specialty subcontractors for specific work features. Biddability issues may be comparable to Olmstead and Portuguese dams.
Duration	• 10.9 years	• 11.75 years
Operability & Aesthetics	• Once complete, the product will be identical to F-I-P Design.	• Once complete, the product will be identical to C-I-P Design.

PROJECT MANAGEMENT		
Technical Approach	Cast-In-Place	Float-In-Place
CG - Project Management	• Project can be broken into multiple contracts, however appropriate phasing plan is critical to reduce government and contractor liability on completed work. Once cofferdam is complete project is difficult to break up into phases.	• Project can be broken into multiple contracts however appropriate phasing plan is critical to reduce government and contractor liability on completed work. Repair or correction of deficiencies may be more challenging to remediate.
Cost Growth Potential	• Potential cost growth areas include jet grouting of cofferdam foundation, dewatering, and the characterization and management of contaminated sediments • Instrumentation	• Potential cost growth areas include Graving Site construction, closure system, dewatering, and Instrumentation. • Gatebay Module construction • Module Weight Control and Transportation • Tolerances and Alignment
Life Cycle Costs	• Life Cycle costs are assumed to be the same beyond initial construction investment	• Life Cycle costs are assumed to be the same beyond initial construction investment
Port User and Navigation Impacts	• Helper Boats Required to assist Navigation Traffic thru By-Pass. • Navigation alignment is tighter at Lock under C-I-P. • WES modeling required to confirm approach alignment around cofferdam corner is navigable.	• Helper Boats Required to assist with Navigation thru the By-Pass. • IHN Canal gets shut-down at least 5 times during Lock Module transportation.
Impacts to Local Communities and Infrastructure	• General Activities: Construction effort is of a continuous and highly active duration. 100% of	• General Activities: Construction is of sequential duration that will result in periods of highly active and

	work will take place at the Lock site. • Noise: High noise activities include cofferdam construction (1 year), lock foundation pile installation (x year), and batch plant operation (2.75 years). • Traffic: Local roadways and infrastructure will be required for delivery of manpower and materials (10.9 years).	minimally active construction periods. Approximately 40% of construction activities will take place at off-site at graving site. • Noise: High noise activities include lock foundation pile installation (3 year), and batch plant operation (6.1 years) • Traffic: Local infrastructure will be required for delivery of manpower and materials (11.75 years).
Dredging, & Disposal Issues	• C-I-P has larger dredge foot print and will result in 3.6 million CY of dredge spoil.	• F-I-P has smaller dredge foot print and will result in 2.2 million CY of dredge spoil. • Casting facility will require 614,000 CY of excavation requiring temporary stockpile location, permanent disposal site, or approved reutilization plan.
Preparation of SEIS to Address Sediment Characterization	• C-I-P construction must be addressed in SEIS, coordinated thru NEPA process and vetted thru HQ and ASA. • SEIS must address sediment characterization and disposal issues.	• F-I-P Construction Alternative has been coordinated thru NEPA process and has been endorsed by HQ and ASA. • SEIS must address sediment characterization and disposal issues.

10. Recommendation from MVN

The CELRD team has reviewed the documents provided by CEMVN for the two methods of construction, and identified critical features of the work for both alternatives that must be further developed to make an accurate comparison of the proposed construction methods. Our goal for this study was to identify issues the

New Orleans District needed to consider to select the construction method that brings the best overall value to the government, sponsors, and stake holders, with regards to cost, schedule, risks, and impacts.

11. Considerations for Future Design

During the process of preparing this letter report issues have arisen that should be passed on to the MVN team for future review. Some issues come from the LRD team and some have been raised by reviewers during the technical review. The issues are as follows:

- The operating case of maximum hurricane, river side 0.0 and lake side 13.0 is so unlikely as to be impossible. There is no way that a thirteen-foot storm surge in the lake will be accompanied by simultaneously draining the river down to its lowest possible stage. Using this as the maximum reverse head condition results in the lock being overbuilt (Dr. Checks - Doyle Hunt).

- The gates will be too large to lift with any reasonable crane; not necessarily too heavy, just too large. Serious consideration should be given to constructing the gates such that they are built in horizontally-stacked sections, so that the gates can be lifted out by a reasonably-sized derrick for maintenance. Failure to do so will dictate that all maintenance be done in place, which will require lengthy lock closures of 120 to 150 days duration to sandblast and paint all four gates, replace pintles and upper hinge sections, and replace seals (Dr. Checks - Doyle Hunt).

- Since 5.11 and 5.12 identify emergency bulkheads for both ends of the lock that means that TWO derricks are required in 5.16, one for each end, since one derrick will not have the reach to place bulkheads at either end. A better solution would be to place ONE set of emergency bulkheads, capable of sealing against either positive or reverse-head flow, midway between the two sets of lock gates, and use a single derrick at that location. For maintenance purposes, the bulkheads could be transferred to the ends of the lock by barge for placement by a floating derrick (Dr. Checks - Doyle Hunt).

- Ladders in the lock wall do NOT need "resting platforms" unless they are intended for maintenance access. For emergency egress purposes, the ladders do not need to comply with OSHA or EM385-1-1 requirements to break up the climb into 25-foot increments with resting platforms in between. However, emergency egress ladders can still have a fall protection system consisting of a modular davit supporting a man-lift winch and self-retracting lanyard system, which can also be used with a Lifesling (tm) system for lifesaving purposes. Ladder recesses in the lock wall should be deep enough (3 feet square in plan view) to allow a fallen mariner to get his body completely out of the way of a vessel even before he climbs up, to minimize the potential for crush injuries(Dr. Checks - Doyle Hunt).

- The use of floating mooring bitts, properly designed, will eliminate the need for line handling by lock personnel, which is desirable for safety reasons as well as allowing the lock to operate with fewer personnel (Dr. Checks - Doyle Hunt).

- Demolition using explosives could be problematical due to the proximity of the main-line Mississippi River levee, and the shock and vibration that may be transmitted through the soil to the levee. There is nothing listed in the letter report about whether or not explosives can even safely be used in close proximity to the levee, and so the entire analysis of in-the-wet versus in-the-dry demolition is fallacious (Dr. Checks - Doyle Hunt).

- I the second paragraph it is proposed that the flat jacks be placed onto the setdown piles. This appears problemactic and risky. All of the pressure lines and the jacks would be exposed to damage. Also, in this configuration if something goes wrong with the jacks or pressure lines it will all have to be addressed with divers. an alternative to this configuration would be similar to the solution used for the Braddock Dam construction. The jacks and pressure lines were built internal to the float-in segments. The jack pushed onto a steel piston that engaged the set down piles when the segment was set down. I this way, the jack and pipes were all accessible and maintanable. (Dr Checks – William Karaffa)

- The last paragraph discusses the temporary transport bulkheads. Based on lessons learned from Braddock Dam. Although you would like to save money on these, make sure that they are designed to handle the maximum head. You may have a stacking scenario thought out the would place the temporaries at a position which requires a lower hydrostatic design load, but when the Contractor actually does the work, they will want to have full flexibility to place these temporary bulkheads in any order or sequance that suits there means and methods. (Dr Checks – William Karaffa)

- Third Paragraph. Is there any significant impact to the ballast plan if the Contractor elects to construct the tremie platform over the full length of the segment? I assume this would be done prior to setdown of the segment. (Dr Checks – William Karaffa)

- Fourth Paragraph. The load is transferred to the foundation piles via the underbase tremie concrete. This concerns me in that in reality, you can expect to have areas on the bottom of the segment not in full contact with the tremie fill. Baring voids due to trapped water pockets, this would be small sized gaps of 1/8 inch or less, caused by shrinkage of the tremie pour or settlement of the soft subbase due to the weight of the tremie. Can this foundation design tolerate some areas not being in full contact? Will loads sufficiently transfer if these gaps and voids occur? Has a rough order settlement calculation been done? Using the tremie to transfer load into the foundation piles, could be a disadvantage of the FIP alternative va CIP alternative, and the risk should be qualified. (Dr Checks – William Karaffa)

- The underbase tremie plan suggest the use of grout bags. It could be suggested that during formal P/S devleopment you consider using water to fill the bags in lieu of grout. Water is more forgiving then grout in the event that trouble is experienced in deploying (opeining) of the bag. (Dr Checks – William Karaffa)

- How will it be determined that the load is being transferred into the foundation piles appropriately? Instrumentation? (Dr. Checks - William Karaffa)

- The sixth paragraph notes that it will have to be determined "if" a tension pile needs to be prestressed. How will this be determined? (Dr Checks – William Karaffa)

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Exhibits

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Existing Site Plan 2 of 2	2-102	COM-4
Completed Site Plan 1 of 2	2-105	COM-5
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Culvert Intake Screens (recommend redesign)	7-8	COM-7
Lakeside Gatebay General Plan	5-501	COM-8
Maintenance Bulkhead Plan and Elev.	9-1	COM-9
Control House Floor Plan & Elevation	6-1	COM-10
Culvert Roller Gate Recess Plan	5-113	COM-11
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Riverside Chamber Module Load Diagrams - Reverse Head and Hurricane	LD-CM2-12	COM-15
Riverside Gatebay Module Load Diagram Operating Conditions	LD-GB1-4	COM-16
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Description	Dwg #	Exhibit #
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Lock Module Plan and Section	1-5	FIP-2
Lock Site Typical Section 3 0f 3	2-109	FIP-3
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Graving Site - Foundation, Grade Beams	GI904J00	FIP-22
Graving Site - Ready for Module Construction	GI905J00	FIP-23
Graving Site - Module Construction	GI906J00	FIP-24
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Bypass Channel Excavation	GI909J00	FIP-28
Lock Excavation	GI910J00	FIP-29
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River Side Gate Bay Module Placement	GI912J00	FIP-31
River Side Gate Bay Module Constructed	GI913J00	FIP-32
Module Construction Completed	GI914J00	FIP-33
Final Module Construction - Lock Operational	GI915J00	FIP-34
Material Handling Flowchart	GI601L00	FIP-35
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CIP Drawings		
Description	Dwg #	Exhibit #
Riverside Gatebay Foundation Pile Layout	S301	CIP-1
Riverside Chamber modules Foundation piles layout	S302	CIP-2
Riverside Plan and Elevation	S401	CIP-3
Lakeside Plan and Elevation	S402	CIP-4
Riverside Gatebay Masonry Plan	S404	CIP-5
Riverside Gatebay Sections A and B	S405	CIP-6
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Lock Concrete Complete	GI921J00	CIP-20
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Lock Operational	GI923J02	CIP-22
Material Handling Flowchart	GI602L00	CIP-23
Material Distribution Site Plan	GI403M00	CIP-24

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Appendix A: Inner Harbor Navigational Lock Replacement - Cost and Schedule Analysis (Project Time & Cost)

Appendix B:

Appendix C: Cast-In-Place Cofferdam - Feasibility Level Design (URS)

Appendix D: VE Study

Appendix E: Comparison Quantities

Appendix F: Quality Management Plan

Appendix G: Environmental Commitment

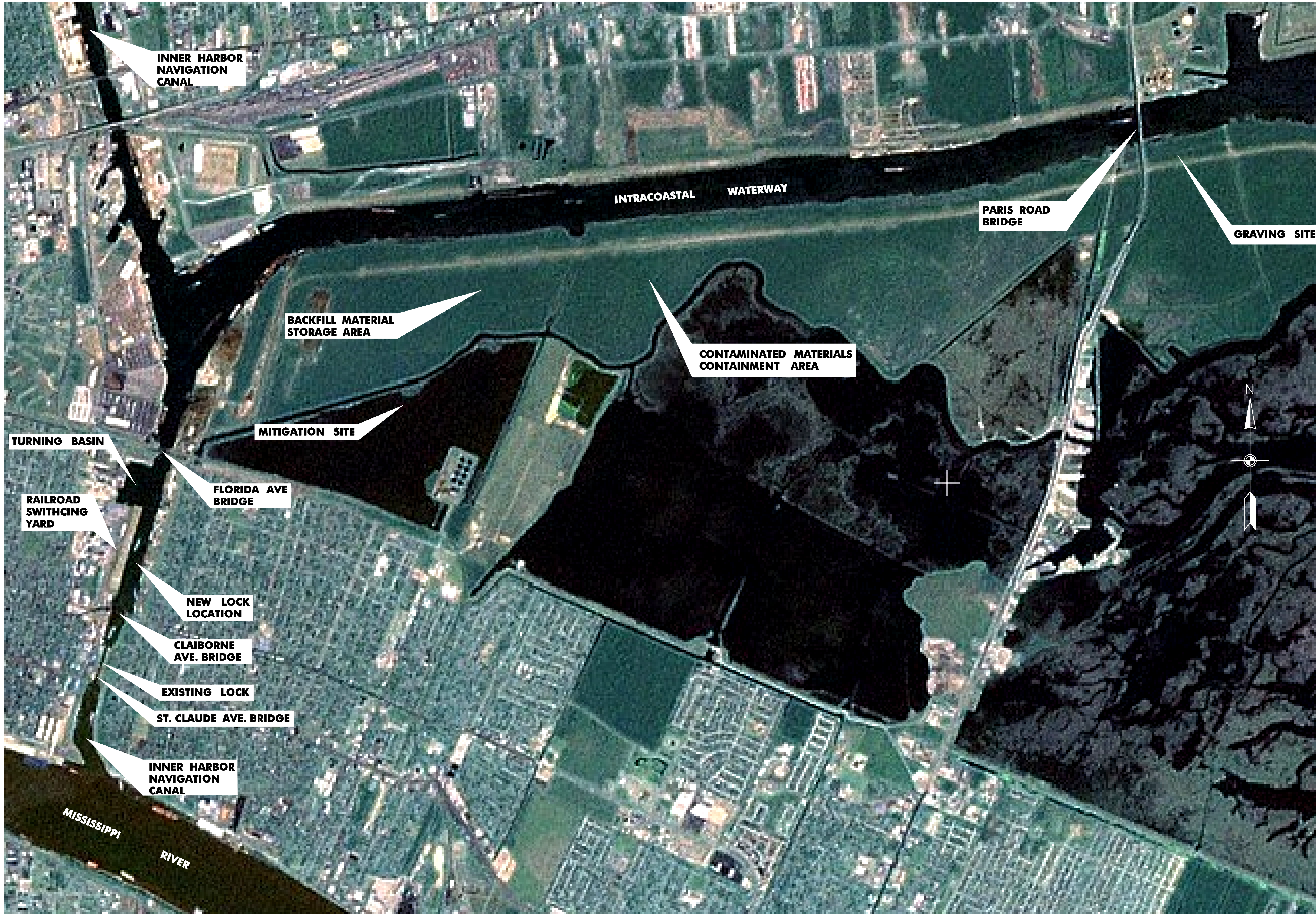
Appendix H: Cast in Place Analysis Investigation

Appendix I: Environmental Impact

Appendix J: MVN CIP Computations

Appendix D

The supporting appendices (A through J) for Appendix D are available on the enclosed CD.



SITE MAP

US Army Corps of Engineers
Huntington District

U.S. ARMY CORPS OF ENGINEERS
HUNTINGTON DISTRICT
HUNTINGTON, WEST VIRGINIA

Designed by: | Dwn by: MER | Ckd by: | Reviewed by: | Submitted by:

Date: 04/30/2007 | Rev. | Design file no. | Drawing code: | File name: GI401M00.dgn | Plot date: | Plot scale: 1" = 1'

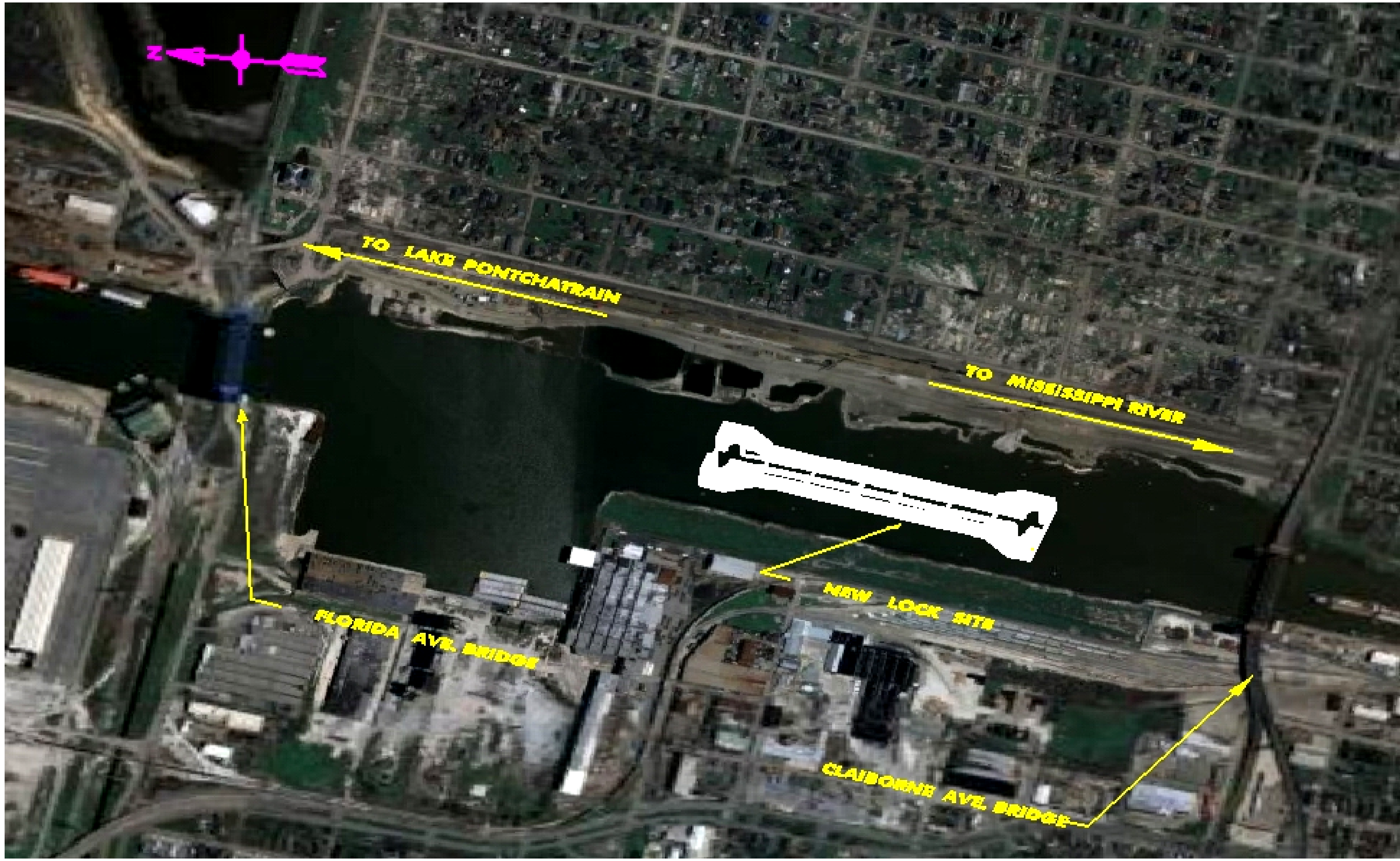
MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

SITE LOCATION MAP

PROJECT FEATURES

Sheet reference number:
GI401
Sheet 0 of 0

EXHIBIT COM-1



NEW LOCK LOCATION PLAN

US Army Corps of Engineers
Huntington District

MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

LOCK SITE PLAN

Sheet reference number:
GI950
Sheet 0 of 0

EXHIBIT COM-2

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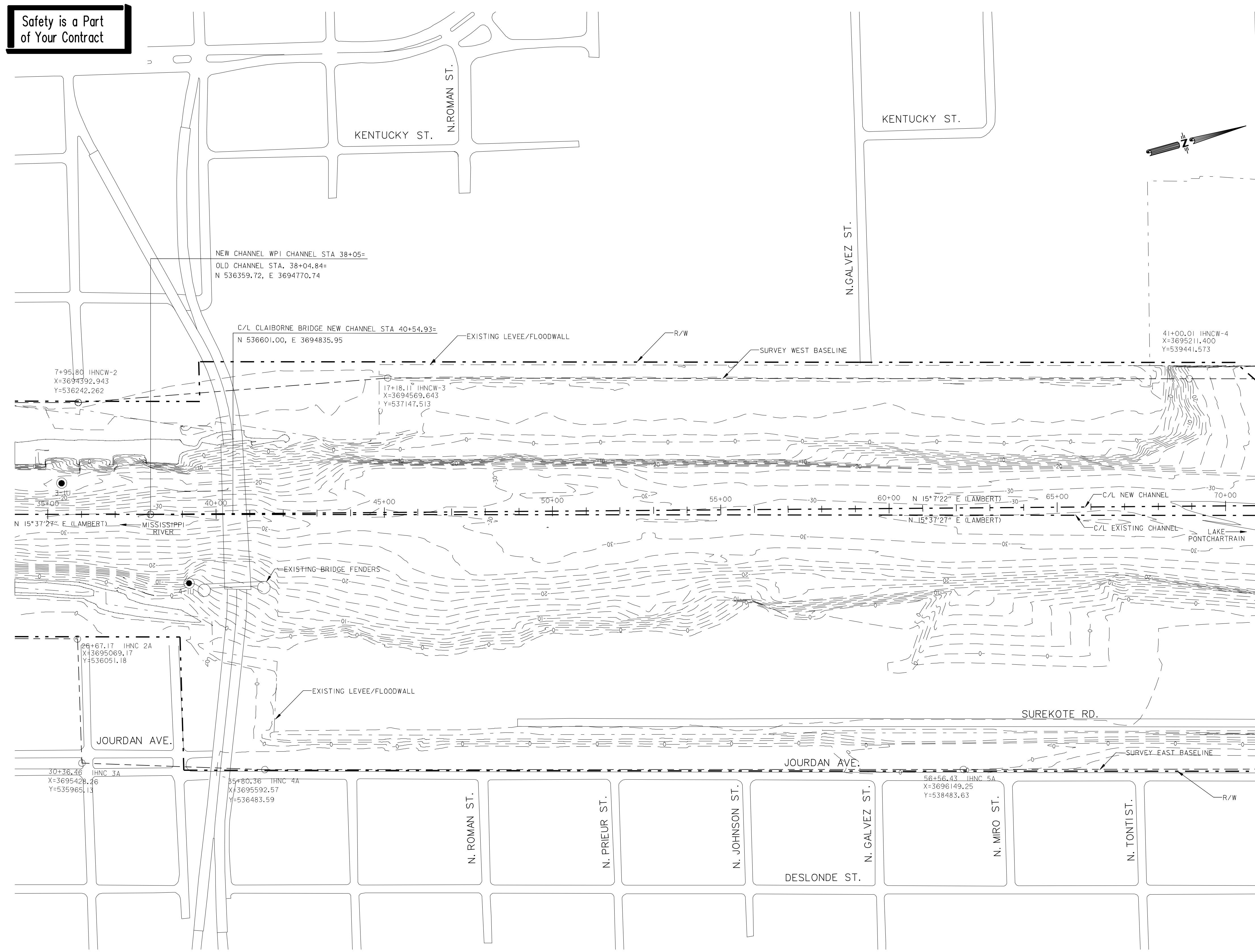

EXISTING SITE PLAN (1 OF 2)

SCALE: 1" = 100'

100' 0 100' 200' 300' 400'

NOTE:
CENTERLINE CHANNEL, EAST AND WEST BASELINES HAVE DIFFERENT STATIONING

This drawing has been reduced to half size

PHASE I DESIGN
100% SUBMITTAL
JULY 2006

FOR INFORMATION ONLY

JACOBS CIVIL INC.

URS
3500 N. Causeway Blvd., Suite 900
Metairie, Louisiana 70002
(504) 837-6326
PROJECT No. 10000586.00000

US Army Corps of Engineers
New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: L. WEAVER
CHECKED BY: RAJH
DRAWN BY: L. WEAVER
DATE: JUNE 2006
PLOT SCALE: 1200
PLOT DATE: JUNE 2006
SUBMITTED BY:
SOLICITATION NO. W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

EXISTING SITE PLAN
SHEET 1 OF 2

FILE NUMBER
H-2-46087
DWG. 2-101

EXHIBIT COM-3

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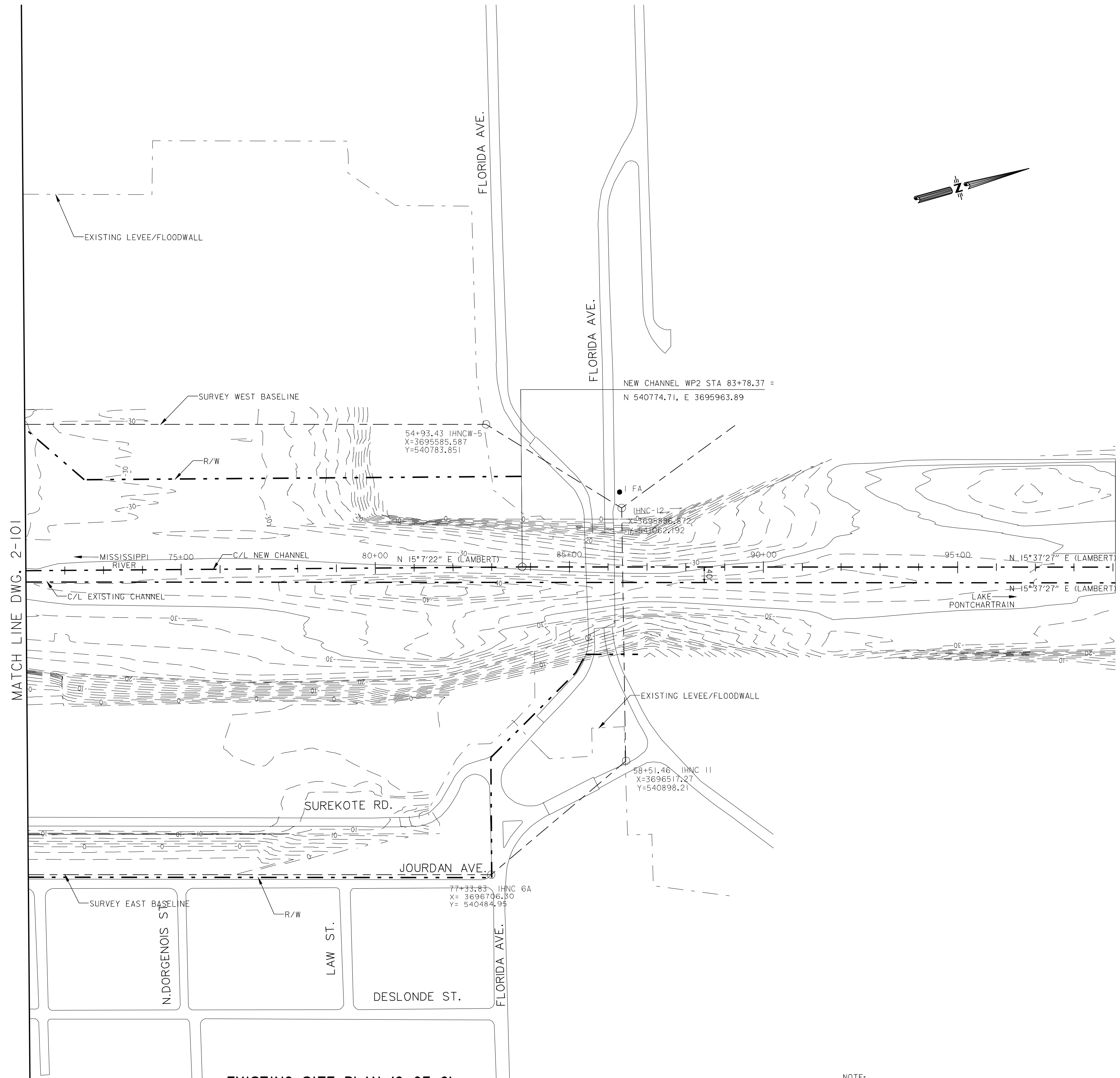

EXISTING SITE PLAN (2 OF 2)

SCALE: 1" = 100'

100' 0 100' 200' 300' 400'

NOTE:
CENTERLINE CHANNEL, EAST AND WEST BASELINES HAVE DIFFERENT STATIONING

This drawing has been reduced to half size

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US Army Corps of Engineers
New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: L. WEAVER
CHECKED BY: RASH
DRAWN BY: L. WEAVER
DATE: JUNE 2006
PLOT SCALE: 1200
PLOT DATE: JUNE 2006
SOLICITATION NO.: W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA
EXISTING SITE PLAN
SHEET 2 OF 2

FILE NUMBER
H-2-46087
DWG. 2-102

EXHIBIT COM-4

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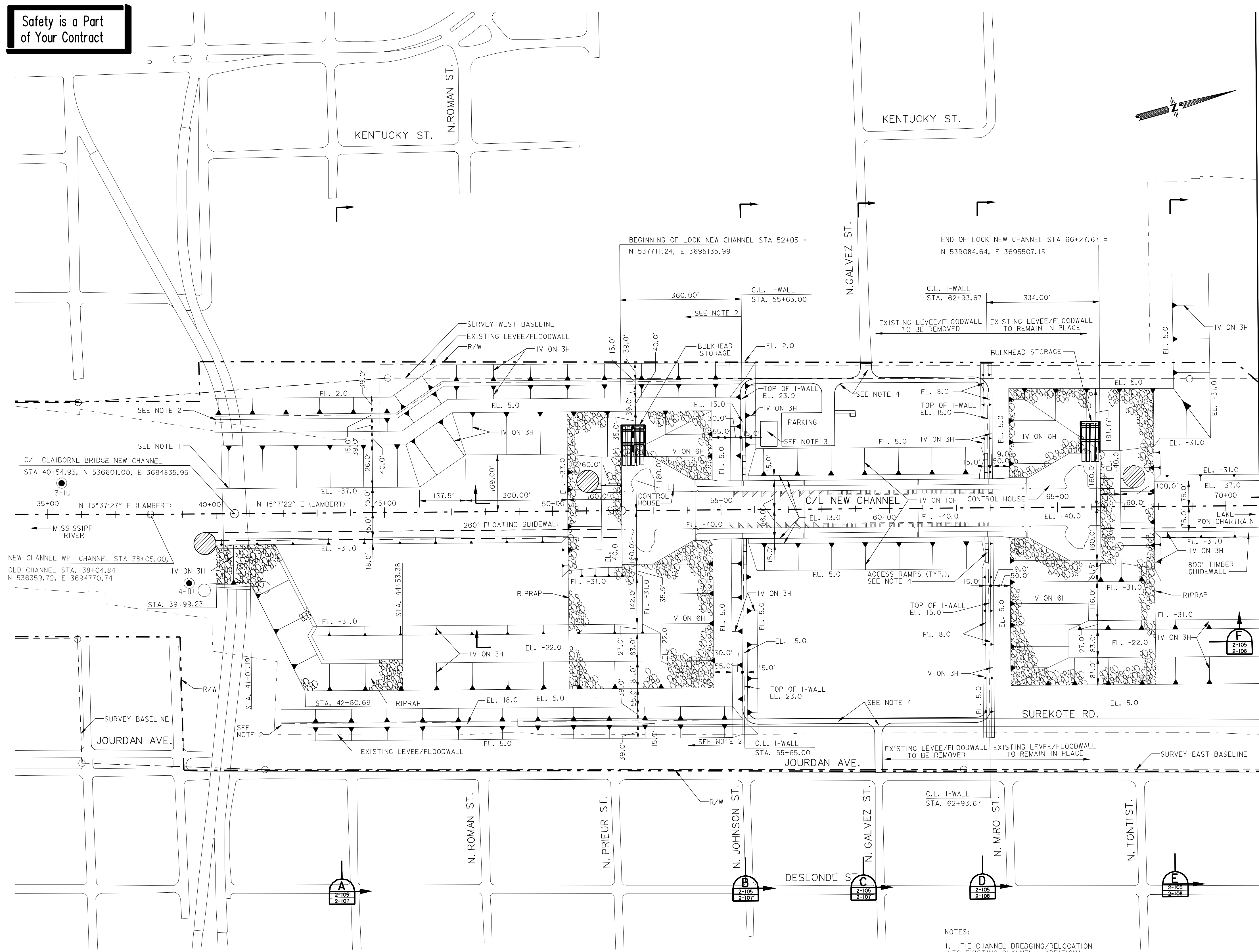

COMPLETED SITE PLAN (1 OF 2)

SCALE: 1" = 100'

This drawing has been reduced to half size

NOTES:

1. TIE CHANNEL DREDGING/RELOCATION INTO EXISTING CHANNEL. ADDITIONAL CHANNEL DREDGING/RELOCATION WILL BE INCLUDED IN A FUTURE COE CONTRACT.

2. LEVEE RELOCATION TO BE IN SEPARATE COE CONTRACT. SEE LEVEE RELOCATION CONTRACT DRAWINGS FOR CONTINUATION OF LEVEE.

3. 50'X75' PRE-FABRICATED MAINTENANCE & ADMINISTRATION BUILDING. FINAL LOCATION AND LAYOUT BY COE.

4. PRELIMINARY LAYOUT OF SITE ROADS. FINAL LOCATION AND LAYOUT BY COE.

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U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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NEW ORLEANS, LOUISIANA

DESIGNED BY: L. WEAVER CHECKED BY: RAUH DRAWN BY: L. WEAVER DATE: JUNE 2006	PLOT SCALE: 1200	PLOT DATE: JUNE 2006	DESIGN FILE NAME: IHNC-2-105.DGN SUBMITTED BY: DESIGN ENGINEER SOLICITATION NO. W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA
COMPLETED SITE PLAN
SHEET 1 OF 2

FILE NUMBER
H-2-46087
DWG. 2-105

EXHIBIT COM-5

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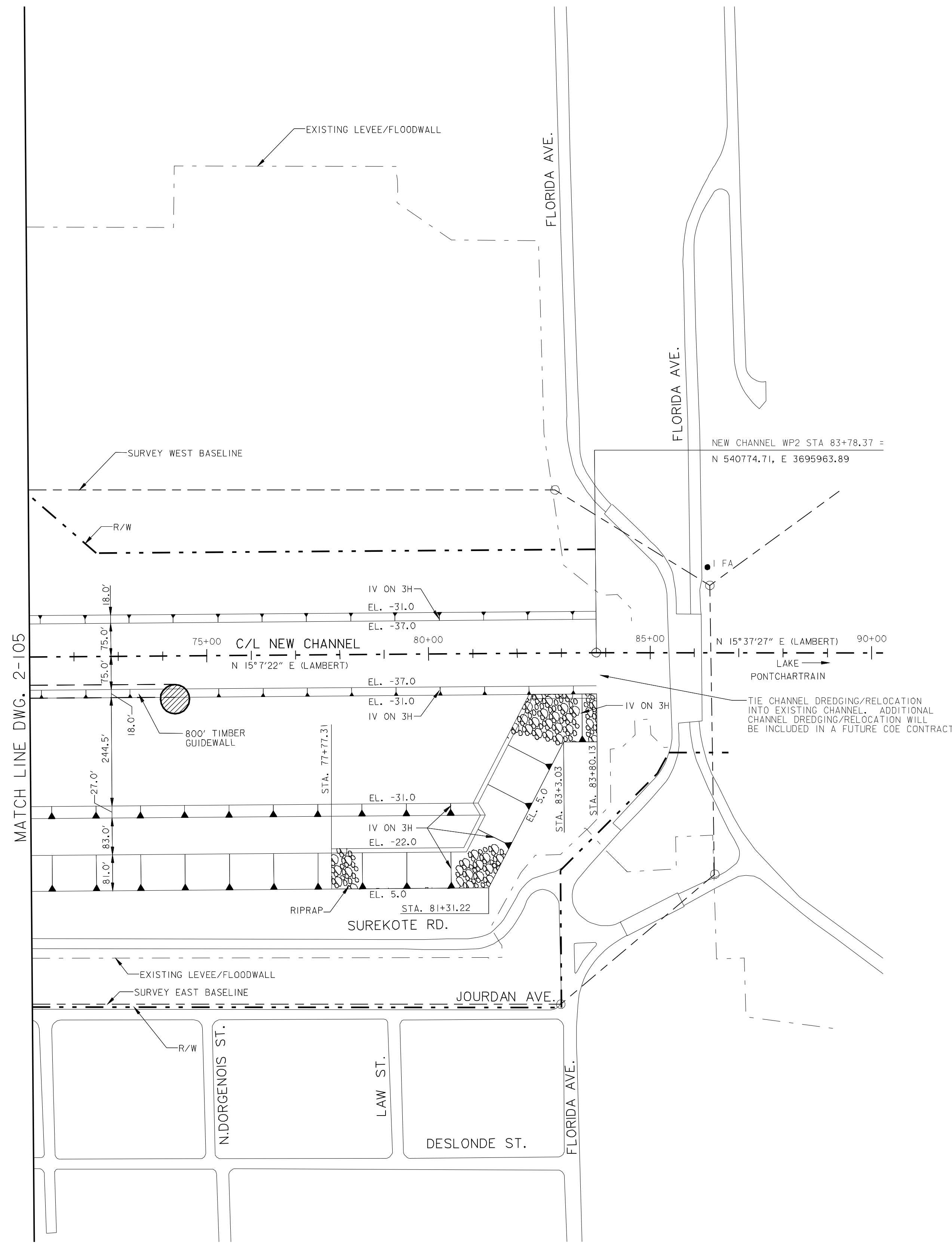

COMPLETED SITE PLAN (2 OF 2)

SCALE: 1" = 100'

100' 0 100' 200' 300' 400'

This drawing has been reduced to half size

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U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: L. WEAVER CHECKED BY: RAUH DRAWN BY: L. WEAVER DATE: JUNE 2006	PLOT SCALE: 1200	PLOT DATE: JUNE 2006	DESIGN FILE NAME: IHNC-2-106.DGN SUBMITTED BY: DESIGN ENGINEER	SOLICITATION NO. W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

COMPLETED SITE PLAN
SHEET 2 OF 2

FILE NUMBER H-2-46087
DWG. 2-106

EXHIBIT COM-6

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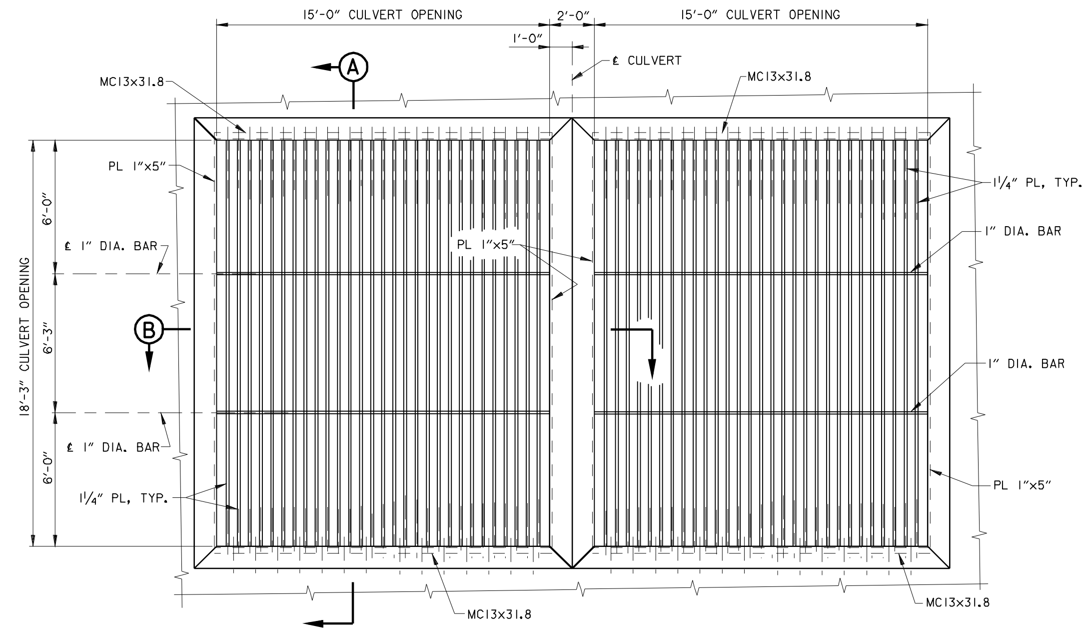
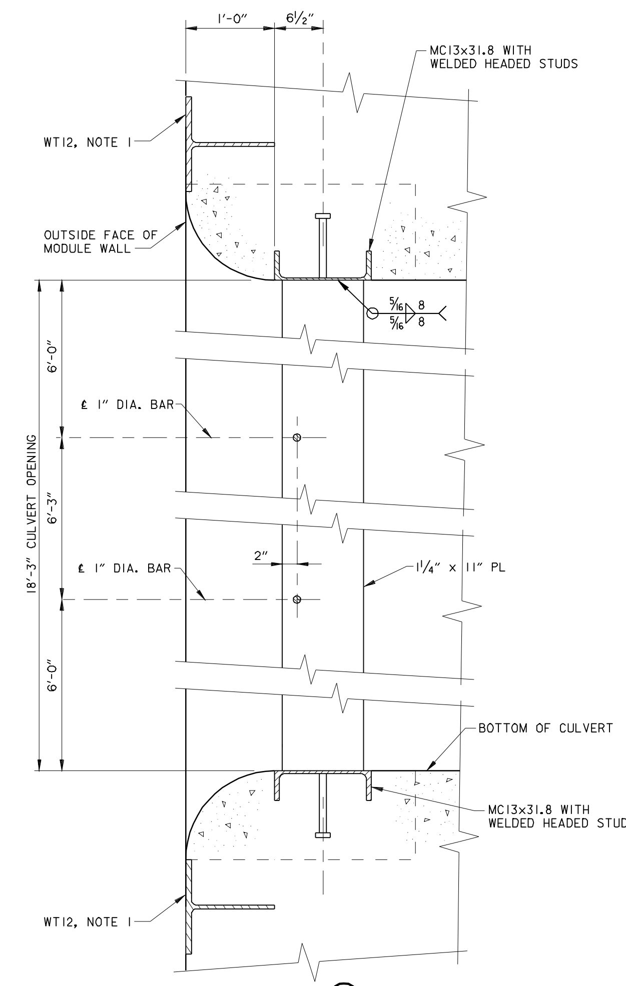
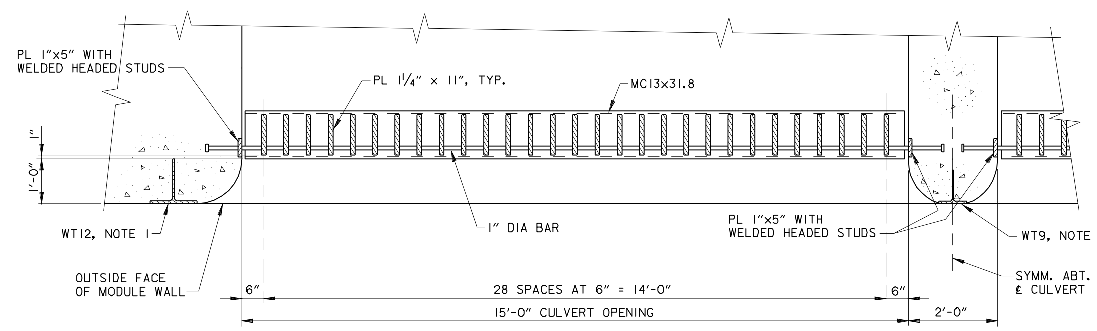


ELEVATION
CULVERT INTAKE SCREEN
SCALE: ⅜" = 1'-0"



SECTION A
SCALE: 1 ½" = 1'-0"



SECTION B
SCALE: ¾" = 1'-0"

NOTE 1:
FOR EMBEDDED METAL, SEE DRAWING 10-8.

This drawing has been reduced to half size

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US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: M. SUPAK CHECKED BY: G. STAUDER DRAWN BY: S. MORR DATE: JUNE 2006	PLOT SCALE: 32	PLOT DATE: 6/2006	DESIGN FILE NAME: IHNC-7-8.DGN SUBMITTED BY: DESIGN ENGINEER	SOLICITATION NO. W912P8

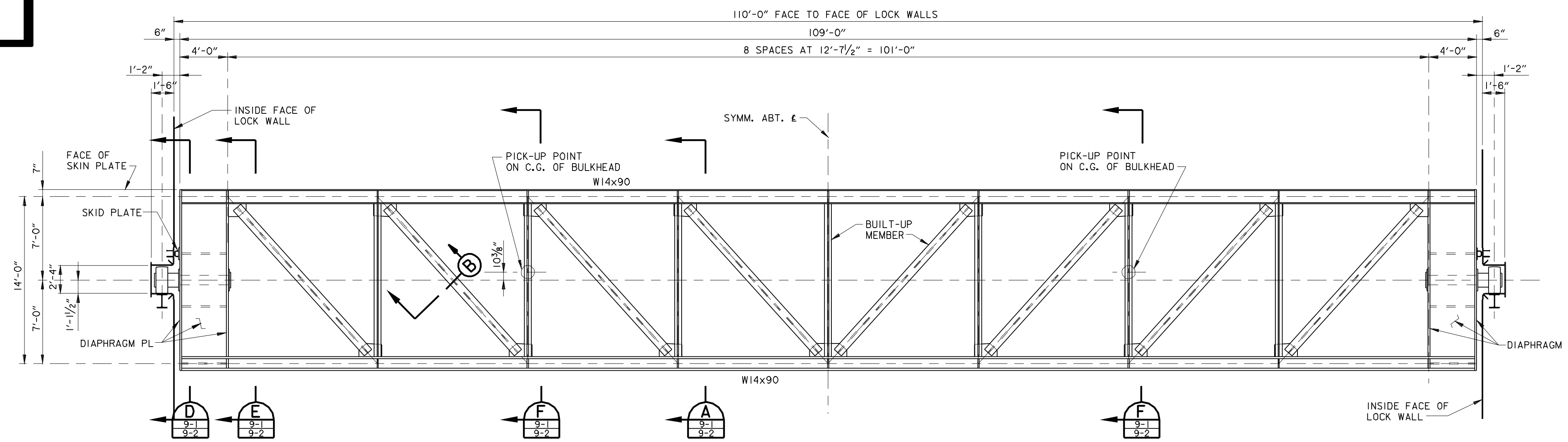
MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

MISCELLANEOUS METALS
CULVERT INTAKE SCREENS

FILE NUMBER H-2-46087
DWG. 7-8

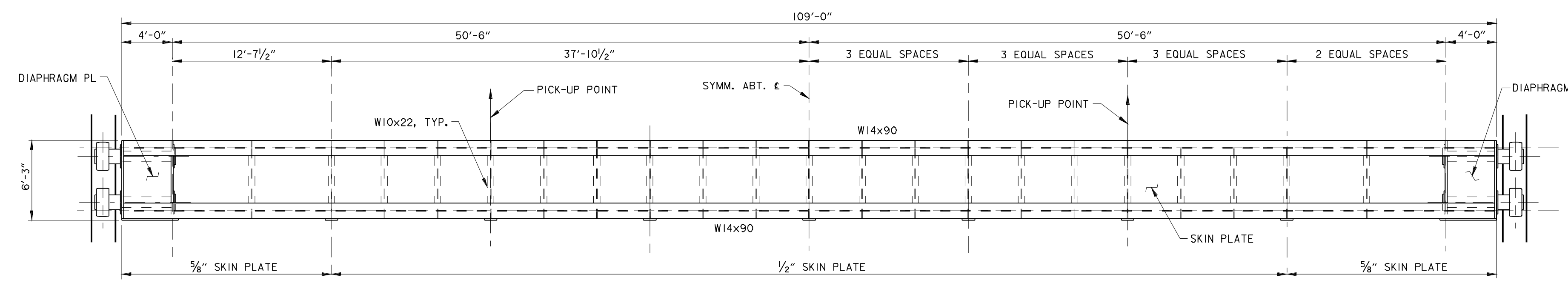
EXHIBIT COM-7

Safety is a Part of Your Contract



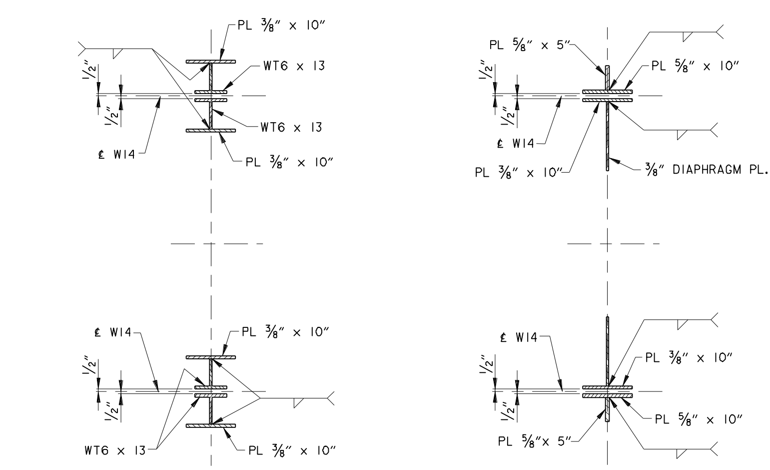
PLAN

SCALE: ¼" = 1'- 0"



BULKHEAD WET SIDE (SKIN PLATE) ELEVATION (UPPER UNIT)

SCALE: ¼"=1'-0"

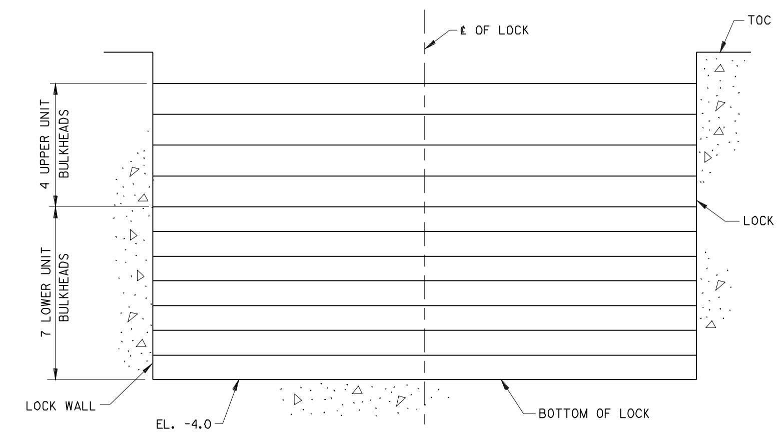


SECTION B

SCALE: 1" = 1'- 0"

SECTION C 9-2 9-1

SCALE: 1"=1'-0"



BULKHEAD LOCATION DIAGRAM

NOTE:
LOWER UNIT BULKHEAD MAX. DESIGN HEAD = 58'-0"
UPPER UNIT BULKHEAD MAX. DESIGN HEAD = 25'-0"
PROVIDE TWO SETS OF BULKHEADS. ONE SET OF BULKHEADS SHOWN, ONE ADDITIONAL SET REQUIRED FOR A TOTAL OF 8 UPPER UNITS AND 14 LOWER UNITS.

DESIGN DATA

CLASS C HYDRAULIC STEEL STRUCTURE (EM 1110-2-2105, CHANGE 1, 31 MAY 94)

UPPER UNIT

DEAD LOAD	SELFWEIGHT (140 KIPS)
HYDROSTATIC LOAD:	25' HEAD
IMPACT:	100 KIPS AT CENTERLINE
COMBINED LOAD:	0.5 X HYDROSTATIC + 1.0 X IMPACT 1.0 X DL + 1.0 X HYDROSTATIC 1.0 X DL + 1.0 X HYDROSTATIC + 1.0 X TRANSPORT
ALLOWABLE STRESS:	UNUSUAL DEWATERING: 1.33 X NORMAL AISC ALLOWABLES MAINTENANCE: 1.16 X NORMAL AISC ALLOWABLES TRANSPORT: 0.83 X NORMAL AISC ALLOWABLES

This drawing has been reduced to half size

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JULY 2006

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PROJECT No. 10000586.00000

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: C. MATOS
CHECKED BY: C. MATOS
DRAWN BY: S. HARR
DATE: JUNE 2006
PLOT SCALE: 48
PLOT DATE: 6/2006
SOLICITATION NO.: W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

MAINTENANCE BULKHEAD
UPPER UNIT- PLAN & ELEV.

FILE NUMBER H-2-46087
DWG. 9-1

EXHIBIT COM-9

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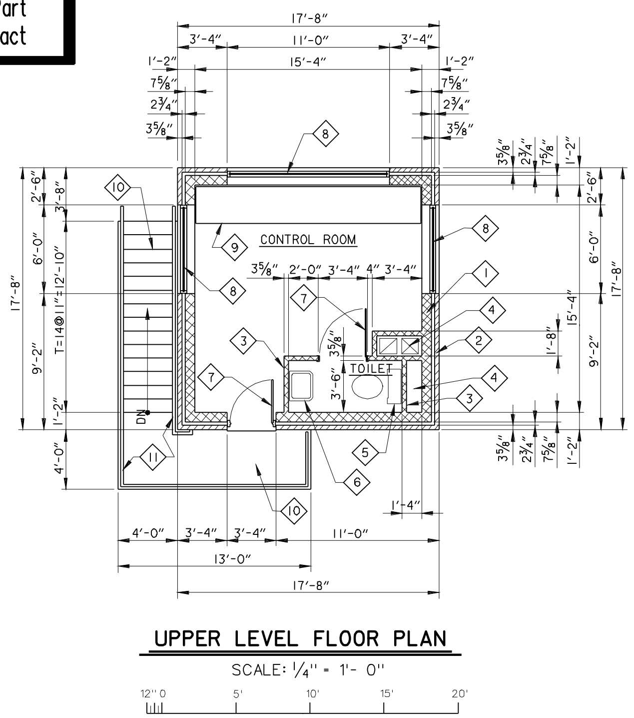
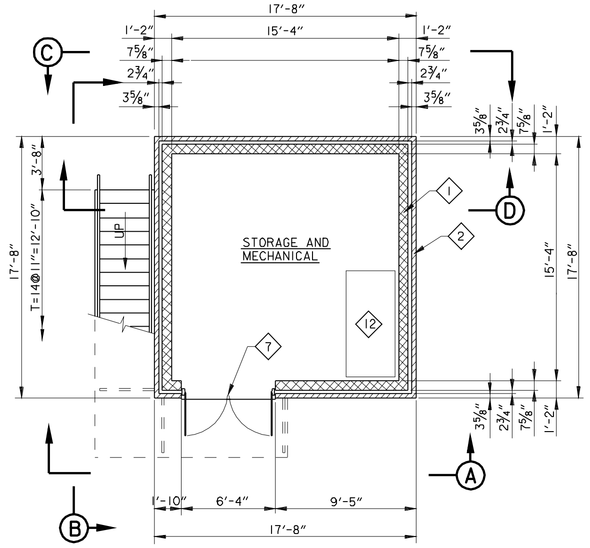


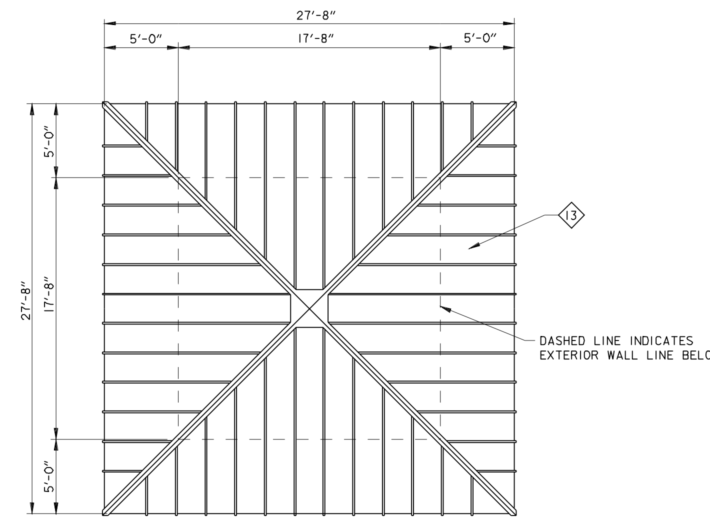
UPPER LEVEL FLOOR PLAN

SCALE: 1/4" = 1'- 0"



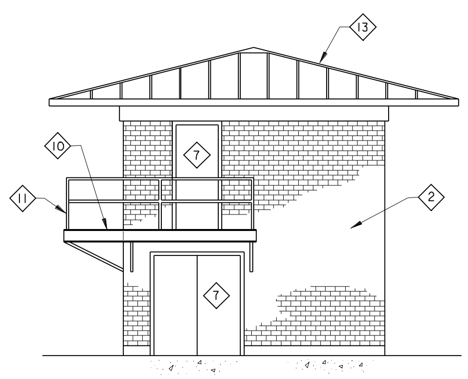
LOWER LEVEL FLOOR PLAN

SCALE: 1/4"=1'-0"



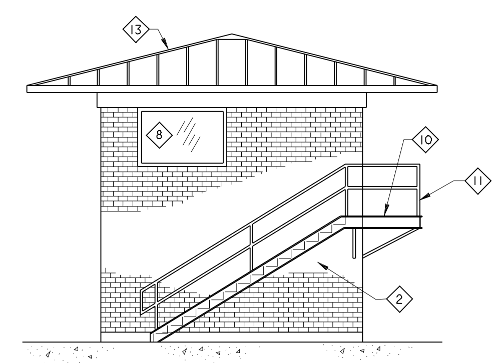
ROOF PLAN

SCALE: 1/4"=1'-0"



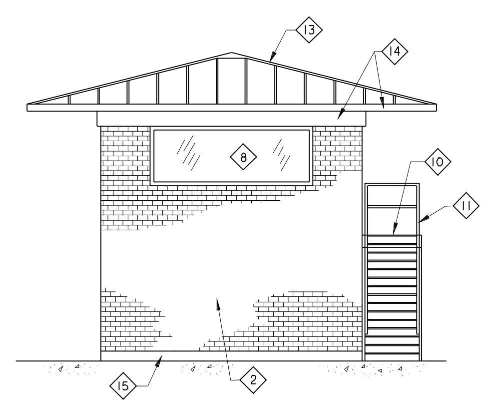
SECTION (A)

SCALE: 1/4"=1'-0"



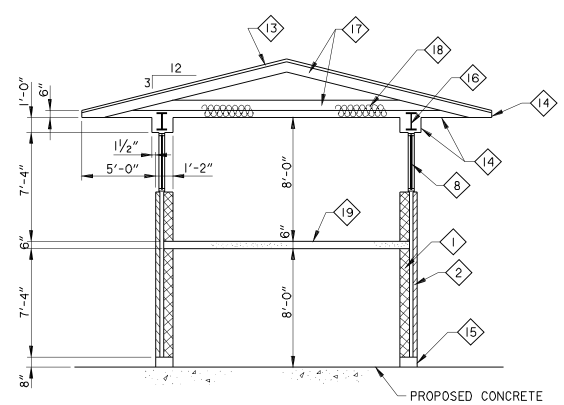
SECTION (B)

SCALE: 1/4"=1'-0"



SECTION (C)

SCALE: 1/4"=1'-0"



SECTION (D)

SCALE: 1/4"=1'-0"

KEY NOTES:

1. CONCRETE MASONRY (CMU) LOAD BEARING WALLS: 8" (Nom) THICK, NORMAL WEIGHT UNITS WITH REINFORCED HORIZONTAL JOINTS AT 16" O.C. VERTICAL REINFORCMENT SHALL BE NO. 5 STEEL REBAR IN GROUT-FILLED CORES AT 24" O.C. AND AT ALL CORNER AND WINDOW/DOOR JAMB CONDITIONS.
2. BRICK VENEER: 4" (Nom) THICK, CLAY BRICK MASONRY VENEER ANHORED TO CMU AND CONCRETE BACKUP BY STAINLESS STEEL ANCHORS. BRICK SHALL BE COMERCIAL GRADE, COLOR AS SELECTED BY ARCHITECT. THE BRICK VENEER SHALL UTILIZE THRU-WALL FLASHING AT THE BASE OF THE WALL CONSISTING OF ADHESIVE-BACKED EPDM TYPE FLASHING WITH STAINLESS STEEL DRIP EDGES EXPOSED TO EXTERIOR. THE AIR GAP BETWEEN THE CMU/CONCRETE BACKING AND THE BRICK VENEER SHALL RECEIVE A 1" LAYER OF RIGID INSULATION ADHERED TO THE BACKUP.
3. CONCRETE MASONRY (CMU) PARITIONS: 4" (Nom) THICK NORMAL WEIGHT UNITS WITH REINFORCED HORIZONTAL JOINTS AT 24" O.C. VERTICAL REINFORCMENT SHALL BE NO. 4 STEEL REBAR IN GROUT-FILLED CORES AT 24" O.C. AND AT ALL CORNER AND WINDOW/DOOR JAMB CONDITIONS.
4. CHASE FOR HVAC AND ELECTRICAL SYSTEMS.
5. WATER CLOSET: PORCELAIN, WALL-MOUNTED, FLUSH VALVE.
6. SINK: PORCELAIN, WALL-MOUNTED.
7. STEEL DOOR AND FRAME: OPENINGS - 3'-0" x7'-0". MATERIAL SHALL BE HEAVY-DUTY, GALVANIZED STEEL - TYPICAL, ALL DOORS AND FRAMES SHALL BE PAINTED (HIGH-PERFORMANCE GLOSS). EXTERIOR DOORS SHALL BE INSULATED. DOOR HARDWARE WILL BE HEAVY-DUTY, STAINLES STEEL TO THE GREATEST EXTENT POSSIBLE.
8. ALUMINUM FRAME WINDOW: THERMALLY BROKEN, 4 1/2" DEEP, ALUMINUM FRAME GLAZED STOREFRONT SYSTEM. GLASS SHALL BE 1" THICK, TEMPERED, TINTED, INSULATED WITH LOW-E COATING.
9. COUNTERTOP AND BASE CABINET: SOLID SURFACING MATERIAL COUNTERTOP ON PLASTIC LAMINATE BASE CABINETS. CONFIGURATION WILL BE CUSTOM DESIGNED TO ACCOMMODATE LOCK CONTROL PANEL EQUIPMENT.
10. STEEL GRATE LANDING AND STAIR TREADS: GRATING SHALL BE GALVANIZED (G90) WITH A SLIP-RESISTANT SURFACE. GRATES WILL BE WELDED TO STEEL SUPPORTING FRAME. ALL EXTERIOR GALVANIZED STEEL WILL BE PAINTED WITH A HIGH-PERFORMANCE, ACRYLIC COATING.
11. STEEL HANDRAIL (GUARDRAIL): 1 1/2" DIAMETER, GALVANIZED (G90) PIPE RAILING. ALL CONNECTIONS ARE TO BE WELDED. GUARDRAILS SHALL BE 42" HIGH AND HANDRAILS SHALL BE 34" HIGH FROM ADJACENT WALKING SURFACE.
12. HVAC EQUIPMENT: AIR-CONDITIONER AND HEATER THERMOSTATICALLY CONTROLLED FROM INSIDE THE CONTROL ROOM.
13. METAL, STANDING SEAM ROOF: 22-GAGE, METALLIC-COATED ("GALVALUM") STEEL SHEET PREPAINTED WITH COIL COATING. THE FINISH SHALL BE A FLUOROPOLYMER, THREE-COAT COLOR SYSTEM. THE METAL ROOF SHALL BE INSTALLED OVER AN ADHESIVE-BACKED WATER BARRIER MEMBRANE APPLIED TO 3/4" PLYWOOD ROOF DECKING.
14. SHEET METAL FASCIA AND TRIM: FINISHED TO MATCH THE METAL ROOFING, THIS MATERIAL SHALL BE PAINTED SHEET STEEL AND WILL INCLUDE FASCIA TRIM, SOFFIT PANELS (VENTED), DRIP EDGES AND COUNTER FLASHING.
15. CONCRETE CURB: POURED-IN-PLACE CONCRETE CURB DIRECTLY ON THE EXISTING CONCRETE LOCK STRUCTURE. CONCRETE SHALL BE REINFORCED WITH STEEL REBAR AND IS TO BE DOWEL-CONNECTED TO THE EXISTING LOCK BASE. A WATER BARRIER (WATER-STOP SEAL) WILL BE CAST INTO THE CURB AT THE PROPOSED BASE STRUCTURE.
16. STEEL LINTEL.
17. STEEL ROOF FRAMING: 16-GAGE, COLD-FORMED STEEL FRAMING THAT CREATES A "HIP" ROOF STRUCTURE.
18. FIBERGLASS THERMAL BATT INSULATION (WITH POLY VAPOR BARRIER).
19. 6" THICK, COMPOSITE CONCRETE FLOOR SLAB BEARING ON 8" CMU WALLS.

GENERAL NOTES:

1 THIS FACILITY IS ASSUMED TO BE EXEMPT FROM HANDICAP ACCESSABILITY REQUIREMENTS.

2 THIS FACILITY WILL BE DESIGNED TO WITHSTAND HIGH, HURRICANE FORCE WINDS.

3 BUILDING MATERIAL IN GENERAL WILL BE OF A TYPE THAT RESISTS THE HIGH TEMPERATURE AND HIGH HUMIDITY CONDITIONS OF THE SITE.

4 CONTROL ROOM:
A. FINISHES: CMU WALLS TO BE PAINTED (SEMI-GLOSS OR EGGSHELL). FLOOR SHALL BE VINYL COMPOSITION TILE WITH TILE RUBBER COVE BASE. CEILING SHALL BE SUSPENDED GYPSUM BOARD - PAINTED.

5 TOILET ROOM:
A. FINISHES: CMU WALLS TO BE PAINTED (GLOSS). FLOOR SHALL BE CERAMIC MOSAIC TILE WITH TILE BASE. CEILING SHALL BE SUSPENDED GYPSUM BOARD - PAINTED.
B. TOILET ACCESSORIES SHALL BE PROVIDED - TISSUE DISPENSOR, PAPER TOWEL DISPENSOR, MIRROR AND SHELF, SOAP DISPENSOR, COAT HOOK.
C. A FLOOR DRAIN WILL BE PROVIDED IN THIS SPACE.

6 STORAGE AND MECHANICAL ROOM: CMU WALLS TO BE PAINTED (SEMI-GLOSS OR EGGSHELL). FLOORING - WHICH IS THE TOP OF THE EXISTING CONCRETE LOCK PLATFORM - SHALL BE CLEANED AND SEALED. CEILING - THE UNDERSIDE OF THE COMPOSITE CONCRETE DECK - SHALL BE PAINTED.

This drawing has been reduced to half size

C·A·D

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JULY 2006

JACOBS CIVIL INC.

URS 3500 N. Causeway Blvd., Suite 900 Metairie, Louisiana 70002 (504) 837-6326

PROJECT No. 10000586.00000

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: B. HENDON
CHECKED BY: B. HENDON
DRAWN BY: S. MORR
DATE: JUNE 2006

PLOT SCALE: 48
PLOT DATE: 6/2006
DESIGN FILE NAME: IHNC-6-1.DGN
SUBMITTED BY: DESIGN ENGINEER
SOLICITATION NO. W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA
CONTROL HOUSE
FLOOR PLANS & ELEVATIONS

FILE NUMBER H-2-46087
DWG. 6-1

EXHIBIT COM-10

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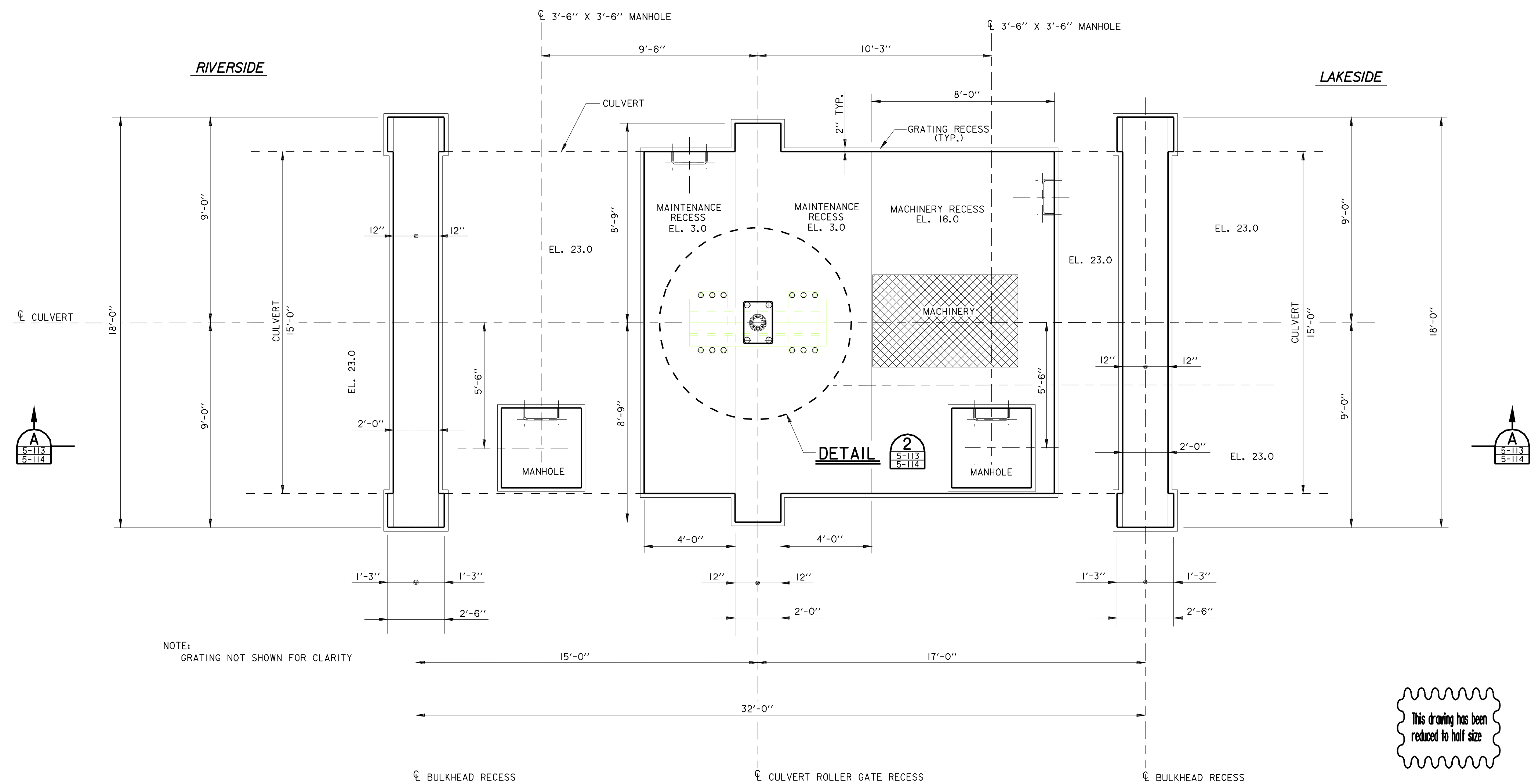

NOTE:
GRATING NOT SHOWN FOR CLARITY

This drawing has been reduced to half size

PLAN
ROLLER GATE RECESS

SCALE: 1/2" = 1'- 0"

12" 0 2' 4' 6' 8'

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Metairie, Louisiana 70002
(504) 837-6326
PROJECT No. 10000586.00000

US Army Corps of Engineers
New Orleans District

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: R.W.Y.
CHECKED BY: J.D.W.
DRAWN BY: B.H.D.
DATE: SEPT. 2005
PLOT SCALE: 24
PLOT DATE: JUNE 2006
DESIGN FILE NAME: RGB5-113.DGN
SUBMITTED BY: DESIGN ENGINEER
SOLICITATION NO. W912P8-

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

RIVERSIDE GATEBAY MODULE
ROLLER GATE RECESS PLAN

FILE NUMBER H-2-46087
DWG. 5-113

EXHIBIT COM-11

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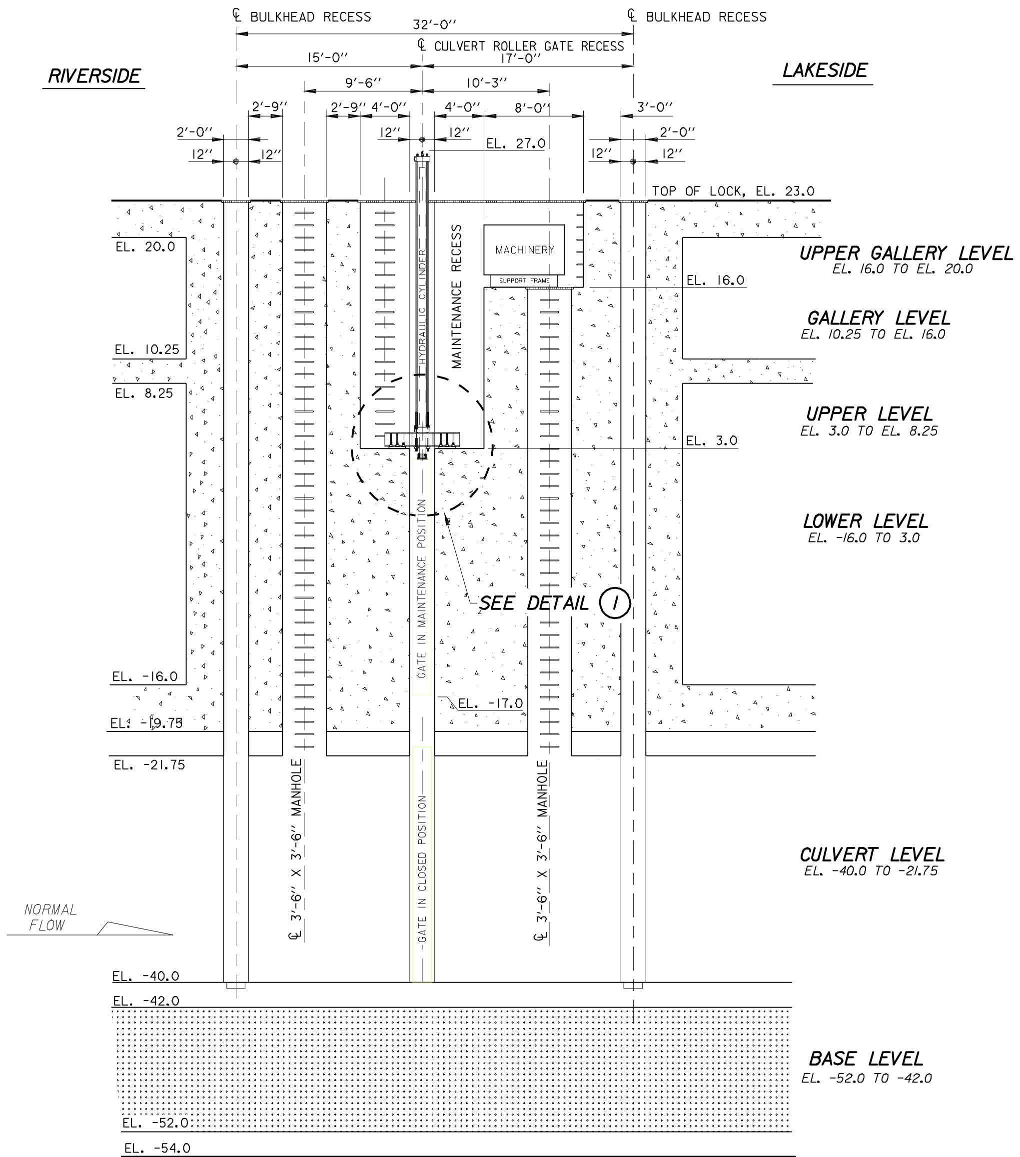
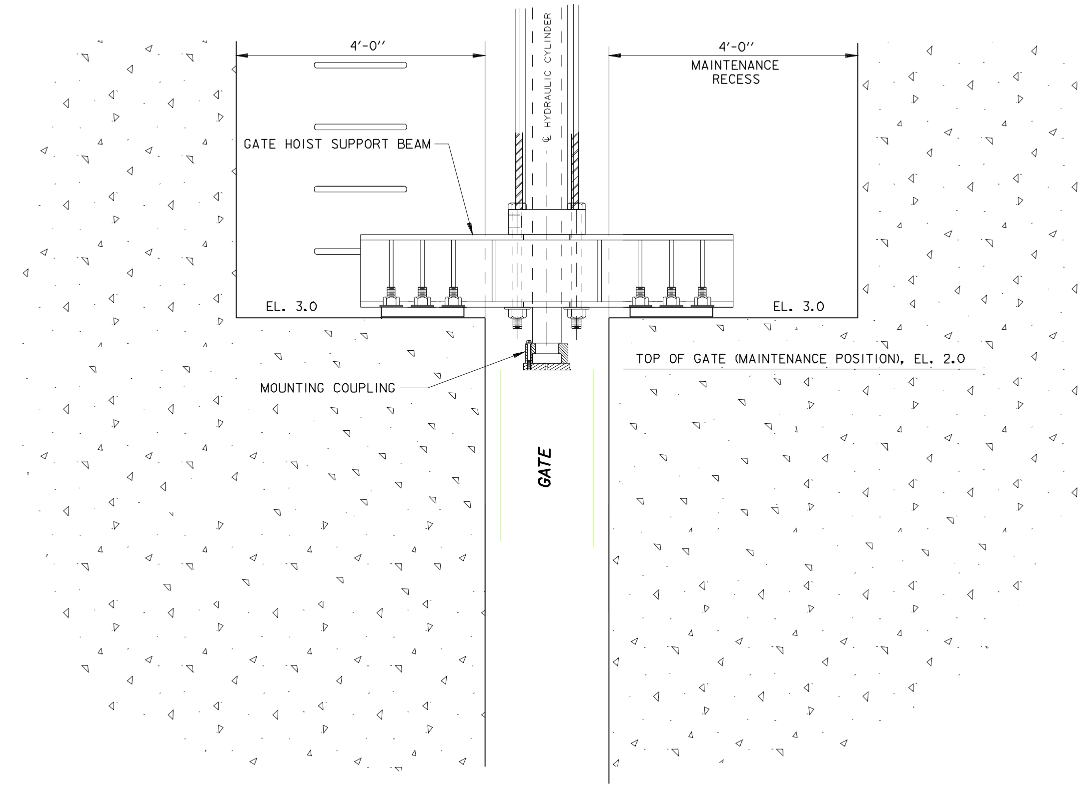


SECTION THRU CULVERT ROLLER GATE RECESS

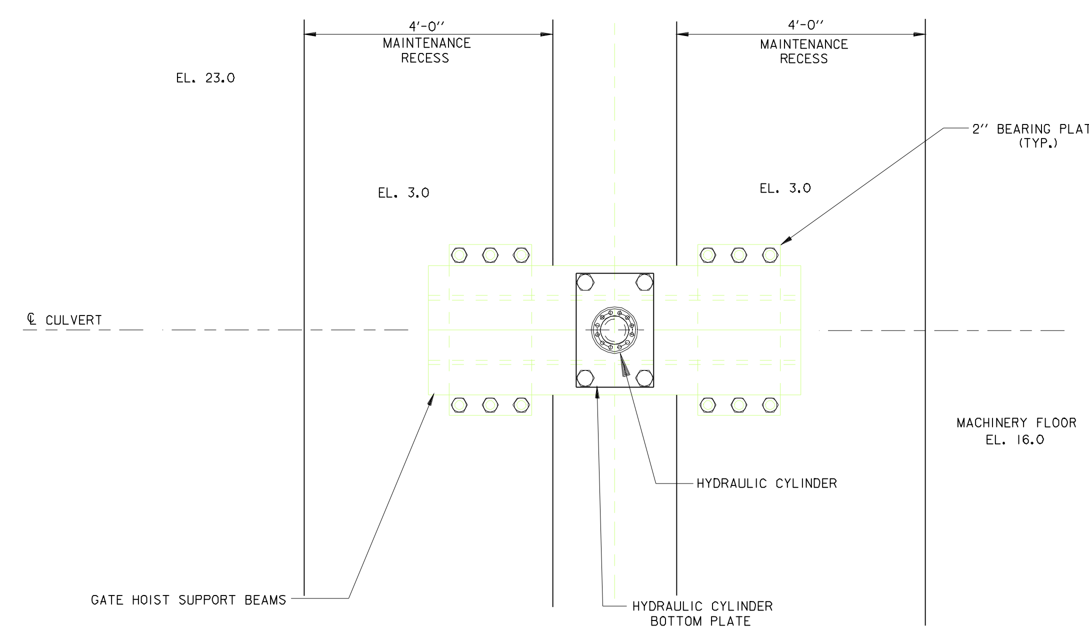
SECTION A 5-113 5-114

SCALE: 1" = 5'



DETAIL 1

SCALE: 1" = 1'-0"



NOTE:
GRATING NOT SHOWN FOR CLARITY.

DETAIL 2 5-113 5-114

SCALE: 1" = 1'-0"

LEGEND

FLOAT-IN SHELL CONCRETE (5000 PSI)

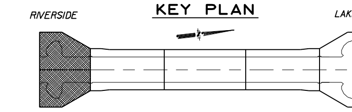
CONCRETE (4000 PSI)

INFILL CONCRETE (3000 PSI)

SCALE: 1" = 5'

SCALE: 1" = 1'- 0"

KEY PLAN



This drawing has been reduced to half size

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3500 N. Causeway Blvd., Suite 900
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PROJECT No. 10000586.00000

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: R.W.Y.
CHECKED BY: J.D.W.
DRAWN BY: B.H.D.
DATE: SEPT. 2005

PLOT SCALE: 60
PLOT DATE: JUNE 2006
DESIGN FILE NAME: RGB5-114.DGN
SUBMITTED BY: X DESIGN ENGINEER
SOLICITATION NO. W912P8-

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

RIVERSIDE GATEBAY MODULE
ROLLER GATE RECESS - SECTIONS

FILE NUMBER H-2-46087
DWG. 5-114

EXHIBIT COM-12

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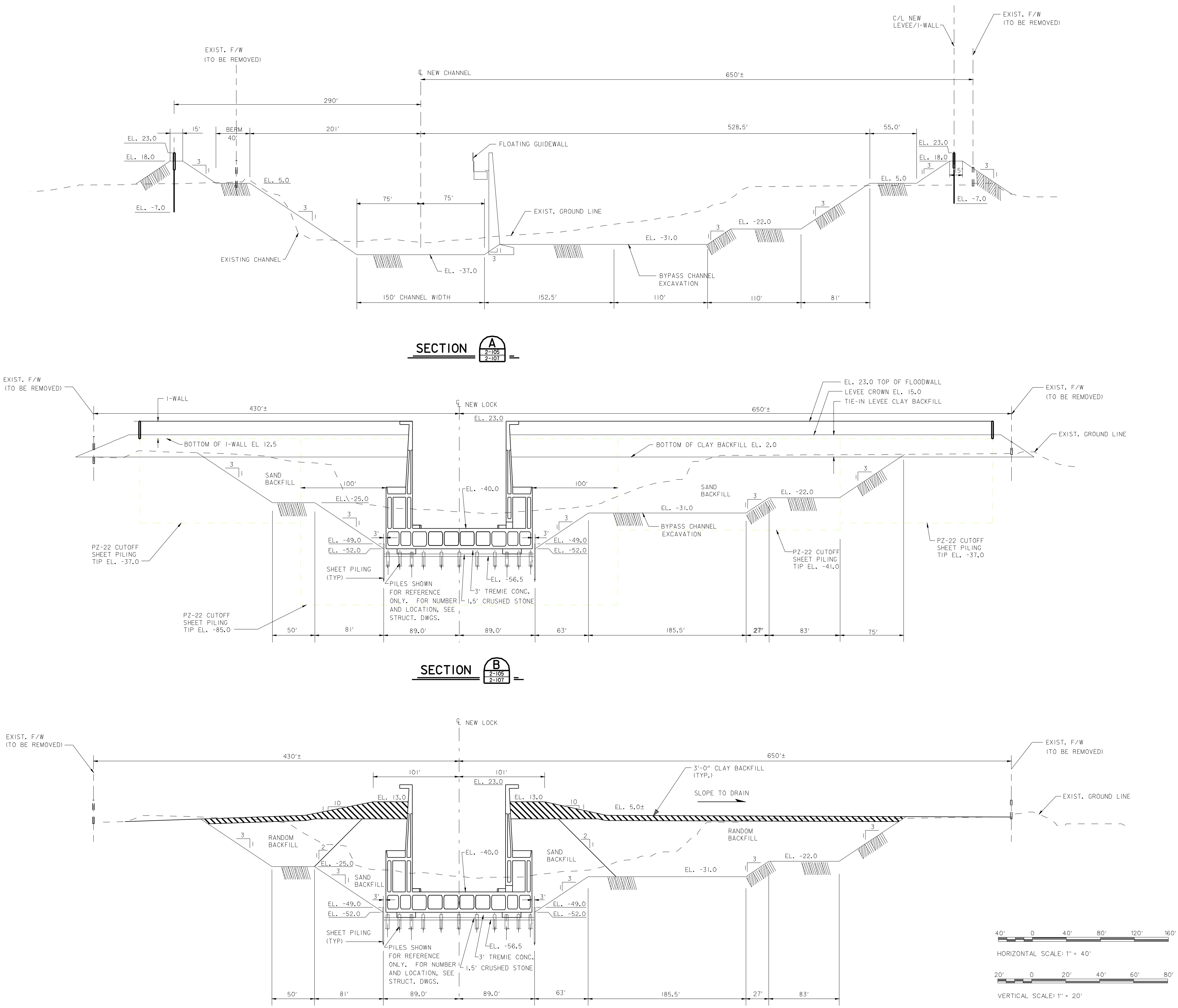

This drawing has been reduced to half size

C-A-D

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Metairie, Louisiana 70002
(504) 837-6326
PROJECT No. 10000586.00000

US Army Corps of Engineers
New Orleans District

MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: HESTERBERG
CHECKED BY: RASH
DRAWN BY: ROSEN
DATE: JUNE 2006
PLOT SCALE: 480
PLOT DATE: JUNE 2006
SOLICITATION NO.: W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

LOCK SITE TYPICAL SECTIONS
SHEET 1 OF 3

FILE NUMBER H-2-46087
DWG. 2-107

EXHIBIT COM-13

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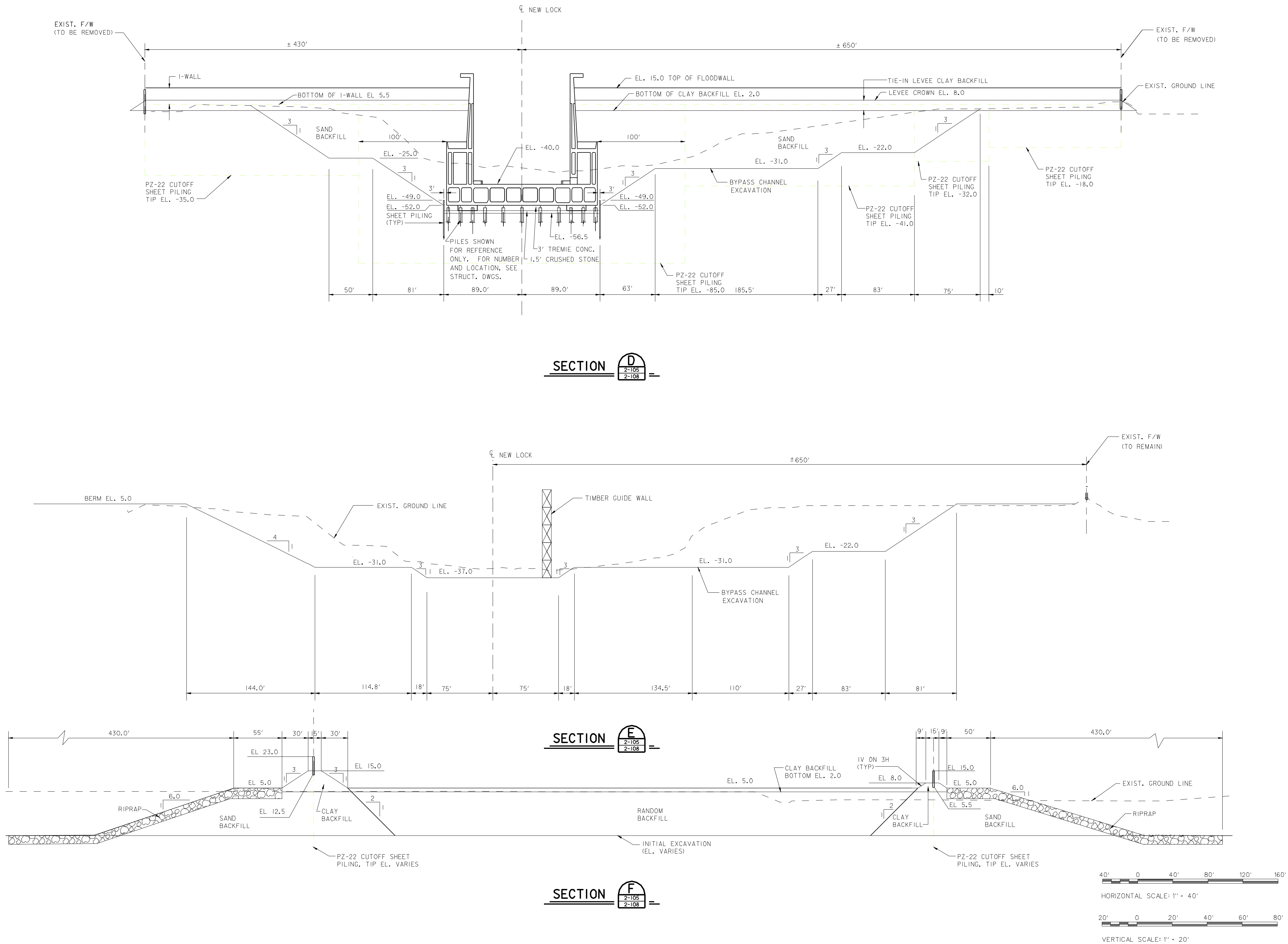

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PROJECT No. 10000586.00000

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: L. WEAVER
CHECKED BY: RAUH
DRAWN BY: ROSEN
DATE: JUNE 2006
PLOT SCALE: 480
PLOT DATE: JUNE 2006
DESIGN FILE NAME: IHNC-2-108.DGN
SUBMITTED BY: DESIGN ENGINEER
SOLICITATION NO. W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

LOCK SITE TYPICAL SECTIONS
SHEET 2 OF 3

FILE NUMBER H-2-46087
DWG. 2-108

EXHIBIT COM-14

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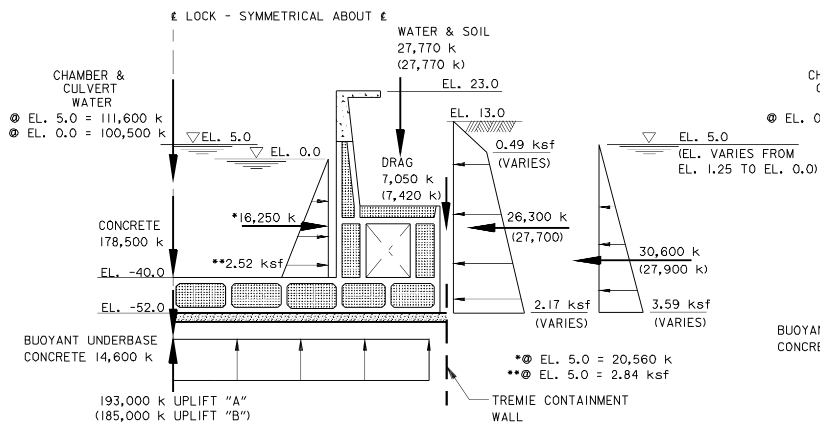
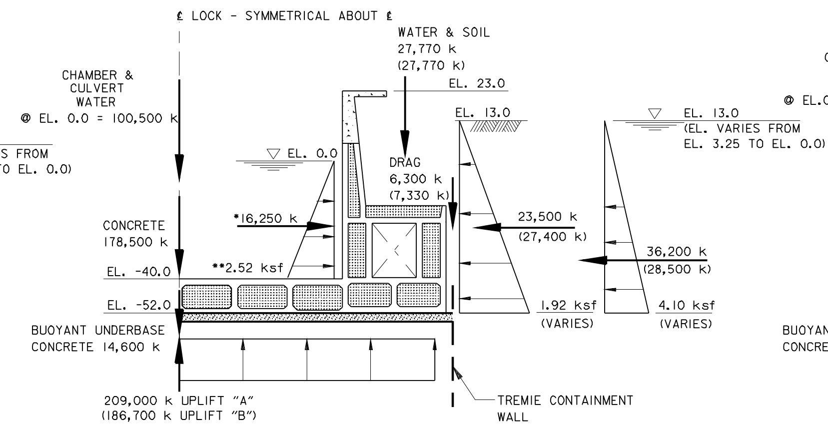


OPERATION-REVERSE HEAD NAVIGATION LIMIT

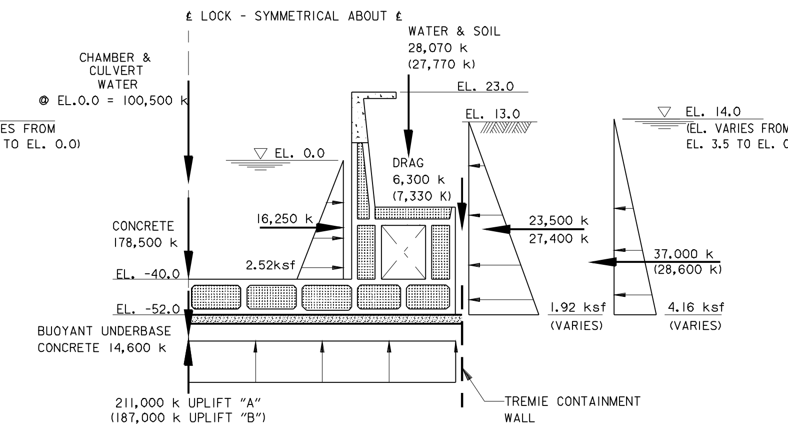
N.T.S.



HURRICANE-MAXIMUM HEAD STILLWATER

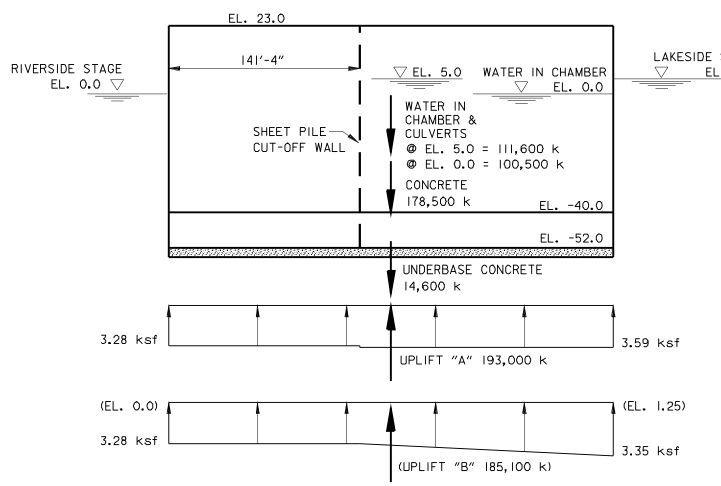
N.T.S.

NOTE:
LOADS NOT SHOWN AT CENTERLINE OF LOCK ARE FOR HALF MODULE.



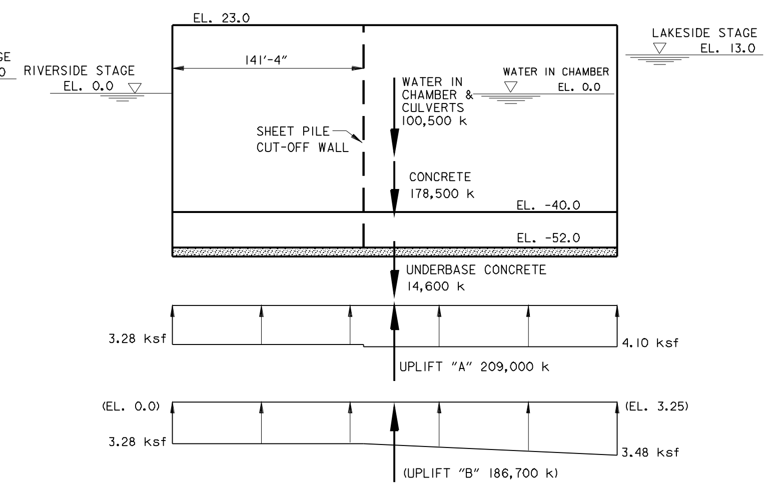
HURRICANE-MAXIMUM HEAD STILLWATER AND FREEBOARD

N.T.S.



OPERATION-REVERSE HEAD NAVIGATION LIMIT

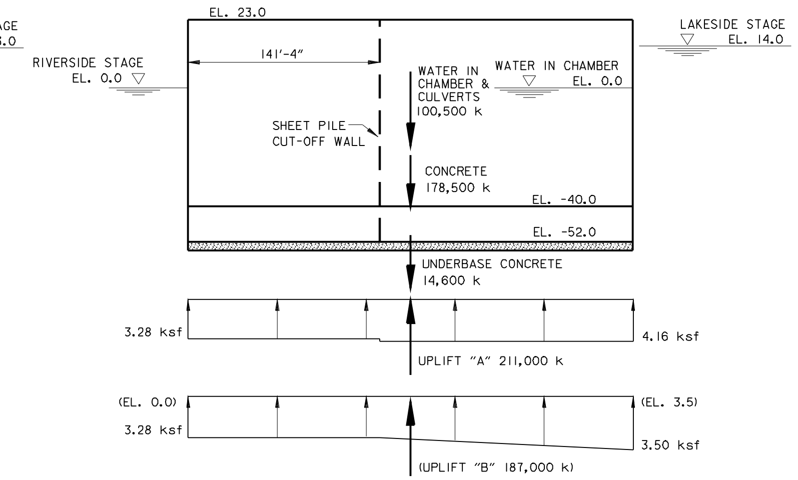
N.T.S.



HURRICANE-MAXIMUM HEAD STILLWATER

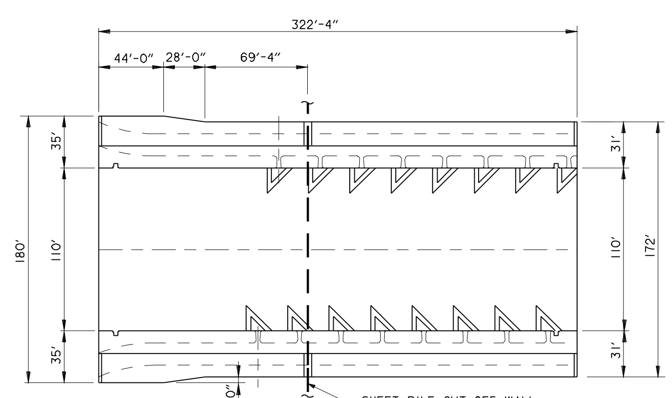
N.T.S.

NOTE:
VALUES ON LONGITUDINAL SECTION ARE FOR ENTIRE MODULE.



HURRICANE-MAXIMUM HEAD STILLWATER AND FREEBOARD

N.T.S.



RIVERSIDE CHAMBER MODULE (CM2)

SCALE: 1" = 40'

NOTES:

1. CASE A: RIVERSIDE SHEET PILE CUT-OFF ASSUMED FULLY EFFECTIVE.
   CASE B: RIVERSIDE SHEET PILE CUT-OFF ASSUMED INEFFECTIVE. SHOWN IN ( )
2. WEIGHT OF BUOYANT UNDERBASE CONCRETE IS ONLY CONSIDERED WITH RESPECT TO PILE LOADS.

SCALE: 1" = 40'

40' 0 40' 80' 120' 160'

This drawing has been reduced to half size

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09/02/05

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PROJECT No. 10000586.00000

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

PLOT SCALE: 20 | PLOT DATE: 9/2/05 | SOLICITATION NO. W912P8-

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

RIVERSIDE CHAMBER MODULE
LOAD DIAGRAMS - REVERSE HEAD AND HURRICANE

FILE NUMBER H-2-46087
DWG. LD-CM2-12

EXHIBIT COM-15

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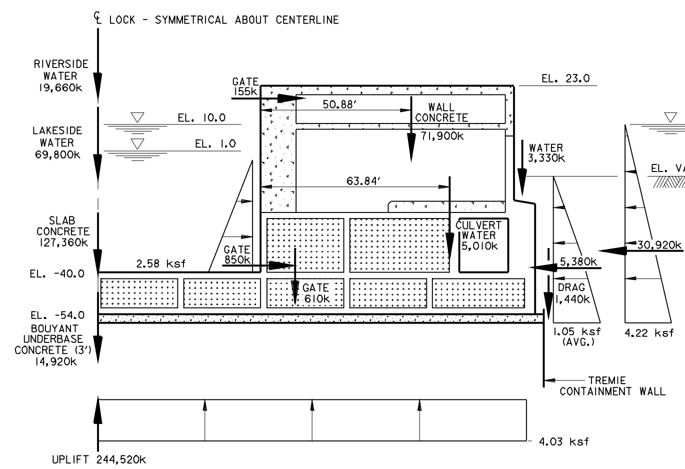
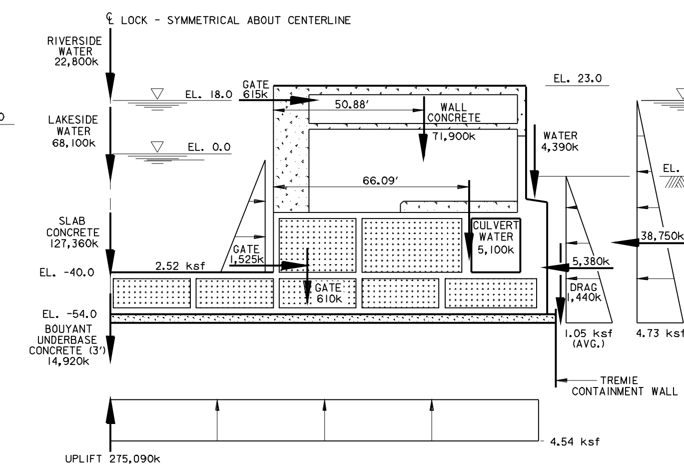


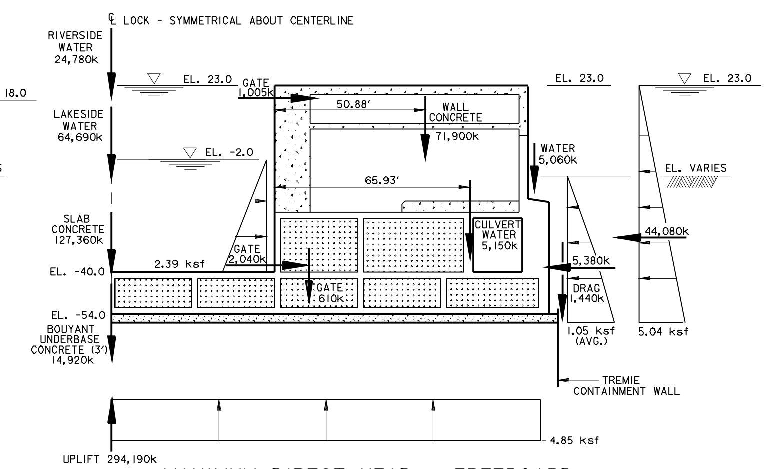
NORMAL OPERATION

RIVER STAGE 10.0 LAKE STAGE 1.0



MAXIMUM DIRECT HEAD

RIVER STAGE 18.0 LAKE STAGE 0.0



MAXIMUM DIRECT HEAD + FREEBOARD

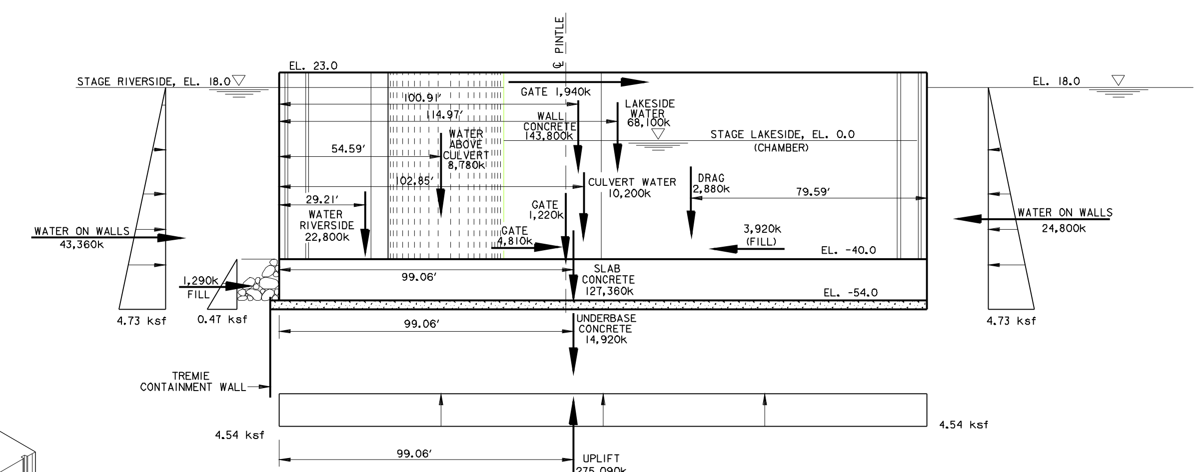
RIVER STAGE 23.0 LAKE STAGE -2.0

NOTE:

UPLIFT PRESSURES TAKEN TO BASE OF LOCK AT EL. -54.0.

HORIZONTAL PRESSURES TAKEN TO BOTTOM OF TREMIE CONCRETE.

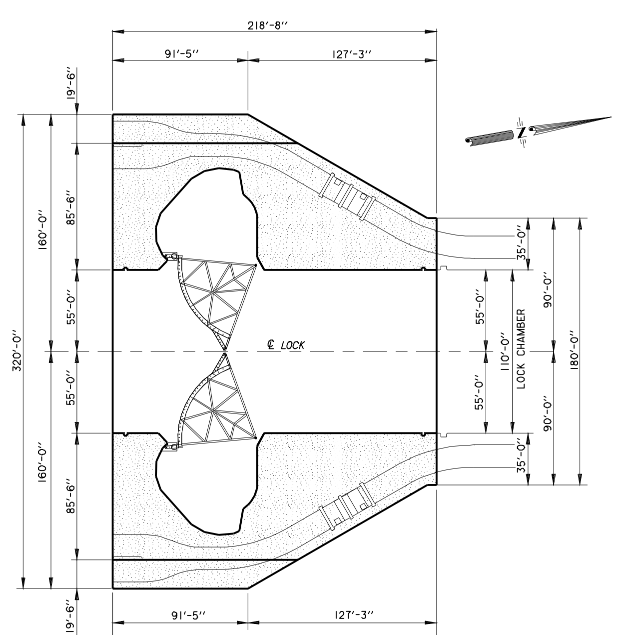
LOADS NOT SHOWN AT CENTERLINE LOCK ARE FOR HALF MODULE.



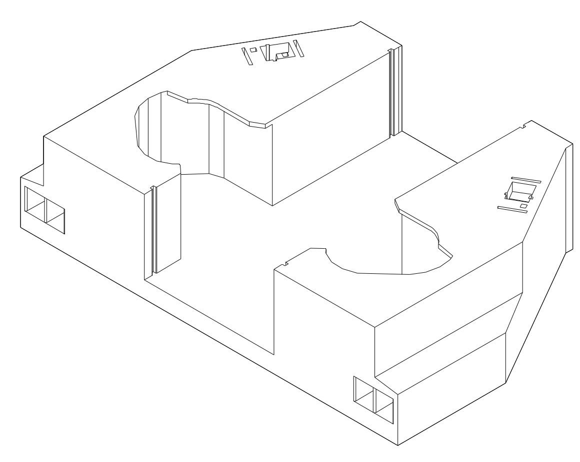
OPERATION - MAXIMUM DIRECT HEAD

RIVER STAGE 18.0 LAKE STAGE 0.0

NOTE: VALUES ON LONGITUDINAL SECTION ARE FOR ENTIRE MODULE.



PLAN



ISOMETRIC VIEW

RIVERSIDE GATEBAY MODULE

SCALE: 1" - 40'

40' 0 40' 80' 120' 160'

LEGEND

FLOAT-IN SHELL CONCRETE (5000 PSI)

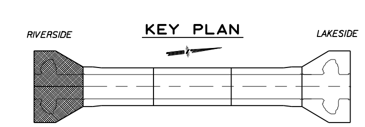
INFILL CONCRETE (3000 PSI)

CONCRETE (4000 PSI)

TREMIE CONCRETE (5000 PSI)

This drawing has been reduced to half size

KEY PLAN



PHASE I DESIGN
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URS 3500 N. Causeway Blvd., Suite 900 Metairie, Louisiana 70002 (504) 837-6326 PROJECT No. 10000586.00000

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS NEW ORLEANS, LOUISIANA

PLOT SCALE: 240 PLOT DATE: 9/2/05 SOLICITATION NO. W912P8-

MISSISSIPPI RIVER-GULF OUTLET NEW LOCK AND CONNECTING CHANNELS INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT ORLEANS PARISH, LOUISIANA

RIVERSIDE GATEBAY MODULE LOAD DIAGRAMS - OPERATING CONDITIONS

FILE NUMBER H-2-46087 DWG. LD-GB1-4

EXHIBIT COM-16

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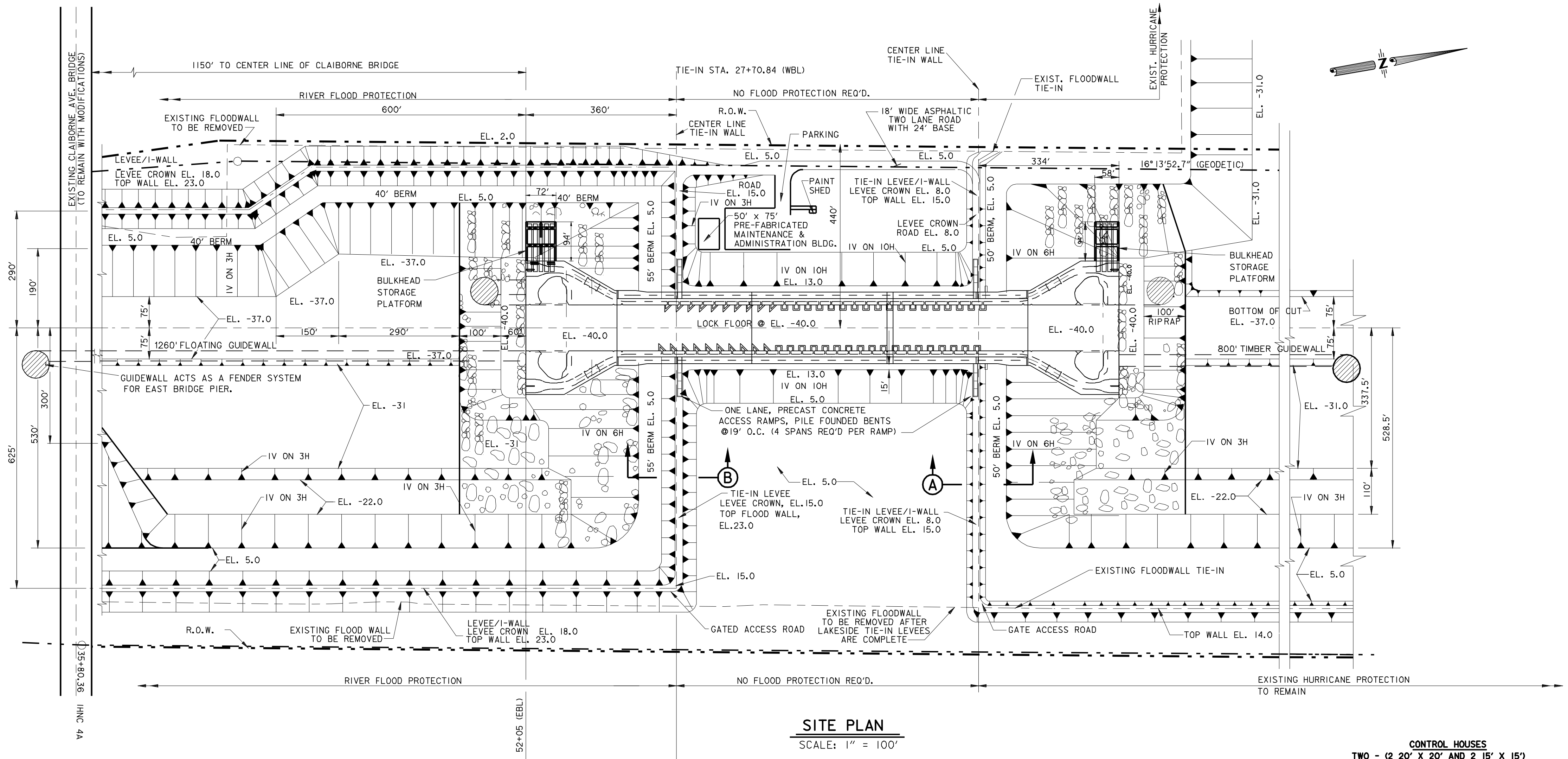
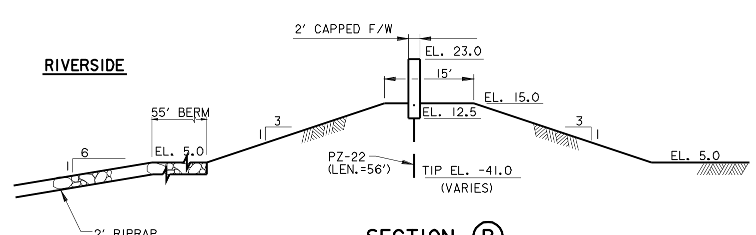


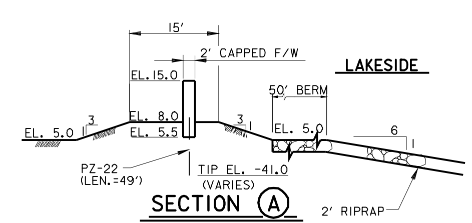
SITE PLAN

SCALE: 1" = 100'



SECTION (B)

SCALE: 1" = 10'



SECTION (A)

SCALE: 1" = 10'

CONTROL HOUSES
TWO - (2 20' X 20' AND 2 15' X 15')

ONE STORY REINFORCED MASONRY BLDG. WITH METAL ROOF SUPPORTED ON OPEN WEB JOISTS. CONTROL HOUSE SHALL INCLUDE STORAGE AREA & RESTROOM FACILITIES.

MAINTENANCE & ADMINISTRATION BLDG.

TWO STORY 50' x 75' PRE-FABRICATED RIGID STEEL FRAME ON A TIMBER PILE FOUNDED 8" SLAB. WALL ENCLOSURES SHALL BE INSULATED 22 GUAGE METAL PANELS. FIRST FLOOR 14'; 2ND FLR 8'.

MACHINERY BUILDING WITH EMERGENCY GENERATORS

25' x 45' CLEAR SPAN MACHINERY BUILDING WHICH INCLUDES EMERGENCY GENERATORS. BLDG. SHALL BE REINFORCED MASONRY LOCATED ON LOCK WALL. METAL ROOF SUPPORTED ON OPEN WEB JOISTS. ONE STORY, 14' CEILING.

SCALE: 1" = 10'

SCALE: 1" = 100'

This drawing has been reduced to half size

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U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: J.C.G.
CHECKED BY: M.A.C.
DRAWN BY: B.M.M.
DATE: 09-02-05

PLOT SCALE:
PLOT DATE: JUNE06
DESIGN FILE NAME: IHNC-1-4.DGN
SUBMITTED BY: X DESIGN ENGINEER
SOLICITATION NO. W912P8-

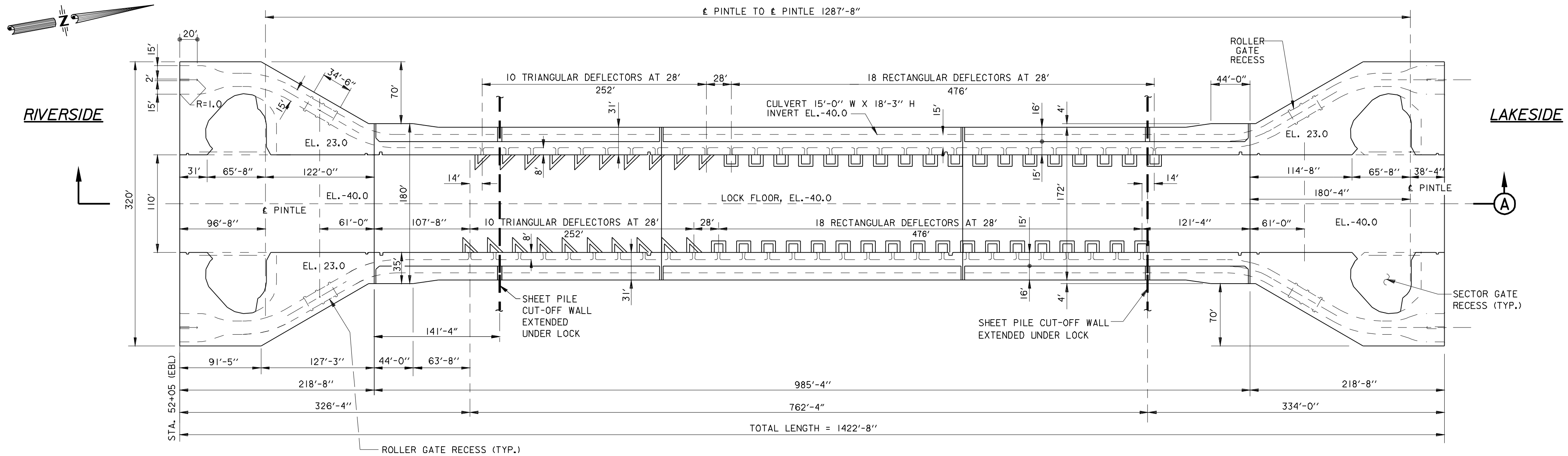
MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

SITE PLAN
PLAN AND SECTIONS

FILE NUMBER H-2-46087
DWG. 1-4

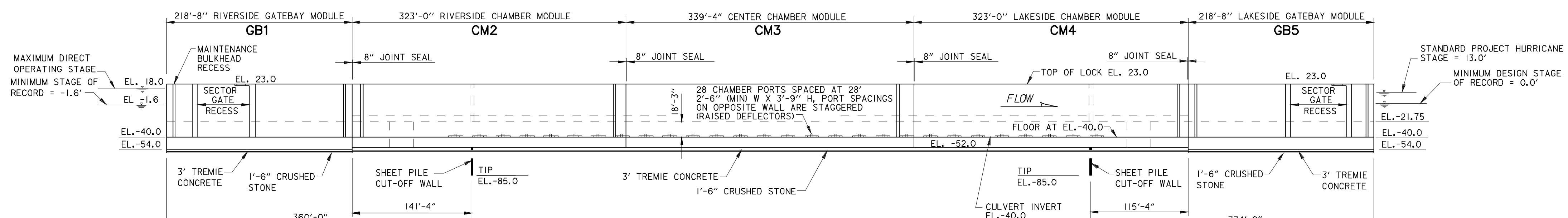
EXHIBIT FIP-1

Safety is a Part of Your Contract



LOCK MODULES PLAN

SCALE: 1" = 50'



LONGITUDINAL CHAMBER SECTION (A)

SCALE: 1" = 50'

NOTES:

1. FOUNDATION PILES NOT SHOWN FOR CLARITY.
2. SHEET PILE TREMIE CONTAINMENT WALL NOT SHOWN FOR CLARITY.

SCALE: 1" = 50'

This drawing has been reduced to half size

FOR INFORMATION ONLY

PHASE I DESIGN
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JULY 2006

URS
3500 N. Causeway Blvd., Suite 900
Metairie, Louisiana 70002
(504) 837-6326
PROJECT No. 10000586.00000

US Army Corps of Engineers
New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

SOLICITATION NO. W912P8-
PLOT SCALE: 50
PLOT DATE: JUNE06

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

LOCK MODULES
PLAN AND SECTION

FILE NUMBER H-2-46087
DWG. I-5

EXHIBIT FIP-2

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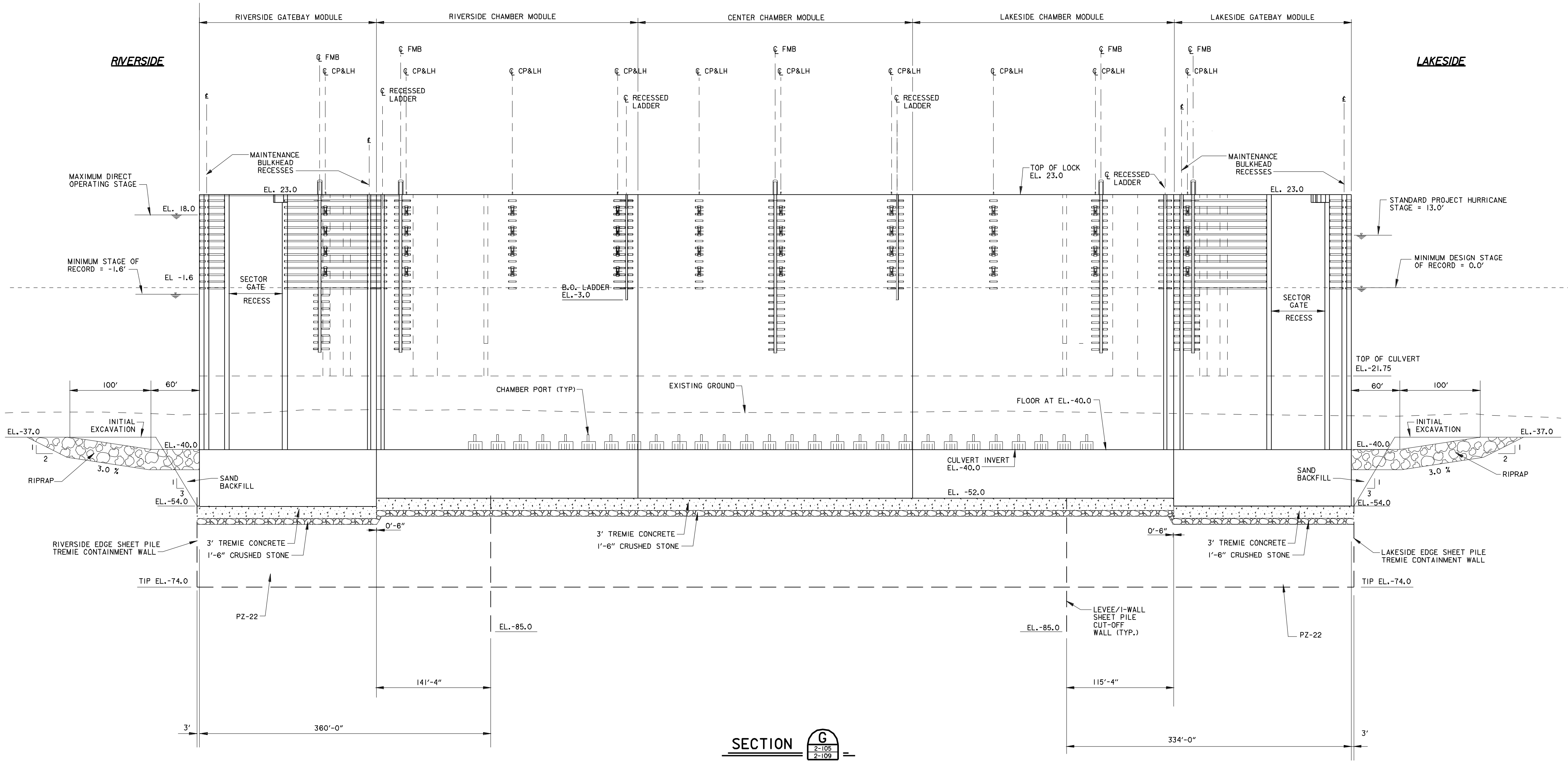

NOTE:
FOUNDATION PILES NOT SHOWN FOR CLARITY.

HORIZONTAL SCALE: 1" = 50'

VERTICAL SCALE: 1" = 10'

This drawing has been reduced to half size

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U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

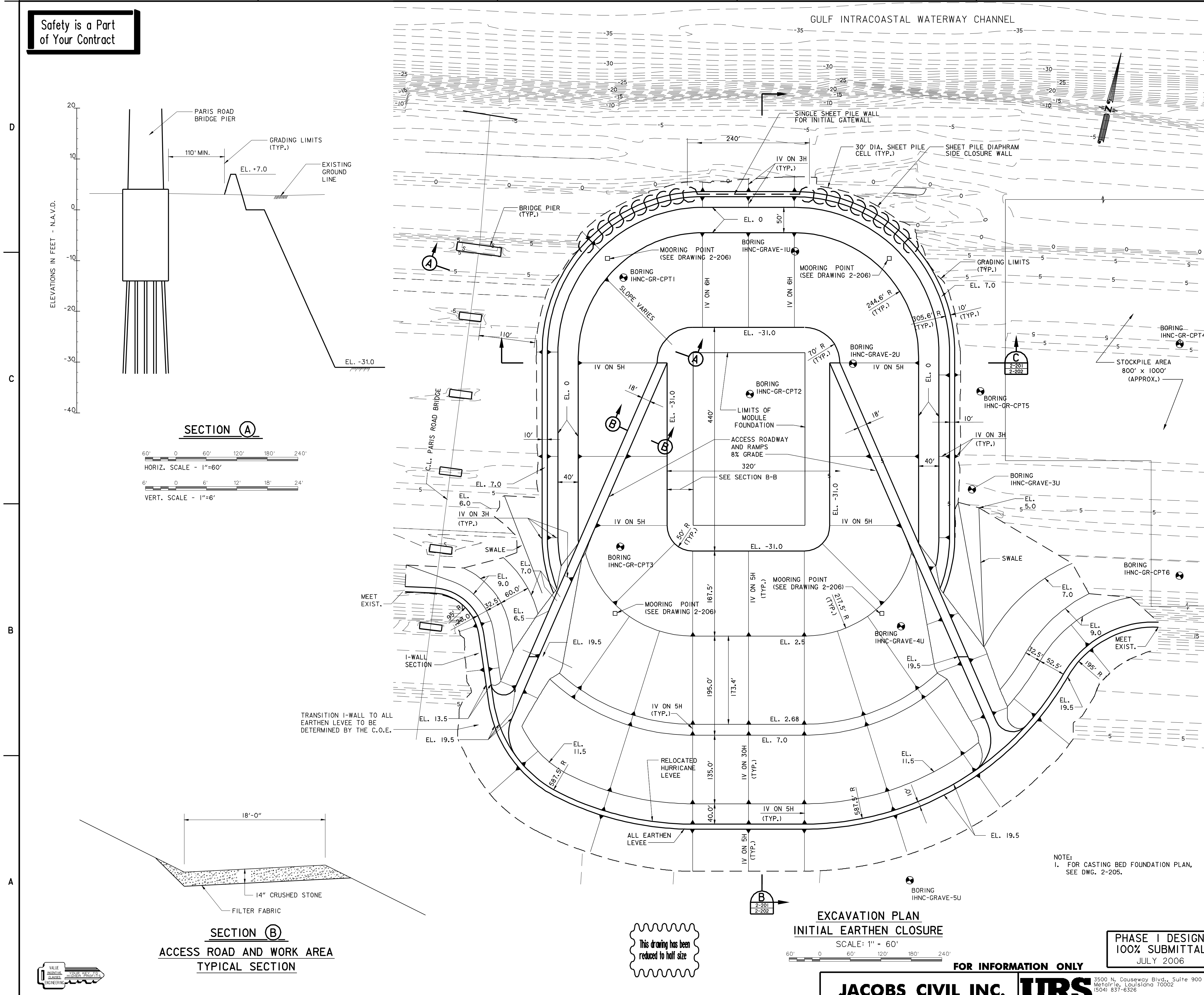
PLOT SCALE: 600
PLOT DATE: JUNE 2006
SOLICITATION NO. W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

LOCK SITE TYPICAL SECTIONS
SHEET 3 OF 3

FILE NUMBER H-2-46087
DWG. 2-109

EXHIBIT FIP-3



NOTE:
1. FOR CASTING BED FOUNDATION PLAN, SEE DWG. 2-205.

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PROJECT No. 10000586.00000

US Army Corps of Engineers
New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: L. WEAVER
CHECKED BY: RASH
DRAWN BY: L. WEAVER
DATE: JUNE 2006
PLOT SCALE: 720
PLOT DATE: JUNE 2006
SOLICITATION NO. W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

GRAVING SITE
GRADING PLAN

FILE NUMBER H-2-46087
DWG. 2-201

EXHIBIT FIP-4

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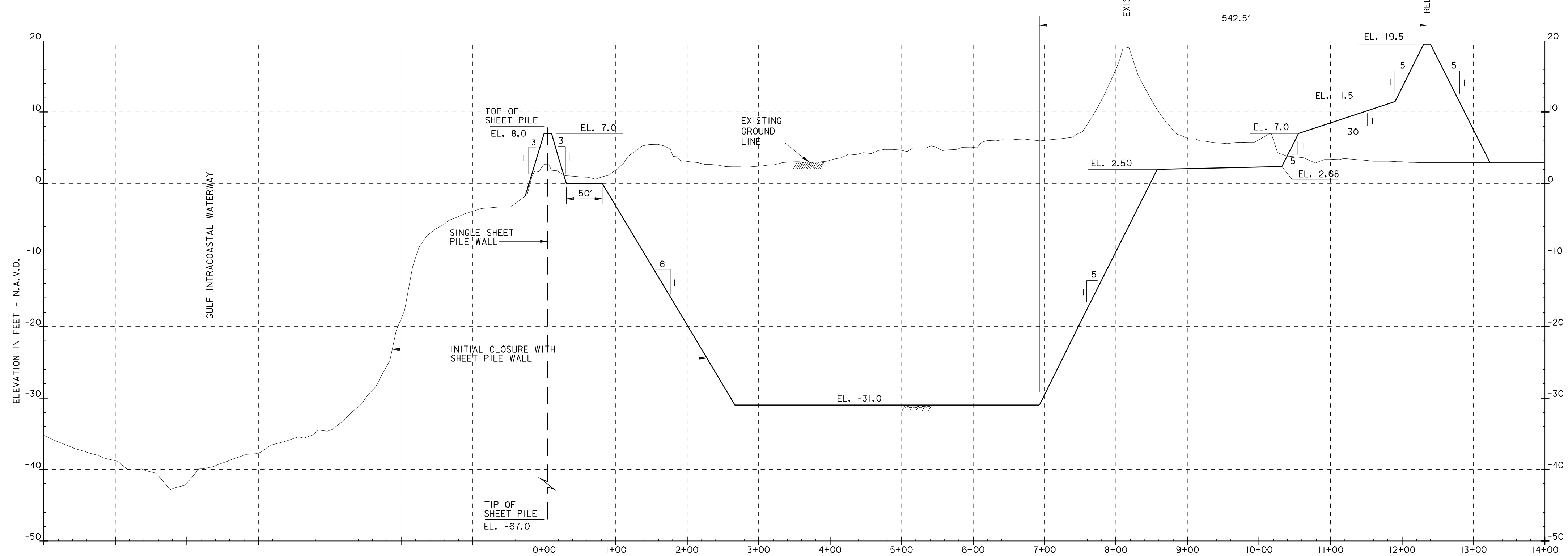
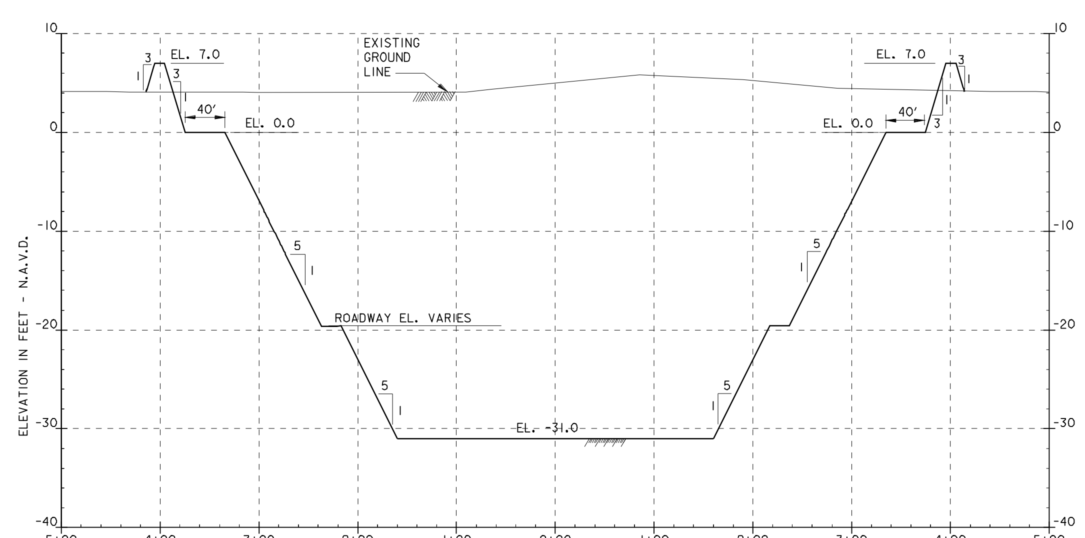


SECTION B (2-201 / 2-202)

HORIZ. SCALE - 1"=60'

VERT. SCALE - 1"=6'



SECTION C (2-201 / 2-202)

HORIZ. SCALE - 1"=60'

VERT. SCALE - 1"=6'

This drawing has been reduced to half size

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JULY 2006

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URS
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(504) 837-6326
PROJECT No. 10000586.00000

US Army Corps of Engineers
New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: L. WEAVER
CHECKED BY: RAJH
DRAWN BY: L. WEAVER
DATE: JUNE 2006
PLOT SCALE: 720
PLOT DATE: JUNE 2006
SOLICITATION NO. W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

GRAVING SITE
CROSS SECTIONS

FILE NUMBER H-2-46087
DWG. 2-202

EXHIBIT FIP-5

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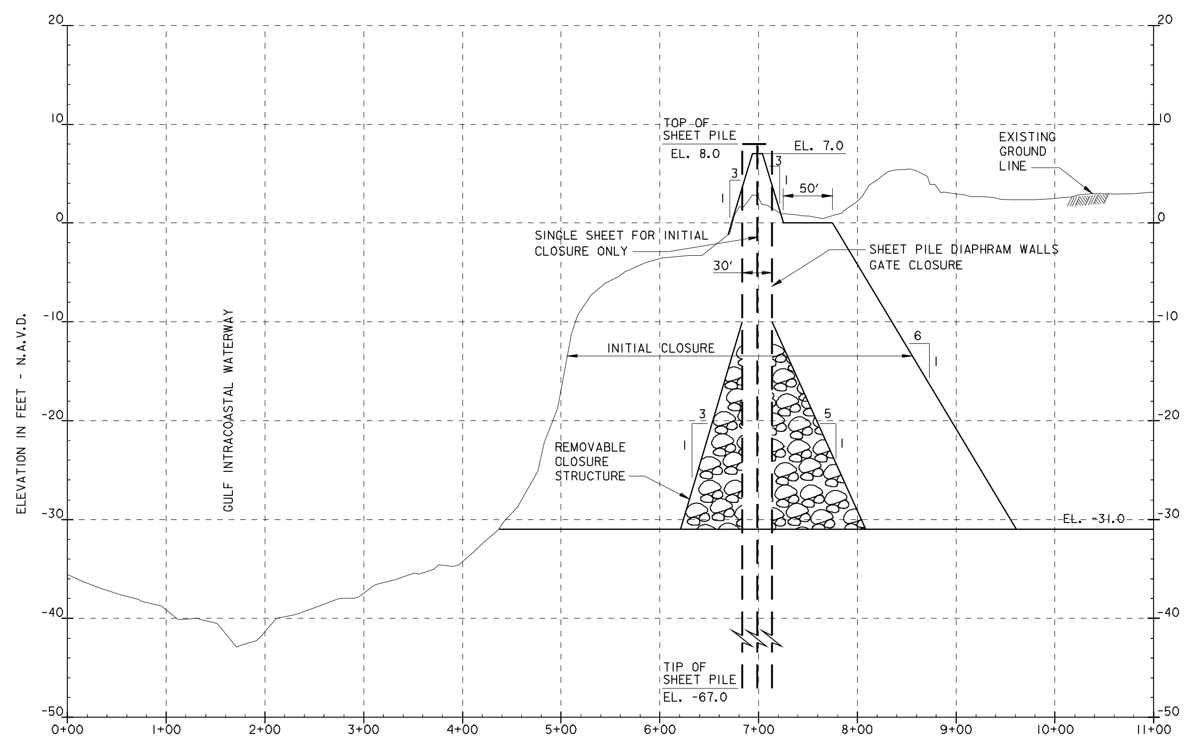

SECTION E 2-203 2-204

CLOSURE STRUCTURE SECTION

HORIZ. SCALE - 1"=60'

VERT. SCALE - 1"=6'

This drawing has been reduced to half size

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JULY 2006

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URS 3500 N. Causeway Blvd., Suite 900 Metairie, Louisiana 70002 (504) 837-6326

PROJECT No. 10000586.00000

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: L. WEAVER
CHECKED BY: RAUH
DRAWN BY: L. WEAVER
DATE: JUNE 2006

PLOT SCALE: 720

PLOT DATE: JUNE 2006

DESIGN FILE NAME: IHNC-2-204.DGN

SUBMITTED BY: DESIGN ENGINEER

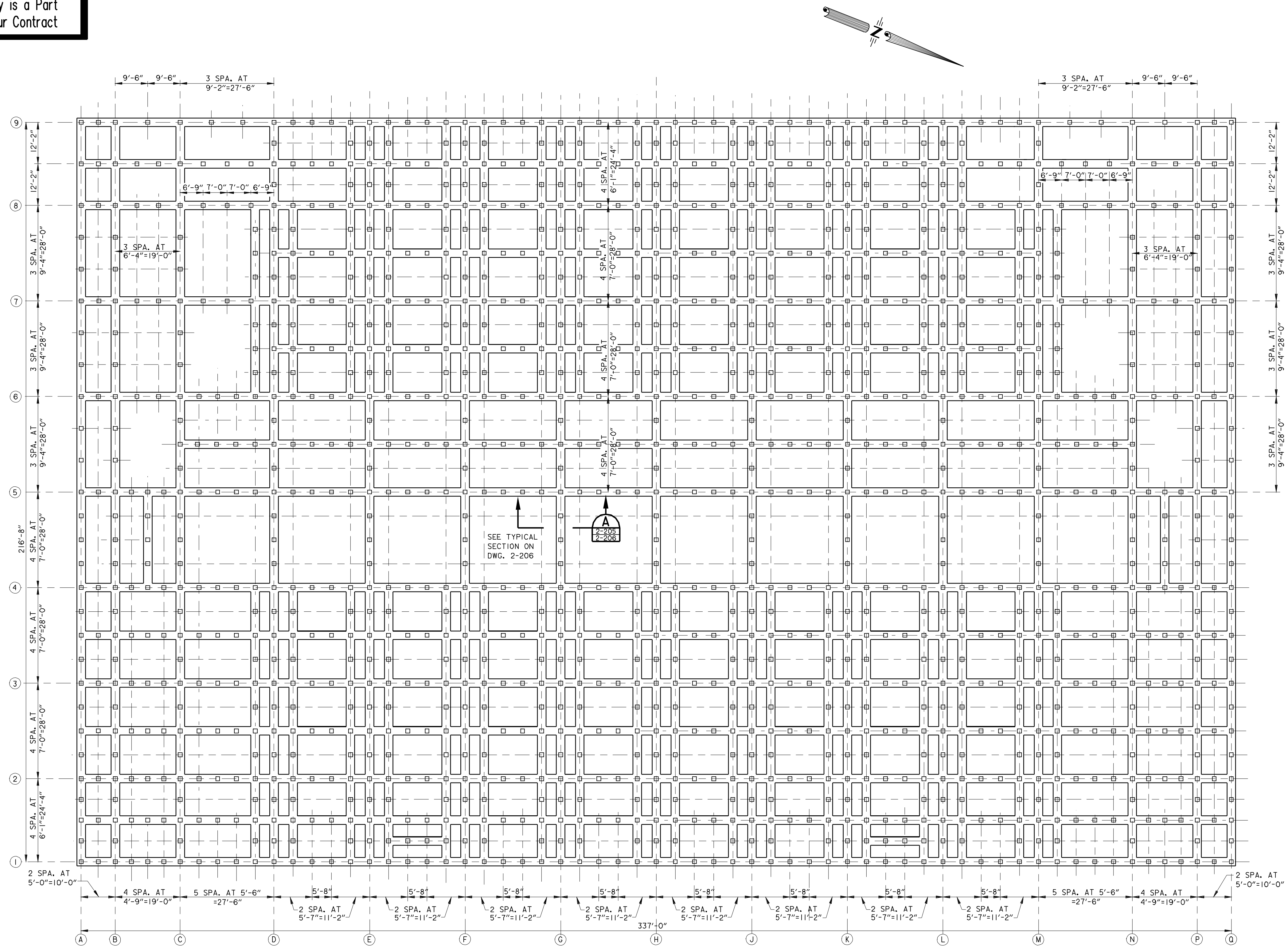
SOLICITATION NO. W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA
GRAVING SITE
CLOSURE STRUCTURE SECTION

FILE NUMBER H-2-46087
DWG. 2-204

EXHIBIT FIP-6

Safety is a Part of Your Contract



CASTING BED FOUNDATION PLAN

SCALE: 3/32 "=1'-0"

NOTES:
TOP OF CONCRETE EL. -31.0
□ = 14" SQ. PRECAST CONCRETE PILE.
FOR LOCATION OF CASTING BED FOUNDATIONS, SEE DWG. 2-201

This drawing has been reduced to half size

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JULY 2006

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PROJECT No. 10000586.00000

US Army Corps of Engineers
New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

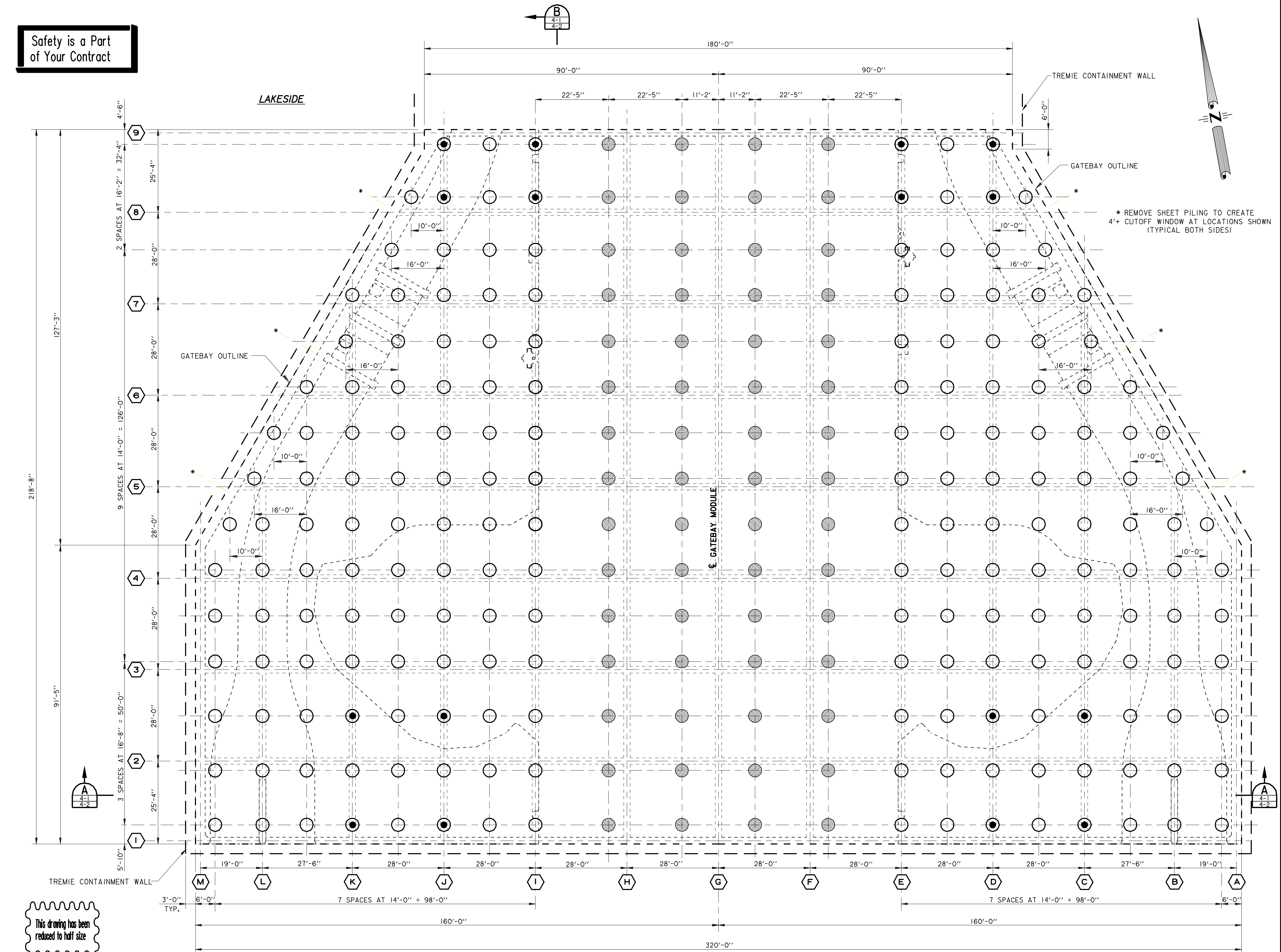
DESIGNED BY: C. MATOS CHECKED BY: B. ERSCHEN DRAWN BY: S. MORR DATE: JUNE 2006	PLOT SCALE: 128	PLOT DATE: 6/2006	DESIGN FILE NAME: IHNC-2-205.DGN SUBMITTED BY: DESIGN ENGINEER	SOLICITATION NO. W912P8

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

GRAVING SITE
CASTING BED FOUNDATION PLAN

FILE NUMBER
H-2-46087
DWG. 2-205

EXHIBIT FIP-7



PILE LEGEND

- COMPRESSION PILES (120' LONG)
- LANDING PILES (120' LONG)
- REQUIRED TENSION PILES (120' LONG) (DEWATERED CONDITION)

NOTE:
ALL PILES 48"Ø, 5/8" WALL
ASTM A252, GR 2

RIVERSIDE GATEBAY MODULE (GBI)
FOUNDATION PILE PLAN

SCALE: 1" = 10'

FOR INFORMATION ONLY

PHASE I DESIGN
100% SUBMITTAL
JULY 2006

Brown·Cunningham·Gannuch
ENGINEERS • ARCHITECTS • CONSULTANTS
2701 KINGMAN ST. METAIRIE, LOUISIANA

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3500 N. Causeway Blvd., Suite 900
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PROJECT No. 10000586.00000

US Army Corps of Engineers
New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: R.W.Y.
CHECKED BY: J.D.W.
DRAWN BY: B.H.D.
DATE: SEPT. 2005
PLOT SCALE: 120
PLOT DATE: JUNE 2006
DESIGN FILE NAME: RGB4-1.DGN
SUBMITTED BY: X DESIGN ENGINEER
SOLICITATION NO. W912P8-

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

RIVERSIDE GATEBAY MODULE
FOUNDATION PILE PLAN

FILE NUMBER H-2-46087
DWG. 4-1

EXHIBIT FIP-8

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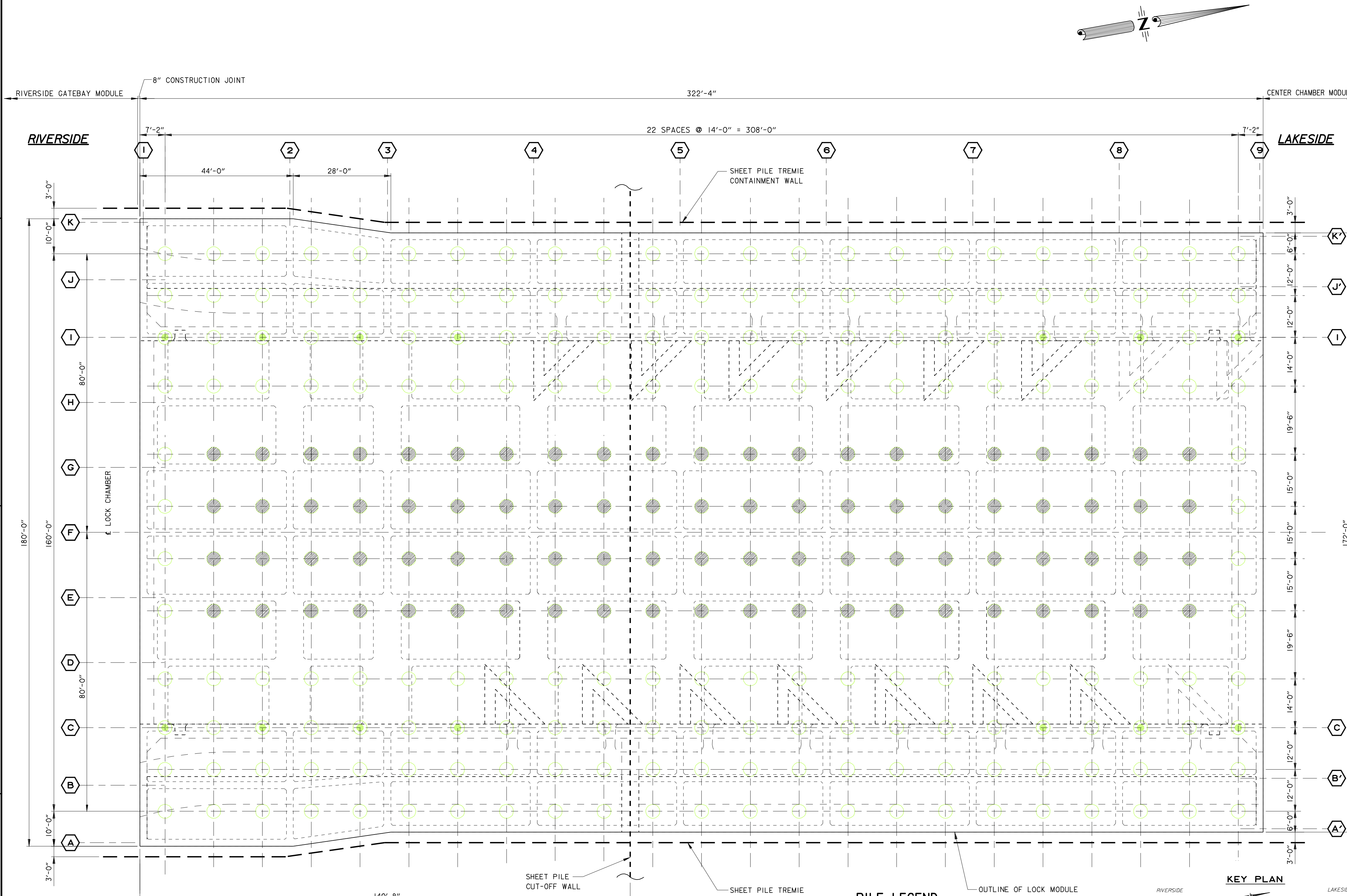

FOUNDATION PILE PLAN
RIVERSIDE CHAMBER MODULE (CM2)

SCALE: 1"=10'-0"

NOTE:

ALL PILES 48"ø, 5/8" WALL
ASTM A252, GR 2

SCALE: 1" = 10'

PILE LEGEND

- COMPRESSION PILE (120' LONG)
- REQUIRED TENSION PILES (120' LONG) (DEWATERED CONDITION)
- LANDING PILES (120' LONG)
- STEEL SHEET PILE CUT-OFF WALL
- STEEL SHEET PILE TREMIE CONTAINMENT WALL

KEY PLAN

RIVERSIDE LAKESIDE

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09/02/05

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PROJECT No. 10000586.00000

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: B.D.F.
CHECKED BY: M.A.C.
DRAWN BY: T.R.E.
DATE: 09-02-05
PLOT SCALE: 10
PLOT DATE: 9/2/05
DESIGN FILE NAME: IHNC-4-03.DGN
SUBMITTED BY: DESIGN ENGINEER
SOLICITATION NO. W912P8-

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

RIVERSIDE CHAMBER MODULE FOUNDATION PILE PLAN

FILE NUMBER H-2-46087
DWG. 4-3

EXHIBIT FIP-9

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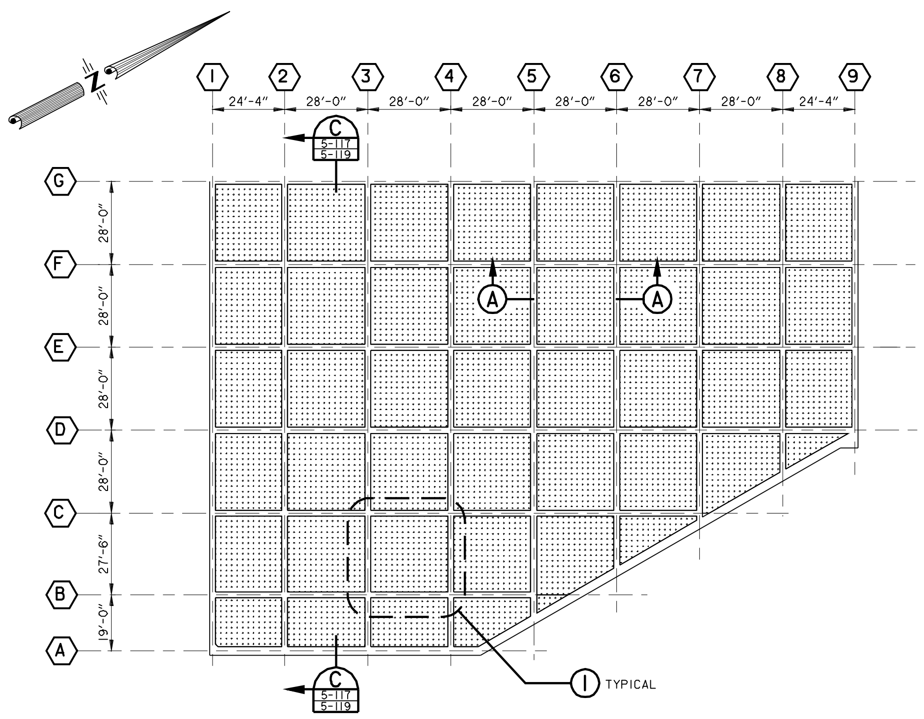
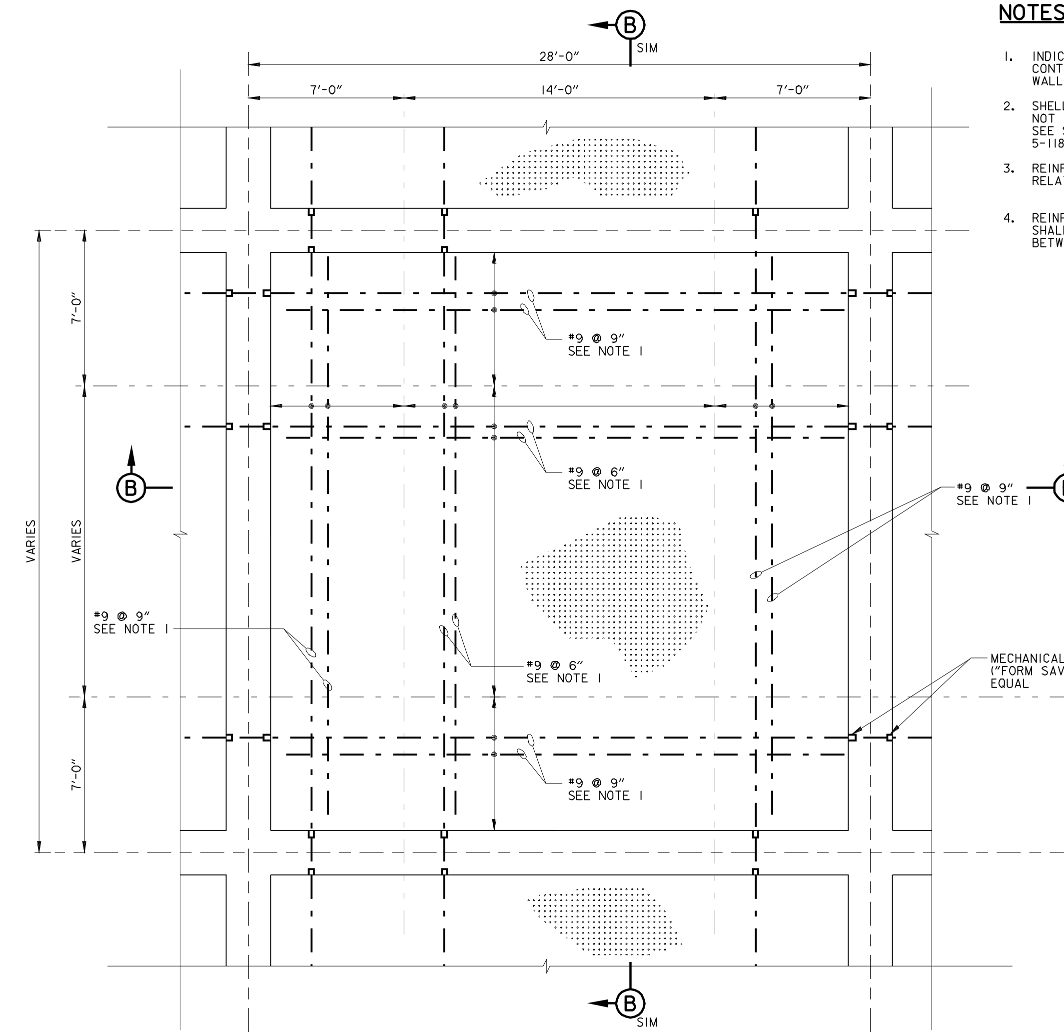


PLAN
EL. -52.0 TO EL. -42.0
SCALE: 1" - 20'

LEGEND:

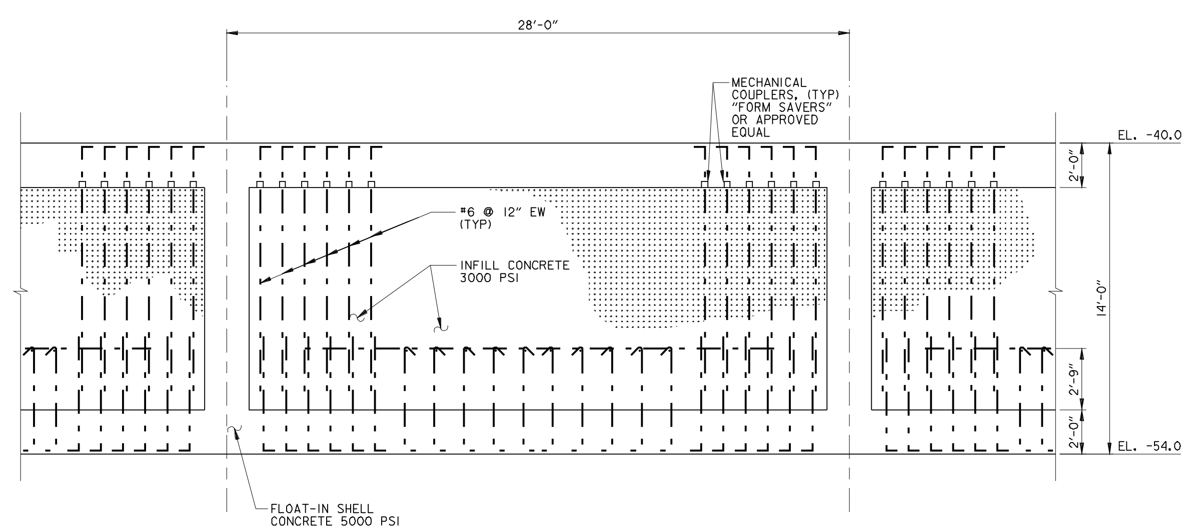
- INFILL CONCRETE (3000 PSI)
- FLOAT-IN SHELL CONCRETE (5000 PSI)



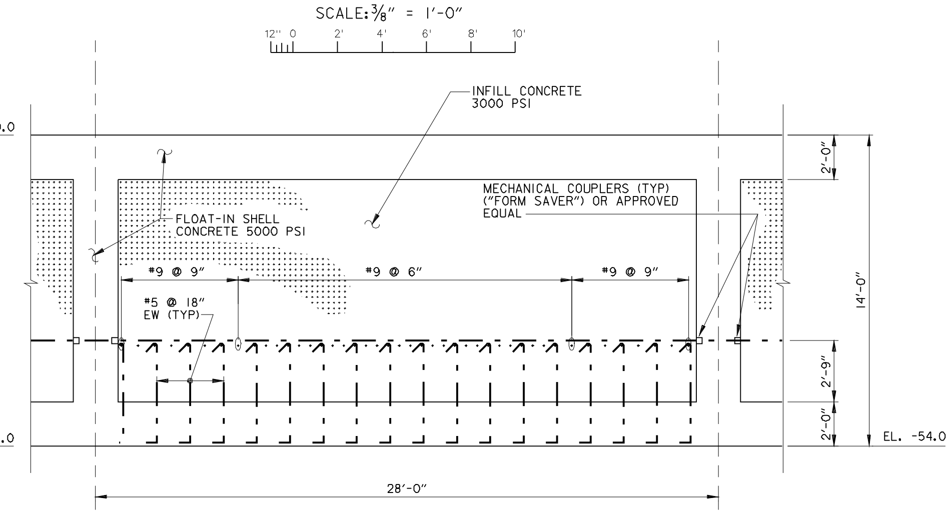
INFILL REINFORCEMENT DETAIL (1)
SCALE: 3/8" = 1'-0"

NOTES:

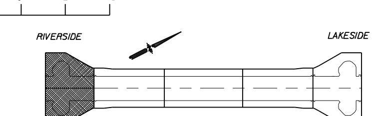
1. INDICATED BARS SHALL CONTINUE THROUGH THE WALLS.
2. SHELL REINFORCEMENT NOT SHOWN FOR CLARITY, SEE SECTION A AND B ON 5-118, 5-119, AND 5-121.
3. REINFORCEMENT AT DETAIL (1) RELATED TO ALL CELLS.
4. REINFORCEMENT AT SECTION (A) SHALL BE ADDED TO CELLS BETWEEN GRIDS C-G.



SECTION (A)
SCALE: 3/8" = 1'-0"



SECTION (B)
SCALE: 3/8" = 1'-0"



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JULY 2006

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PROJECT No. 10000586.00000

US Army Corps of Engineers
New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: DM
CHECKED BY: VR
DRAWN BY:
DATE: 09-02-05
PLOT SCALE: 32
PLOT DATE: JUNE06
DESIGN FILE NAME:
SUBMITTED BY:
SOLICITATION NO. W912P8-

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

INFILL REINFORCMENT

FILE NUMBER H-2-46087
DWG. 5-117

EXHIBIT FIP-10

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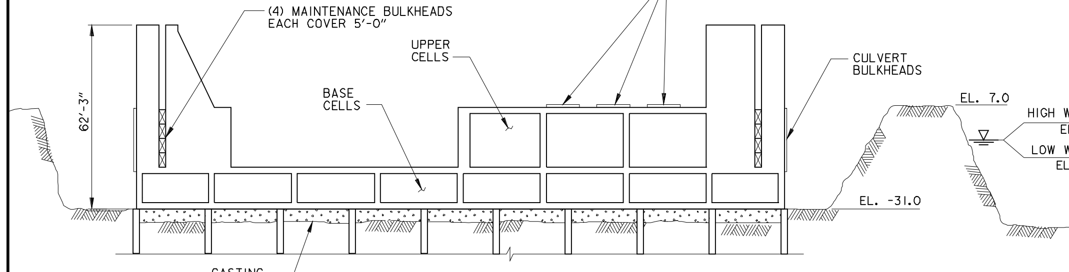
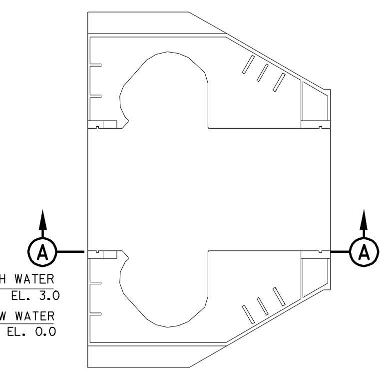


GRAVING SITE (A)

SCALE: 1" = 20'

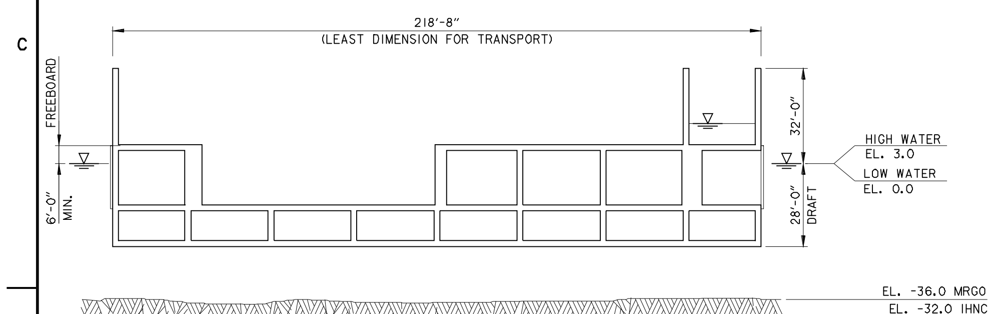
STEP

1. INSTALL 4 MAINTENANCE BULKHEADS EACH END
2. INSTALL TEMPORARY CULVERT BULKHEADS
3. INSTALL TEMPORARY ROLLER GATE COVERS



PLAN

NTS

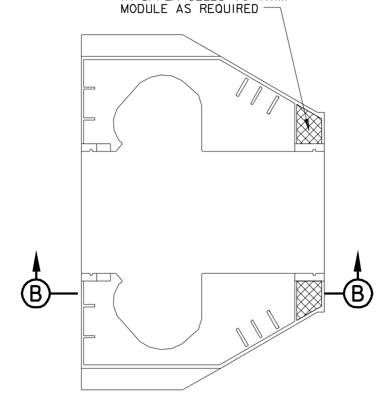


TRANSPORT (B)

SCALE: 1" = 20'

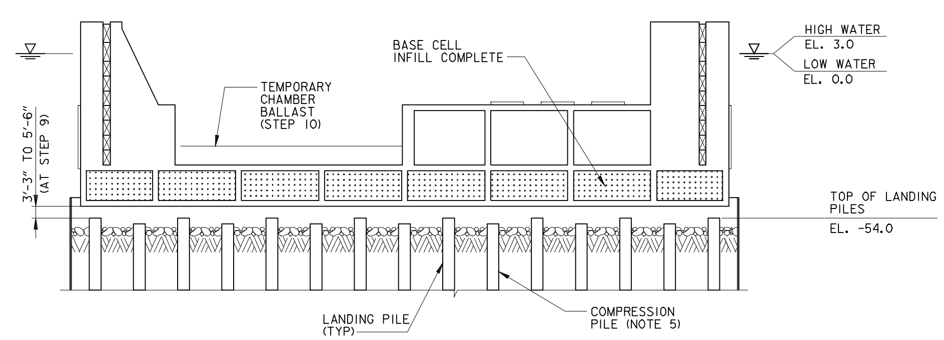
STEP

4. FLOOD GRAVING SITE
5. ADJUST TO LEVEL WITH TEMPORARY BALLAST
6. TRANSPORT TO LOCK SITE



PLAN

NTS

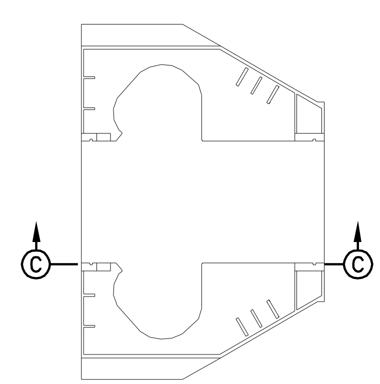


SUBMERGENCE (C)

SCALE: 1" = 20'

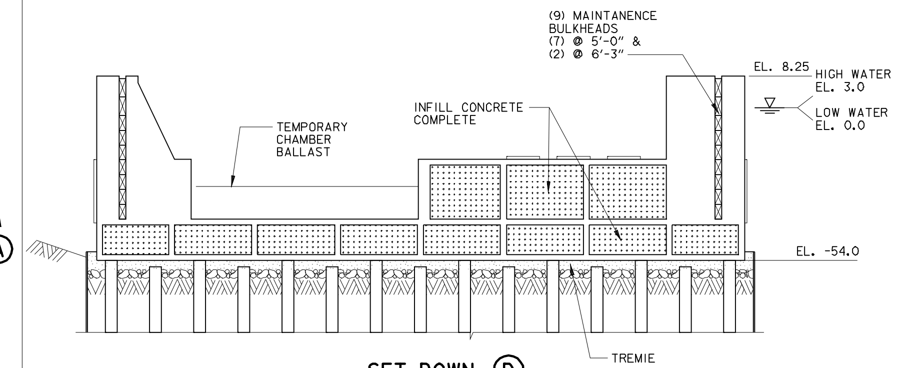
STEP

7. INSTALL CONCRETE DELIVERY EQUIPMENT
8. INFILL BASE CELLS IN 5 FOOT LIFTS. REFER TO DWG 16-6 FOR SEQUENCE.
9. FLOOD CULVERTS IN ACCORDANCE WITH CELL STATUS TABLE.
10. PARTIALLY PLACE TEMPORARY SAND BALLAST IN CHAMBER TO ACHIEVE SET-DOWN.
11. DEWATER CULVERTS IF NECESSARY TO REALIGN



PLAN

NTS

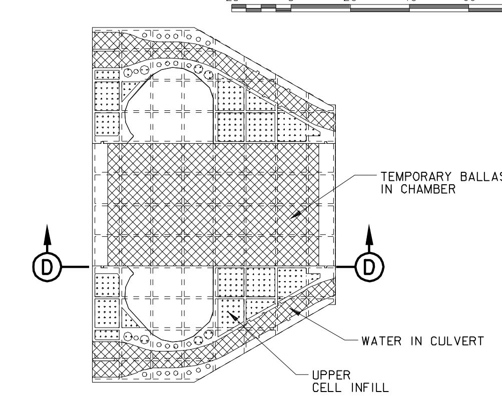


SET-DOWN (D)

SCALE: 1" = 20'

STEP

12. FLOOD CULVERTS FULLY.
13. ADD CHAMBER BALLAST TO ACHIEVE 5% NEGATIVE BUOYANCY
14. CONNECT TENSION PILES, FOR LOCATIONS OF TENSION PILE, SEE DWG 16-3
15. INSTALL UNDERBASE TREMIE CONCRETE
16. COMPLETE MODULE CONSTRUCTION



PLAN

NTS

LEGEND:

FLOAT-IN SHELL CONCRETE (5000 PSI)

INFILL CONCRETE (3000 PSI)

CELL STATUS TABLE

	CONDITION	BASE CELLS	CULVERT WATER	UPPER CELLS	CHAMBER SAND	DRAFT	COMMENTS
		%	%	%	FT	FT	
1	FLOATOUT	0%	0%	0%	0	27.9	TRANSPORT OUTFITTING
2	LOCK SITE	0%	0%	0%	0	28.0	SET-DOWN OUTFITTING
3	FILL BASE CELLS	100%	0%	0%	0	47.8	POOL ELEV VARIES
4	CULVERT	100%	50%	0%	0	49.1	POOL ELEV VARIES
5A	SET-DOWN ON LANDING PILES LOW WATER	100%	50%	0%	7.2	54.0	POOL AT EL. 0.0
5B	SET-DOWN ON LANDING PILES HIGH WATER	100%	50%	0%	11.7	57.0	POOL AT EL. 3.0
6	APPLY 5% NEGATIVE BUOYANCY	100%	100%	0%	11.8	55.5	5% DETERMINED AT POOL AT EL. 1.5

NOTES:

1. CLEARANCE AND DRAFT RESTRICTIONS:

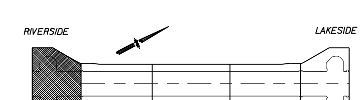
   TWO BRIDGES WILL REMAIN BETWEEN THE GRAVING SITE AND THE LOCK SITE AT THE TIME OF CONSTRUCTION. BRIDGE VERTICAL AND HORIZONTAL RESTRICTIONS ARE SUMMARIZED BELOW

   PARIS ROAD BRIDGE (FIXED SPAN)
   VERTICAL CLEARANCE 135 FT ABOVE EL. 5.0
   HORIZONTAL CLEARANCE 500 FT AT EL. -36.0

   NEW FLORIDA AVE. BRIDGE (VERTICAL LIFT)
   VERTICAL CLEARANCE 156 FT ABOVE EL. 5.0
   HORIZONTAL CLEARANCE 216 FT AT EL. -25.0
   150 FT AT EL. -36.0

2. AUTHORIZED CHANNEL DEPTHS:
   MRGO/IHNC - 36.0 NAVD
   FLORIDA AVE. BRIDGE - 25.0 NAVD
   THE MINIMUM BOTTOM CLEARANCE IS 2 FT

3. ELEVATIONS AND ORIENTATION REFER TO LOCK INSTALLED CONFIGURATION
4. INCREASE TRIM BALLAST TO MAINTAIN LEVEL MODULE AS DRAFT INCREASES
5. TOP OF COMPRESSION PILES AT EL. -56.0



KEY PLAN

This drawing has been reduced to half size

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JULY 2006

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PROJECT No. 10000586.00000

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: JC
CHECKED BY: DA
DRAWN BY: VR
DATE: 09-02-05

PLOT SCALE: 240
PLOT DATE: JUNE06
DESIGN FILE NAME: $FILEABBREV$
SUBMITTED BY: JC DESIGN ENGINEER
SOLICITATION NO. W912P8-

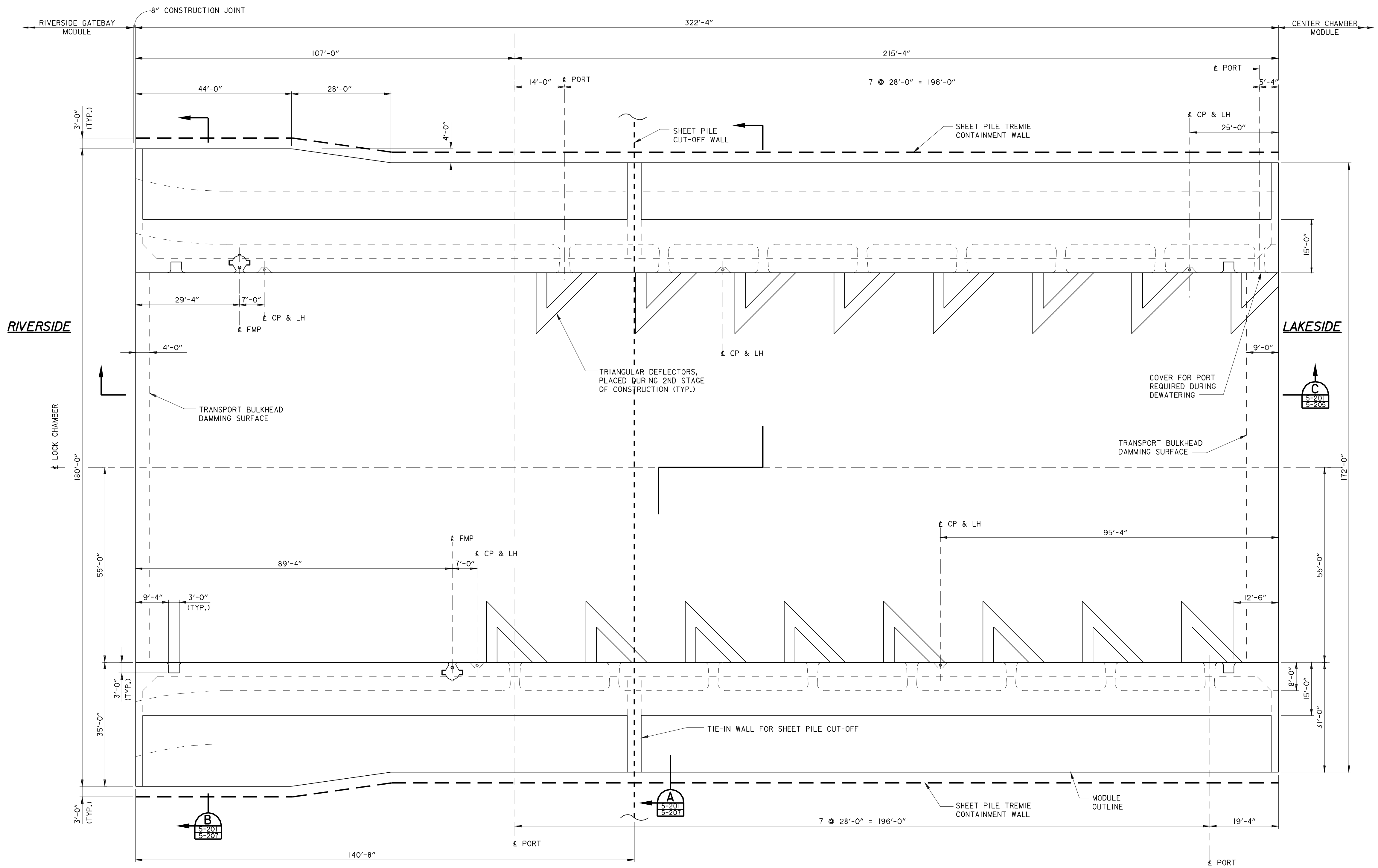
MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

SET-DOWN PROCEDURES

FILE NUMBER H-2-46087
DWG. 5-143

EXHIBIT FIP-17

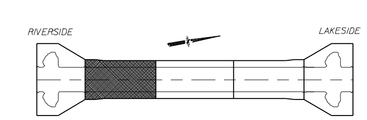
Safety is a Part of Your Contract



GENERAL PLAN

RIVERSIDE CHAMBER MODULE (CM2)

SCALE: 1"=10'-0"



KEY PLAN

SCALE: 1" = 10'

10' 0 10' 20' 30' 40'

This drawing has been reduced to half size

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JULY 2006

URS
3500 N. Causeway Blvd., Suite 900
Metairie, Louisiana 70002
(504) 837-6326
PROJECT No. 10000586.00000

US Army Corps of Engineers New Orleans District

MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: B.D.F.
CHECKED BY: M.A.C.
DRAWN BY: C.A.D.
DATE: 09-02-05
PLOT SCALE: 10
PLOT DATE: JUNE06
SOLICITATION NO.: W912P8-

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

RIVERSIDE CHAMBER MODULE
GENERAL PLAN

FILE NUMBER H-2-46087
DWG. 5-201

EXHIBIT FIP-11

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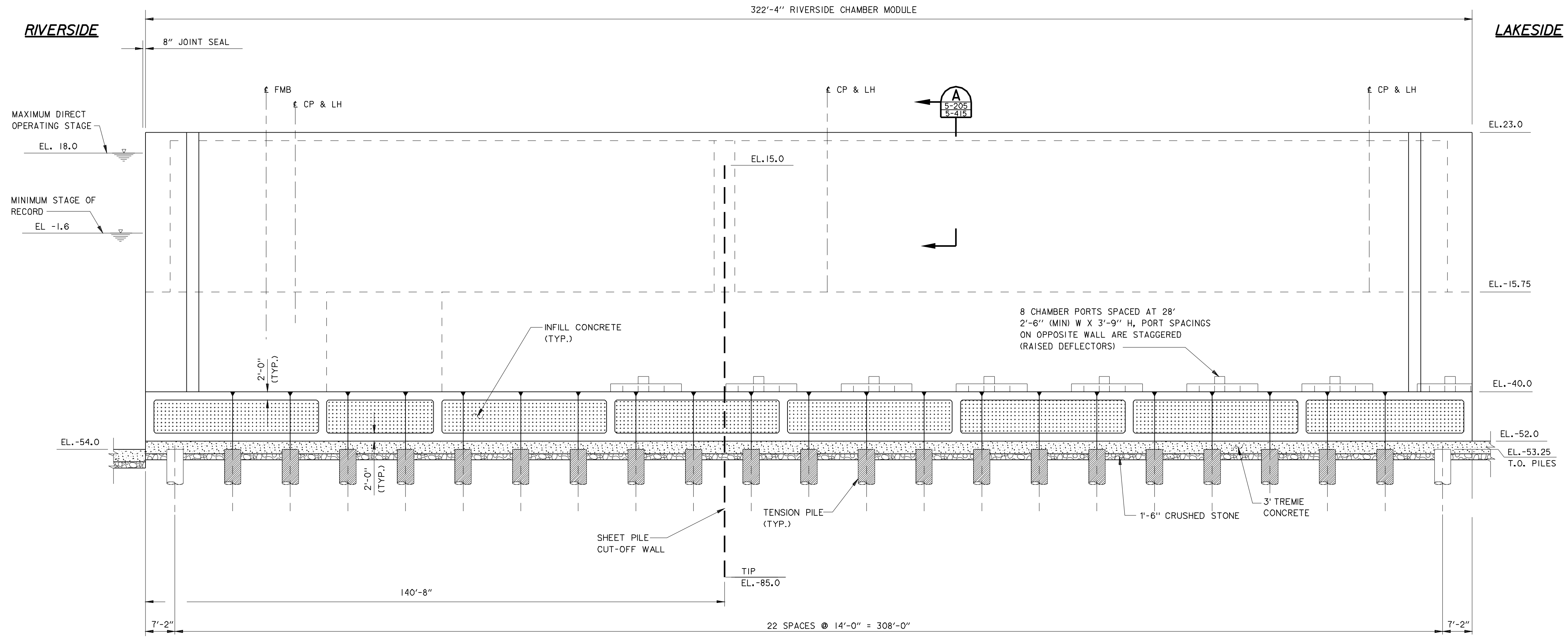

LONGITUDINAL CHAMBER

SECTION C 5-201 5-205

SCALE: 1"=10'-0"

LEGEND:

FLOAT-IN SHELL CONCRETE (5000 psi)

INFILL CONCRETE (3000 psi)

TREMIE CONCRETE

CRUSHED STONE

48"ø STEEL PIPE PILE

48"ø STEEL PIPE PILE W/TENSION CONNECTOR

SCALE: 1" = 10'

This drawing has been reduced to half size

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MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

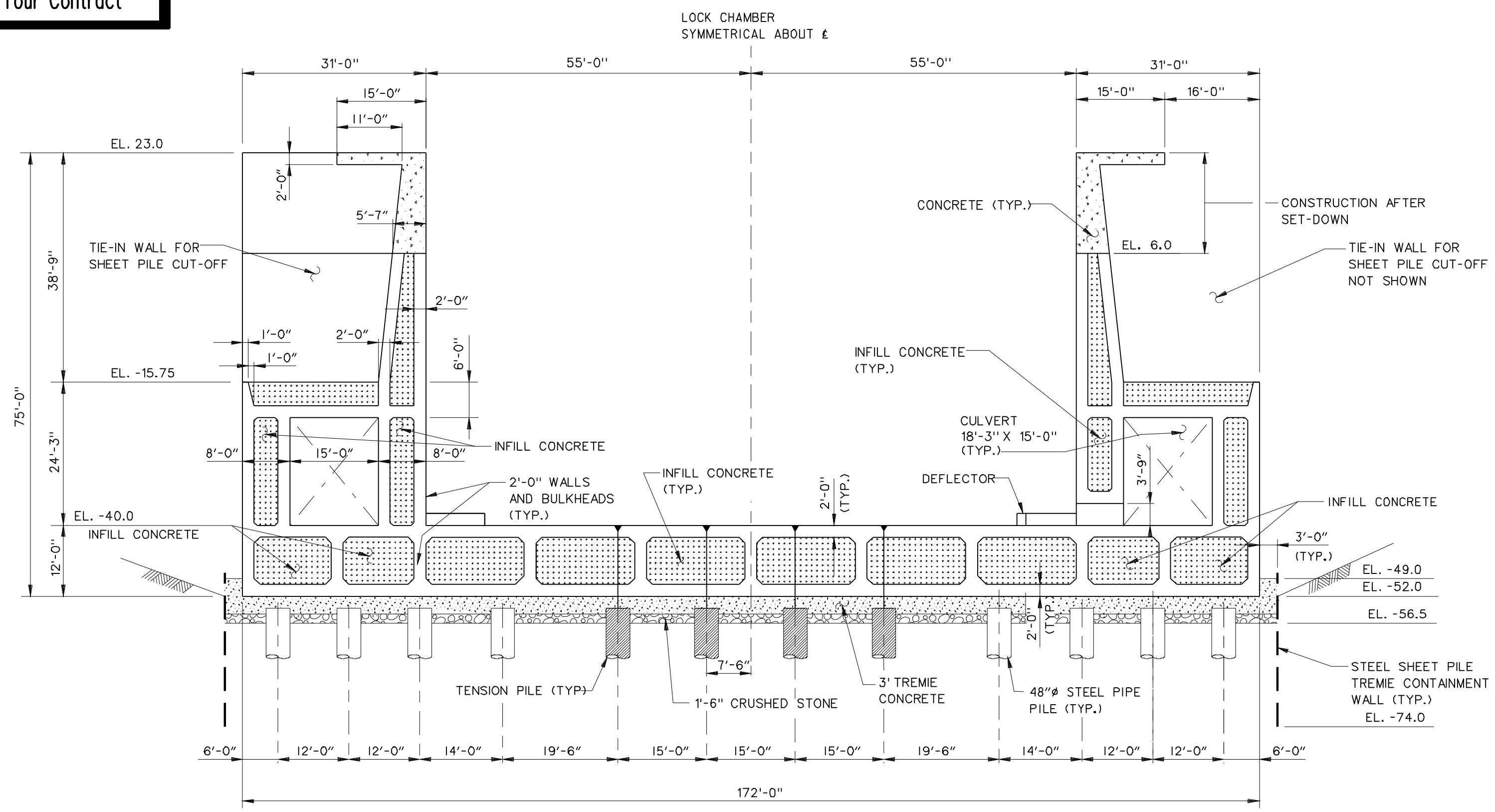
RIVERSIDE CHAMBER MODULE
ELEVATION-LONGITUDINAL THROUGH LOCK

FILE NUMBER H-2-46087

DWG. 5-205

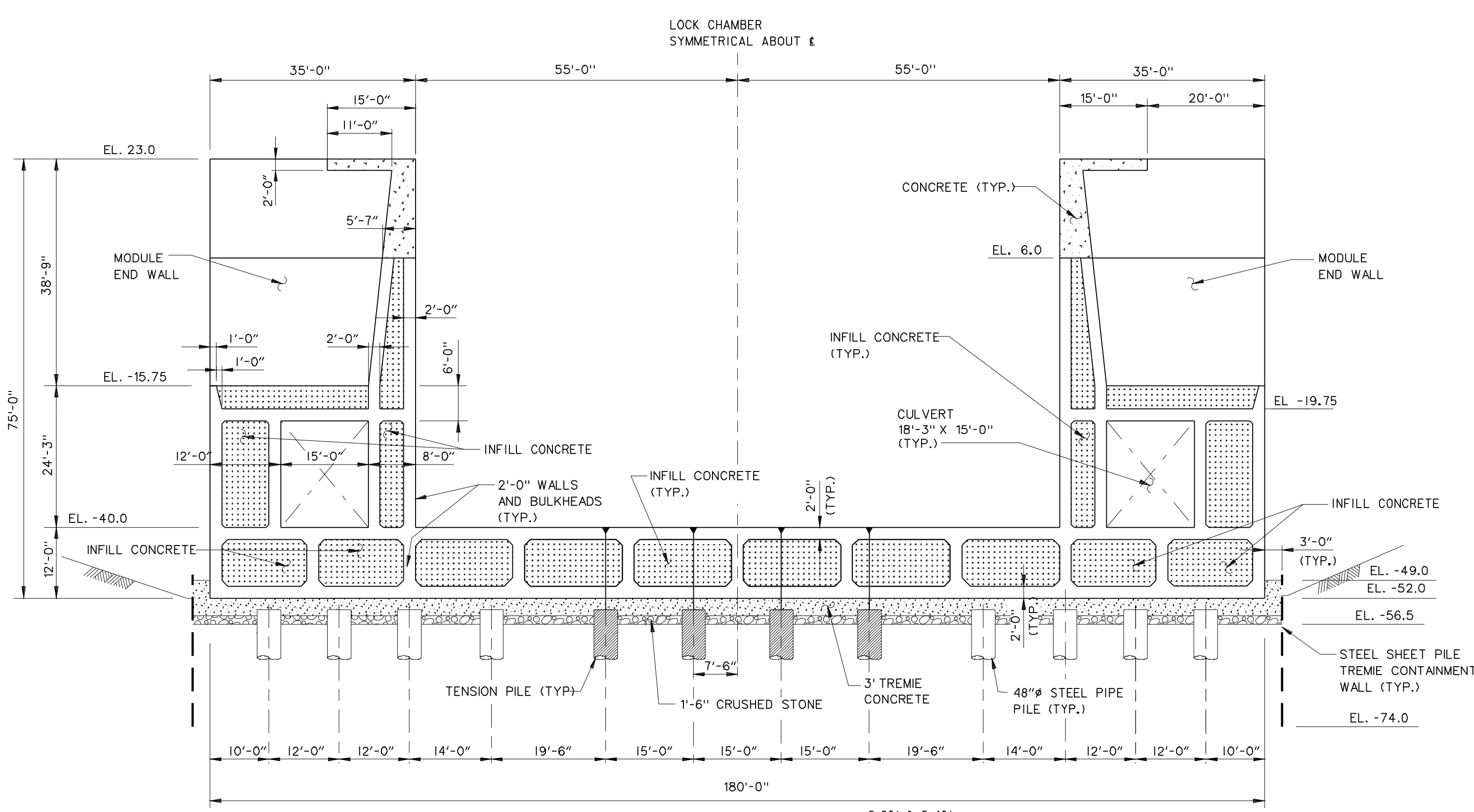
EXHIBIT FIP-12

Safety is a Part of Your Contract



SECTION A 5-207 (5-201 & 5-401)

SCALE: 1" = 10'



SECTION B 5-207 (5-201 & 5-401)

SCALE: 1" = 10'

LEGEND:

- FLOAT-IN SHELL CONCRETE (5000 PSI)
- INFILL CONCRETE (3000 PSI)
- CONCRETE (4000 PSI)
- TREMIE CONCRETE (5000 PSI)
- CRUSHED STONE
- 48"⌀ STEEL PIPE PILE
- 48"⌀ STEEL PIPE PILE W/TENSION CONNECTOR

This drawing has been reduced to half size

SCALE: 1" = 10'

10' 0 10' 20' 30' 40'

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US Army Corps of Engineers
New Orleans District

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: B.D.F. CHECKED BY: M.A.C. DRAWN BY: T.R.E. DATE: 09-02-05	PLOT SCALE: 120	PLOT DATE: JUNE06	DESIGN FILE NAME: IHNC-5-207.DGN
			SUBMITTED BY: X DESIGN ENGINEER / SOLICITATION NO. W912P8-

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

RIVERSIDE CHAMBER MODULE SECTIONS

FILE NUMBER H-2-46087

DWG. 5-207

EXHIBIT FIP-13

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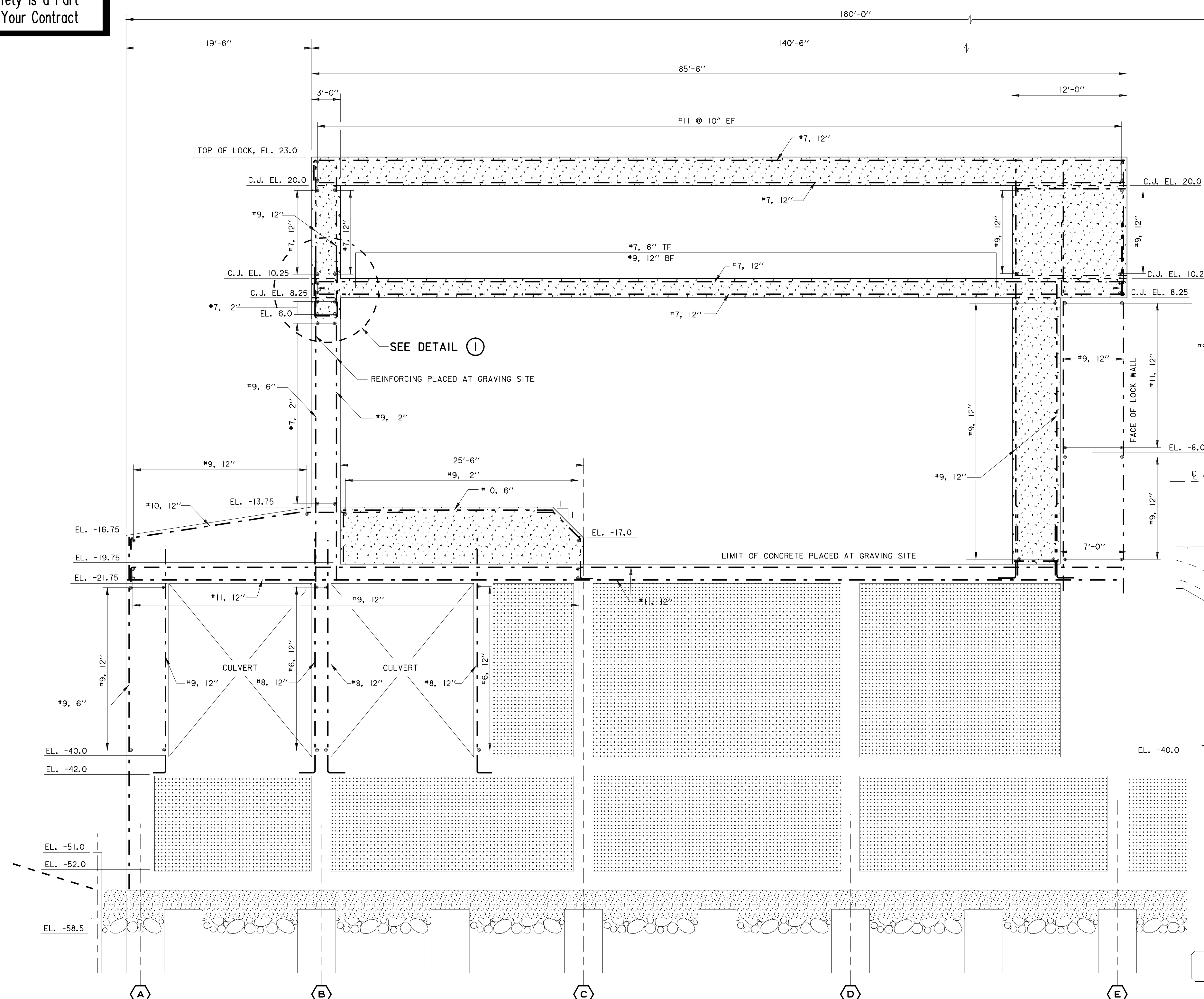
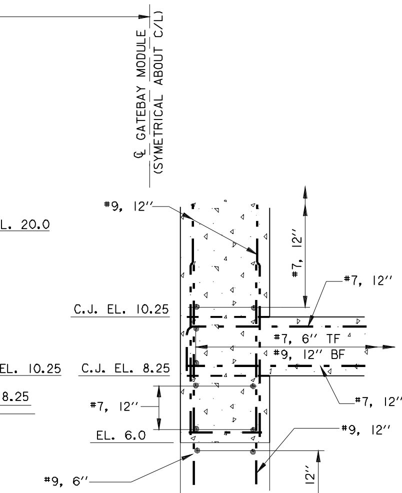


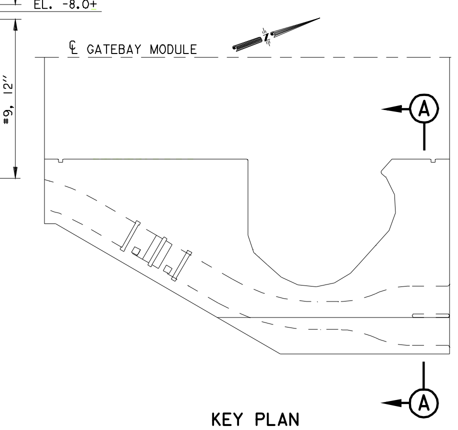
SECTION A

SCALE: 1/4" = 1'- 0"



DETAIL 1

SCALE: 1/2" = 1'- 0"



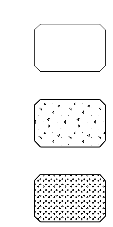
KEY PLAN

LAKESIDE GATEBAY MODULE

NOTES:

1. SEE DWG. 5-517 FOR COMPOSITE INFILL REINFORCEMENT.
2. SEE DWG. 5-519 FOR ADDITIONAL FLOAT-IN SHELL CONCRETE REINFORCEMENT.

LEGEND

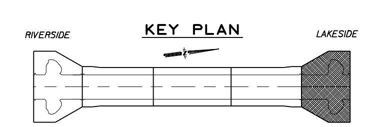


FLOAT-IN SHELL CONCRETE (5000 PSI)

CONCRETE (4000 PSI)

INFILL CONCRETE (3000 PSI)

KEY PLAN



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PROJECT No. 10000586.00000

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: B.D.F.
CHECKED BY: M.A.C.
DATE: 09-02-05

PLOT SCALE: 48
PLOT DATE: JUNE06
SOLICITATION NO. W912P8-

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

LAKESIDE GATEBAY MODULE
REINFORCING SECTION

FILE NUMBER H-2-46087
DWG. 5-530

EXHIBIT FIP-14

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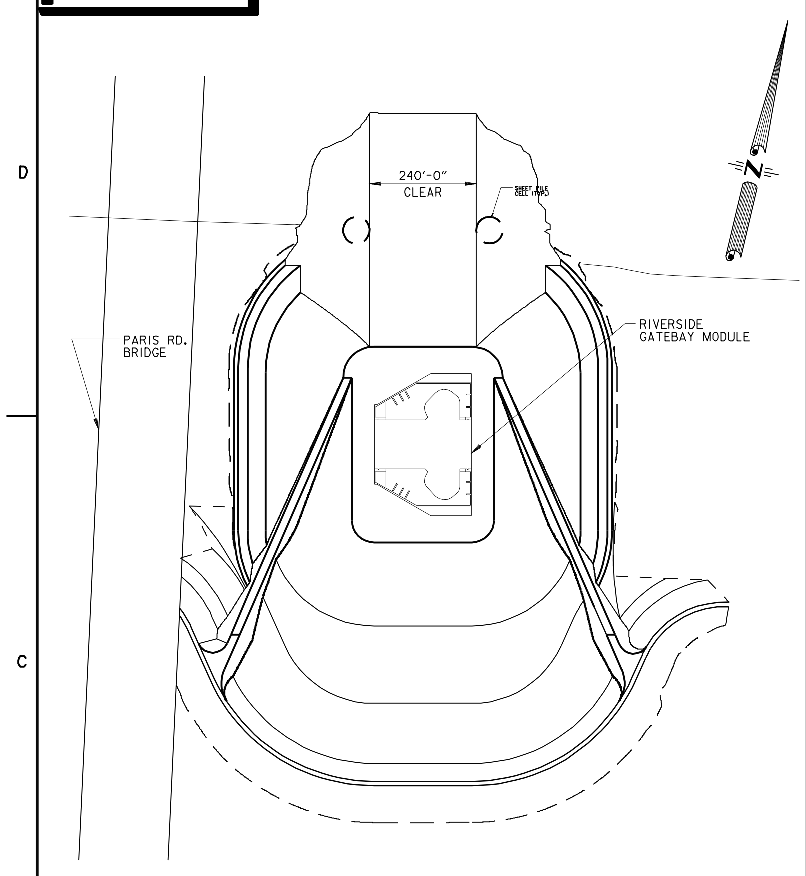
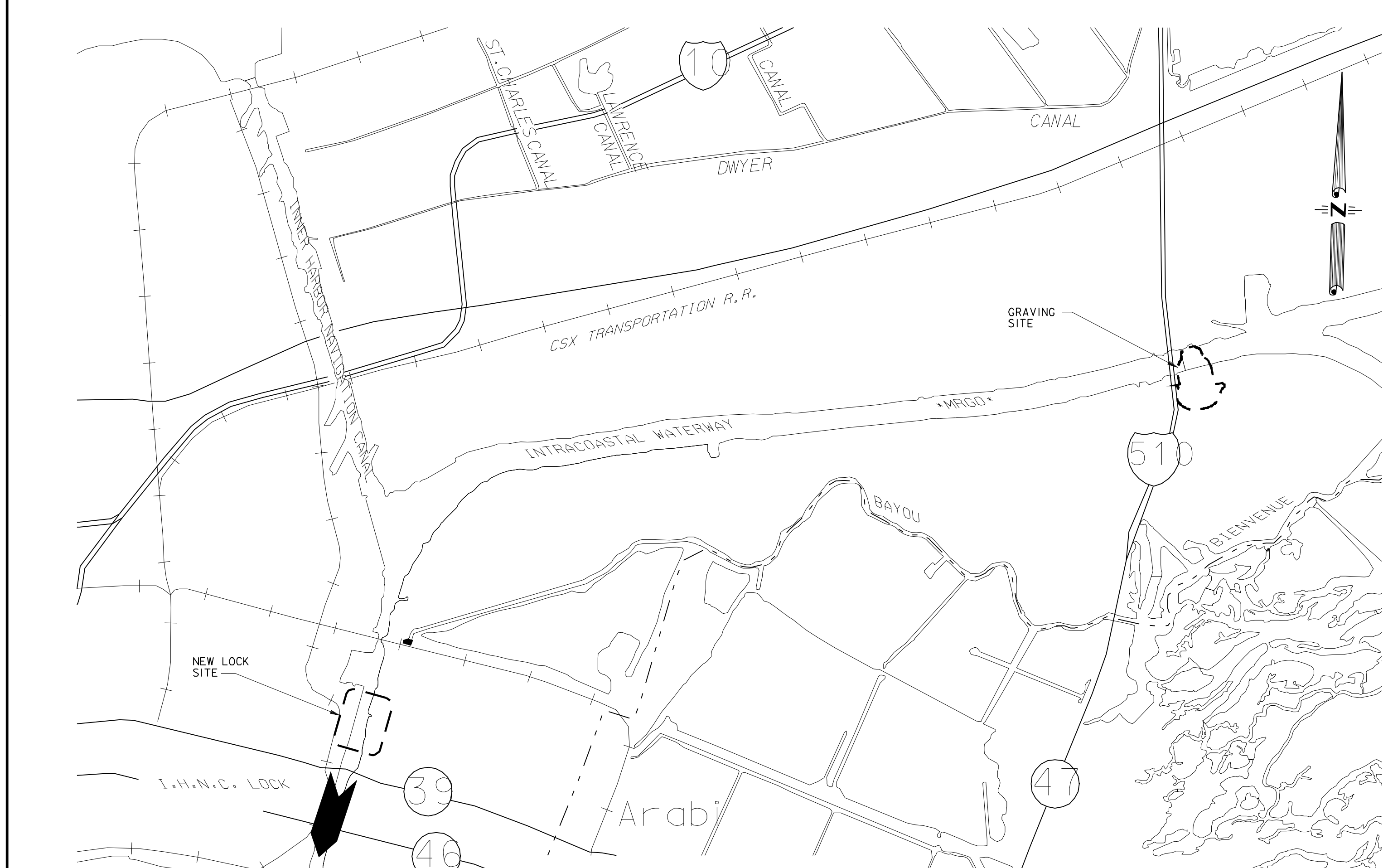


GRAVING SITE

NTS

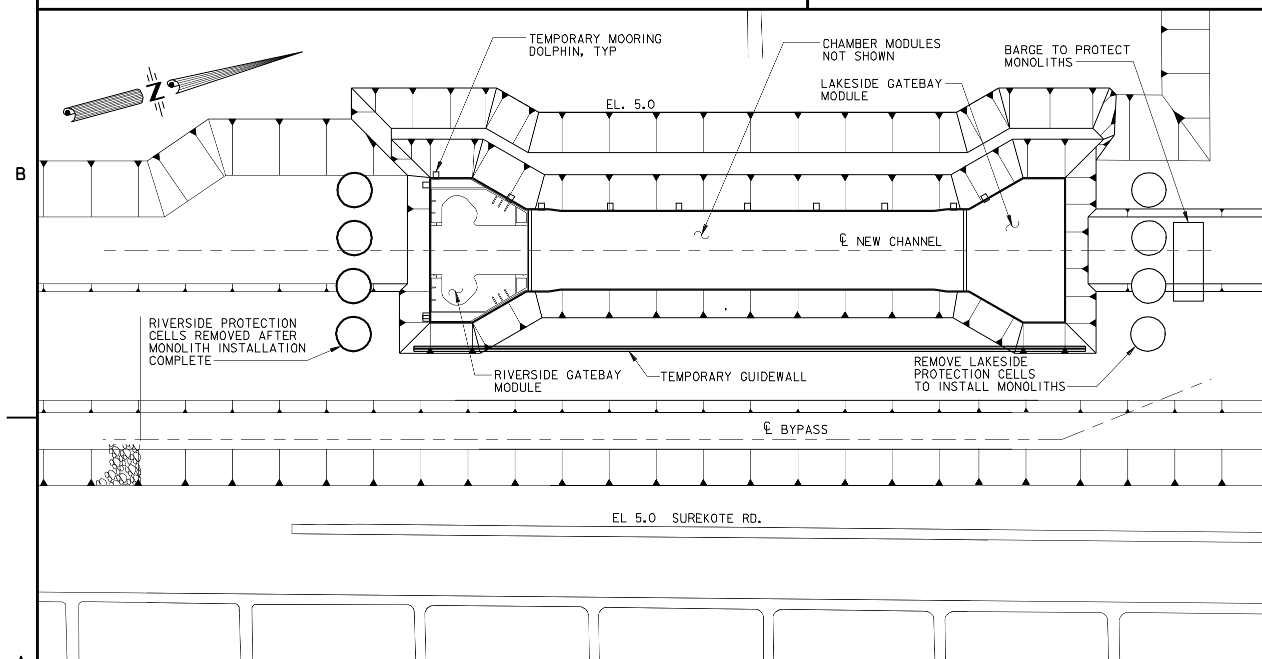
SEE NOTES 1,2,3,4,7, AND 16



GENERAL SITE PLAN

NTS

SEE NOTES 5,6,8,11,12,13, AND 15



LOCK SITE PLAN

NTS

SEE NOTES 7,9,10, AND 14

NOTES:

1. GRAVING SITE CONSTRUCTION REQUIRES TIGHT CONTROL OF CONCRETE DIMENSIONS. SEE SPECIFICATION FOR REQUIREMENTS. THE CONTRACTOR IS REQUIRED TO MAINTAIN A FLOTATION HANDBOOK DURING CONSTRUCTION. THIS HANDBOOK ACCOUNTS FOR VARIANCES IN CONSTRUCTION FROM THE CONTRACT PLANS, CONCRETE DENSITY VARIATIONS, CONTRACTOR ADDED OUTFITTING (ALLOWED UNTIL MAXIMUM TRIMMED DRAFT FOR TRANSPORT IS ATTAINED).

2. DEBONDING OF THE MODULES FROM THE CASTING SLAB IS THE CONTRACTORS RESPONSIBILITY. HIS PLAN SHALL BE SUBMITED FOR APPROVAL. POSSIBLE ADDITIONAL (SECONDARY) METHODS MAY BE PROVIDED INCLUDING SUPER FLOODING, PIPING OF WATER UNDERNEATH THE GRADE BEAMS. NOTE THE GATEBAY MODULES ARE POST-TENSIONED IN THE TRANSVERSE DIRECTION. THE POST-TENSIONING MAY BE SUFFICIENT TO DEBOND THE KEEL SLAB FROM THE GRADE BEAMS.

3. THE GRAVING SITE IS DESIGNED TO CONSTRUCT ONE MODULE AT A TIME. THE SLAB DESIGN ACCOUNTS FOR ALL MODULE'S WALL GRID LAYOUT. THE CONTRACTOR SHALL SUBMIT FOR APPROVAL HIS CONSTRUCTION SEQUENCE, INCLUDE METHODS OF WALL AND SLAB CONSTRUCTION, ORIENTATION OF MODULES WITHIN THE GRAVING SITE, CONSTRUCTION SCHEDULE.

4. THE CONTRACTOR IS ALERTED TO THE CLOSE PROXIMITY OF PARIS RD BRIDGE. LIMITATIONS EXIST ON GRAVING SITE DEWATERING TO PROTECT THE BRIDGE'S FOUNDATION. SEE SPECIFICATIONS FOR REQUIREMENTS AND LIMITATIONS.

5. THE CONTRACTOR IS TO SUBMIT FOR APPROVAL A TRANSPORT PLAN. THIS PLAN SHALL PRESENT THE METHODS OF BALLASTING, CONTROL, AND EMERGENCY PROCEDURES. EMERGENCY PROCEDURES MUST ADDRESS ACCIDENTAL CELL FLOODING, RUNAWAY BARGES, BROKEN MOORING LINES, AND HURRICANE WARNINGS.

6. THE DESIGN DRAFT OF THE MODULES DOES NOT EXCEED 28 FT. THE CONTRACTOR IS ALLOWED AN ADDITIONAL 5% DRAFT TO ACCOUNT FOR CONSTRUCTION VARIATIONS. ANY RESERVE DRAFT MAY BE UTILIZED BY THE CONTRACTOR TO TRANSPORT ADDITIONAL MATERIAL ON THE MONOLITH. PLACEMENT OF ADDITIONAL MATERIAL SHALL BE NEAR OR AT THE CENTERLINE OF THE MODULE TO MINIMIZE INCREASE OF INTERNAL FORCES IN THE MODULES.

7. TO MINIMIZE MISALIGNMENT OF THE CHAMBER FLOOR, THE CONTRACTOR IS REQUIRED TO PRODUCE A SURVEY OF THE GRAVING SITE AND OF THE COMPLETED MODULES SUFFICIENT TO LOCATE THE TOP OF LANDING PILES.

8. THE RIVER TRANSPORT IS APPROXIMATELY 6 RIVER MILES. THREE BRIDES MUST BE PASSED IN ROUTE TO THE NEW LOCK SITE. THE COE IS RESPONSIBLE FOR PROVIDING THE UNDER KEEL CLEARANCE AT THE NEW FLORIDA AVE BRIDGE.

9. THE CONTRACTOR MUST SUBMIT FOR APPROVAL AN INSTALLATION PLAN. THIS PLAN WILL DESCRIBE THE STEPS FOR PREPARING THE NEW LOCK SITE FOUNDATION AND SAFELY INSTALLING THE MODULES DURING LOW WATER AND HIGH WATER CONDITIONS. THE PLAN SHALL INCLUDE DETAILS FOR MOORING, CONTROL, AND FENDERING. THE CONTRACTOR IS ALERTED THAT NO TRAFFIC WILL BE ALLOWED ON THE IHNC WHILE TRANSPORTING MODULES FROM THE GRAVING SITE TO THE NEW LOCK SITE AND WHILE SETTING THE MODULES. TRAFFIC THROUGH THE BYPASS MAY OCCUR DURING CONSTRUCTION OF THE MODULE UPPER WALLS.

10. THE SEQUENCE OF SETTING THE MONOLITHS IS ASSUMED TO BE GB-1,CM-2, CM-3, CM-4, AND GB-5. THE CONTRACTOR IS ALERTED FOR THE POTENTIAL BUILDUP OF MISALIGNMENT DUE TO THIS SEQUENCE. THE CONTRACTOR MUST SUBMIT HIS PLAN FOR CONTROLLING LOCK ALIGNMENT.

11. THE MODULES ARE NOT DESIGNED FOR IMPACT LOADS DURING TRANSPORT OR INSTALLATION. THE CONTRACTOR IS ADVISED TO TAKE MEASURES DURING THESE ACTIVITIES TO PROTECT THE MODULES FROM DAMAGE.

12. HURRICANES POSE A SUBSTANTIAL THREAT TO THE FLOATING MONOLITHS. THE CONTRACTOR WILL NOT BE ALLOWED TO TRANSPORT MODULES DURING THE HURRICANE SEASON. TRANSPORT OF THE MODULES IS PROHIBITED FROM JUNE 1 TO NOVEMBER 31.

13. MODULE DRAFT MAY BE AFFECTED BY THE ACCUMULATION OF RAIN IN THE MODULE CHAMBERS. THE CONTRACTOR IS TO SUBMIT FOR APPROVAL A METHOD FOR REMOVING RAIN WATER FROM THE CHAMBER PRIOR TO AND DURING TRANSPORT.

14. THE CONTRACTOR IS REQUIRED TO SUBMIT FOR APPROVAL COMPLETE AND DETAILED AS-BUILT DIMENSIONS OF THE MONOLITHS AND THEIR CORRESPONDING FLOATATION HANDBOOK PRIOR TO TRANSPORT AND INSTALLATION.

15. THE CONTRACTOR IS ALERTED THAT THE IHNC IS SUBJECT TO SCOUR AND SILT ACCUMULATION ALSO HOURLY AND DAILY FLUCTIONS IN THE RIVER ELEVATION OCCUR. THE SEVERITY OF WHICH MAY VARY ACCORDING TO THE SEASONS.

16. BULKHEAD OUTFITTING FOR MODULE TRANSPORT:

GRAVING SITE CYCLE		BULKHEAD OUTFITTING
1.	RIVERSIDE GATEBAY MODULE	MAINTENANCE BULKHEADS
2.	RIVERSIDE CHAMBER MODULE	TRANSPORT BULKHEADS
3.	CENTER CHAMBER MODULE	TRANSPORT BULKHEADS
4.	LAKESIDE CHAMBER MODULE	TRANSPORT BULKHEADS
5.	LAKESIDE GATEBAY MODULE	MAINTENANCE BULKHEADS

This drawing has been reduced to half size

C·A·D

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JULY 2006

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3500 N. Causeway Blvd., Suite 900
Metairie, Louisiana 70002
(504) 837-6326
PROJECT No. 10000586.00000

US Army Corps of Engineers
New Orleans District

MARK | DESCRIPTION | DATE | APPR. | MARK | DESCRIPTION | DATE | APPR.

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: JC
CHECKED BY: DA
DRAWN BY: VR
DATE: 09-02-05

PLOT SCALE: 1600
PLOT DATE: JUNE06
DESIGN FILE NAME: $FILEABBREV$
SUBMITTED BY: JC DESIGN ENGINEER
SOLICITATION NO. W912P8-

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

CONSTRUCTION SEQUENCE
GENERAL NOTES, SHT 1 OF 2

FILE NUMBER
H-2-46087
DWG. 16-1

EXHIBIT FIP-15

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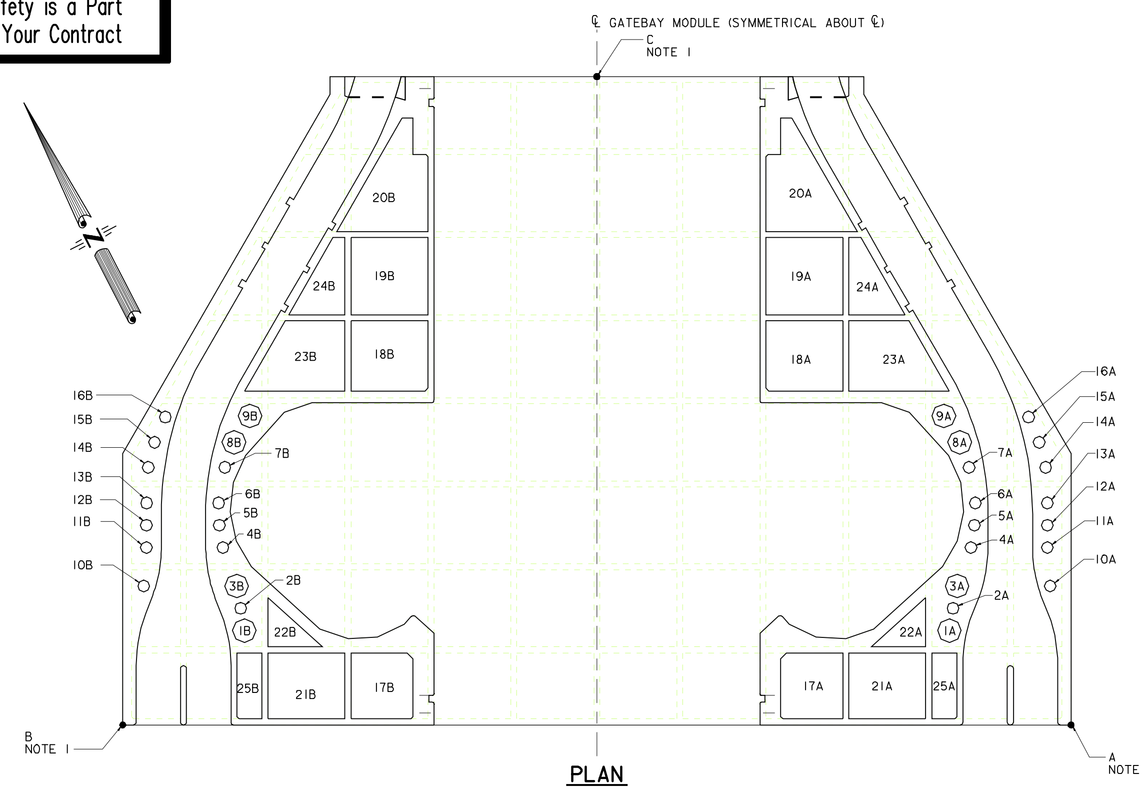
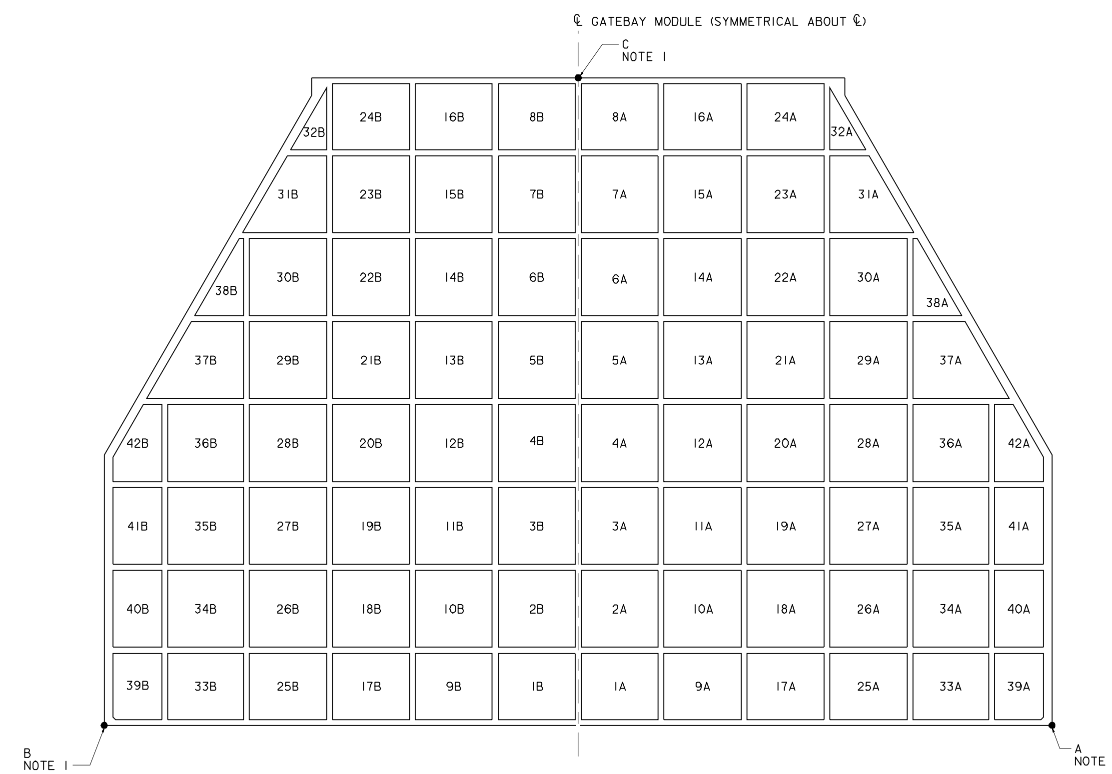


PLAN
UPPER CELLS
SCALE: 1" = 20'

20' 0 20' 40' 60' 80'



PLAN
BASE CELLS
SCALE: 1" = 20'

20' 0 20' 40' 60' 80'

		INFILL SEQUENCE (NOTE 4)												
	LIFT													
BASE CELLS	1	2	3	4	5	6	7	8	9	10	11	12	13	
	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	NOTE 1
1		5	5											
2		5	5											
3		5	5											
4		5	5											
5		5	5											
6		5	5											
7		5	5											
8		5	5											
9	5	5												
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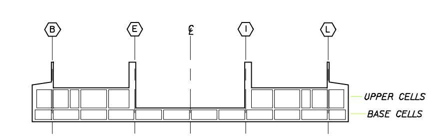
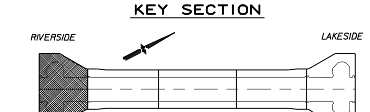
37				5	5									
38				5	5									
39				5	5									
40				5	5									
41				5	5									
42				5	5									
UPPER CELLS														NOTE 2 NOTE 3
1 TUBE										5	5	5	3.25	
2 TUBE										5	5	5	3.25	
3 TUBE										5	5	5	3.25	
4 TUBE										5	5	5	3.25	
5 TUBE										5	5	5	3.25	
6 TUBE										5	5	5	3.25	
7 TUBE										5	5	5	3.25	
8 TUBE										5	5	5	3.25	
9 TUBE										5	5	5	3.25	
10 TUBE										5	5	5	3.25	
11 TUBE										5	5	5	3.25	
12 TUBE										5	5	5	3.25	
13 TUBE										5	5	5	3.25	
14 TUBE										5	5	5	3.25	
15 TUBE										5	5	5	3.25	
16 TUBE										5	5	5	3.25	
17						5	5	5	3.25					
18						5	5	5	3.25					
19						5	5	5	3.25					
20						5	5	5	3.25					
21						5	5	5	3.25					
22						5	5	5	3.25					
23						5	5	5	3.25					
24						5	5	5	3.25					
25						5	5	5	3.25					
AVE DRAFT	30	35.5	41.2	45.6	48.1	57	57	57	57	57	57	57	57	NOTE 3,6



KEY SECTION



KEY PLAN

NOTES:

1. DRAFT POINTS A, B AND C ARE FOR DETERMINING AVERAGE DRAFT.
2. UPPER CELLS ARE INFILLED WITH CONCRETE AFTER UNDERBASE TREMIE CONCRETE HAS BEEN PLACED.
3. DRAFT FOR SHELL WITH UPPER CELLS INFILLED BASED ON HIGH WATER AT EL. 3.0.
4. INFILL SEQUENCE IS SYMMETRICAL. ALTERNATE LIFTS BETWEEN "A" CELLS AND "B" CELLS.
5. DRAFT BEFORE INFILL IS 28.2 FT

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JULY 2006

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PROJECT No. 10000586.00000

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: JC
CHECKED BY: DA
DRAWN BY: VH
DATE: 09-02-05
PLOT SCALE: 240
PLOT DATE: JUNE06
SOLICITATION NO. W912P8-

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

INFILL SEQUENCE PLAN

FILE NUMBER H-2-46087
DWG. 16-6

EXHIBIT FIP-16

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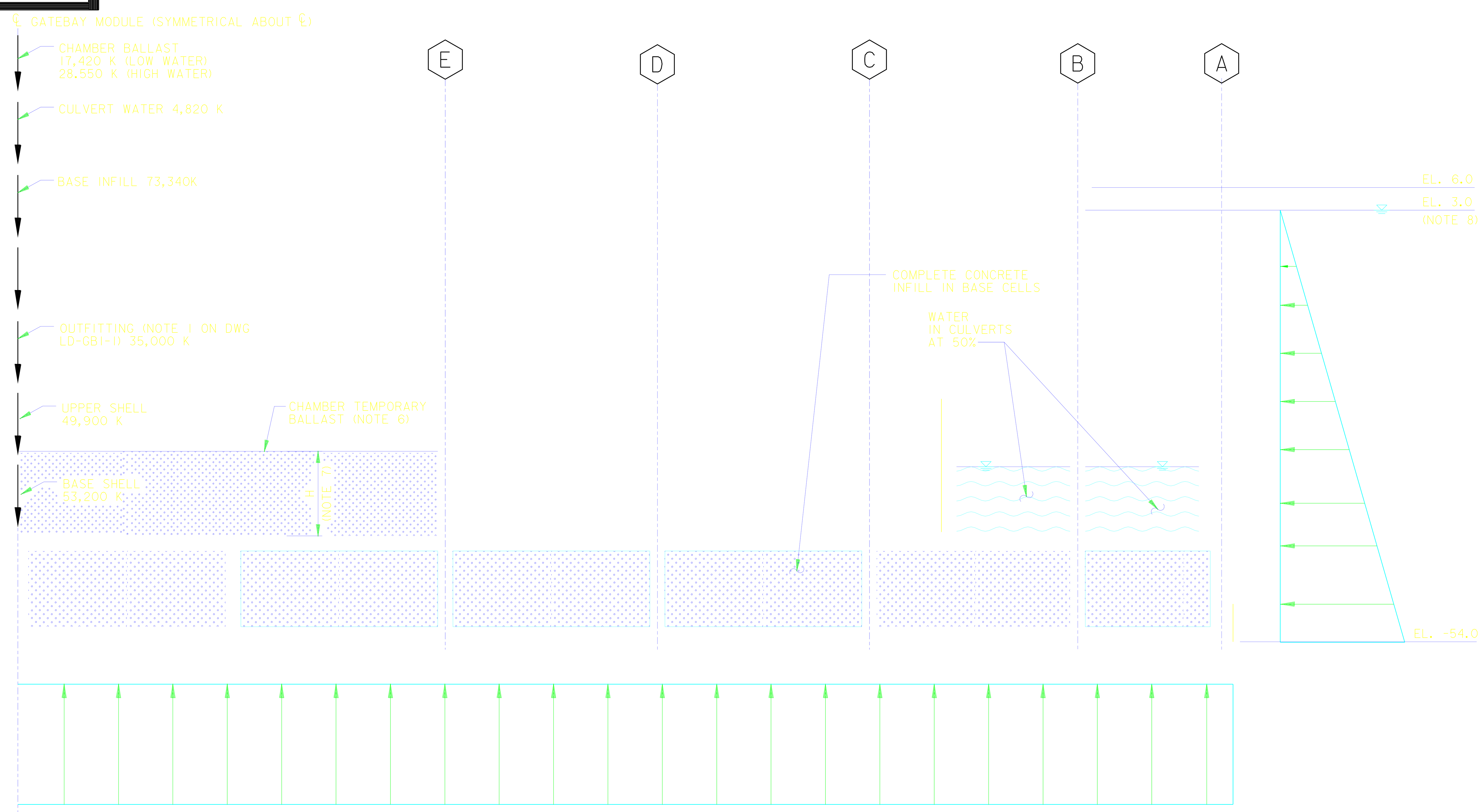
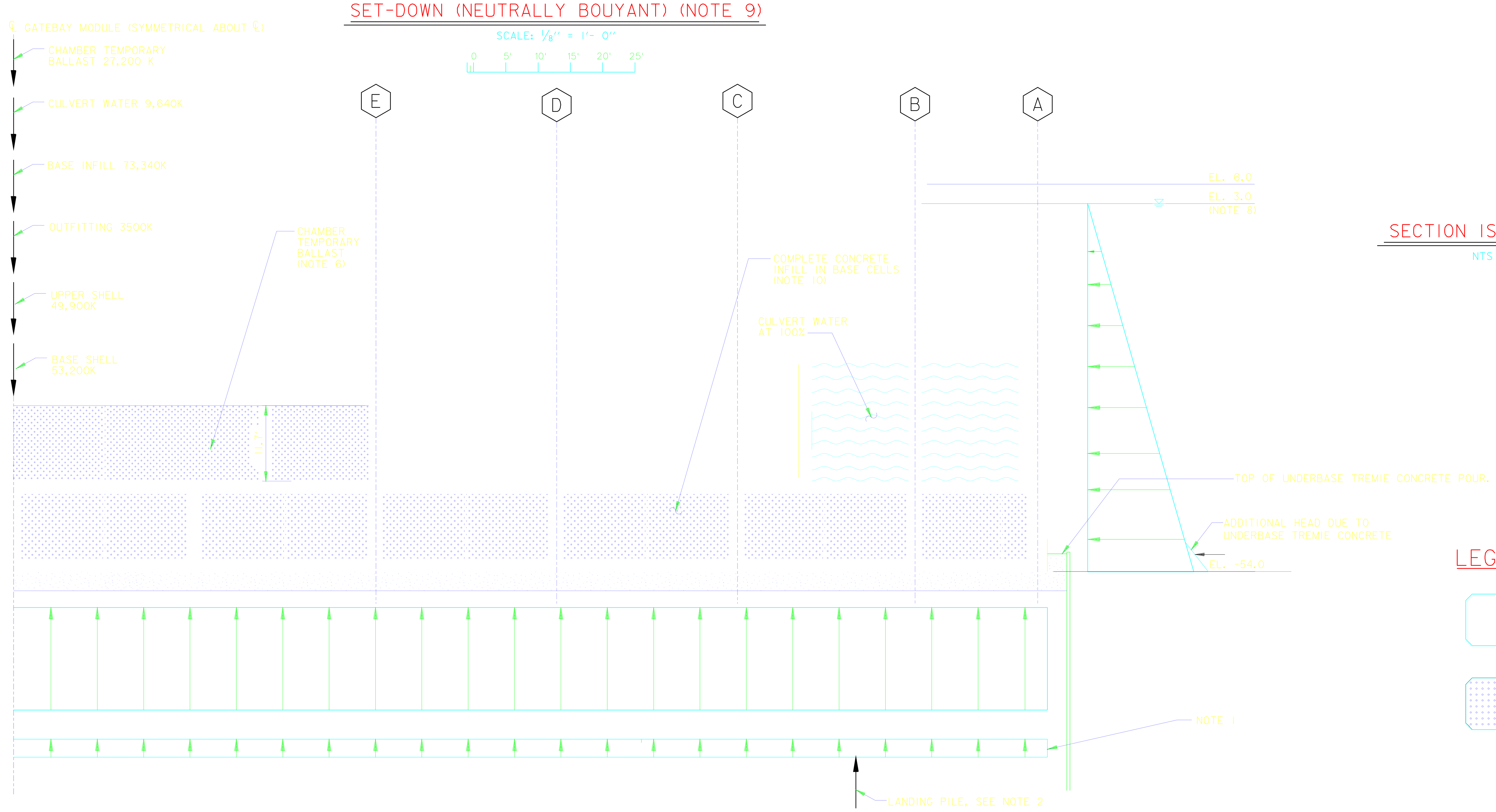


SET-DOWN (NEUTRALLY BOUYANT) (NOTE 9)

SCALE: 1/8" = 1'- 0"



UNDERBASE TREMIE CONCRETE PLACEMENT (NOTE 9)

SCALE: 1/8" = 1'- 0"

NOTES:

1. SOIL PRESSURE TEMPORARILY SUPPORTS THE WEIGHT OF THE UNDERBASE TREMIE AND MODULE.
2. WITH WATER ELEVATION AT EL. 1.5 THERE IS A 5% NEGATIVE BOUYANCY REACTION (10,375 KIPS) ON THE LANDING PILES. THIS REACTION INCREASES TO 7.76% (15,700 KIPS) WHEN THE WATER LEVEL DROPS TO EL. 0.0 AND IT DECREASES TO 2.36% (5,025 KIPS) WHEN THE WATER LEVEL RISES TO EL. 3.0
3. WAVE LOADS NOT SHOWN.
4. FOUNDATION PILES NOT SHOWN.
5. REFER TO ADDITIONAL NOTES ON LD-GBI-1.
6. TEMPORARY BALLAST IN THE CHAMBER IS ASSUMED TO BE MOIST SAND $\gamma = 110\ LB/FT^3$
7. DEPTH OF SAND AT CONTACT WITH LANDING PILES
   H = 7.2 FT LOW WATER
   H = 11.8 FT HIGH WATER
8. SET-DOWN MAY OCCUR WITHIN THE FOLLOWING POOL ELEVATIONS:
   LOW WATER EL. 1.0
   HIGH WATER EL. 3.0
9. LOADS SHOWN DEPEND ON CONTRACTORS INSTALLATION PLAN AND MAY VARY. SEE LOADS ON SHEET LD-GB5-26 FOR LOADS BASED ON AN ALTERNATIVE SETDOWN METHOD. CONTRACTOR TO SUBMIT FOR APPROVAL LOAD SUMMARY FOR INTENDED SETDOWN PROCEDURE FOR APPROVAL.
10. REFER TO DWG 16-6 FOR INFILL SEQUENCE.

SECTION ISOMETRIC

NTS

LEGEND:

FLOAT-IN SHELL CONCRETE (5000 PSI)

INFILL CONCRETE (3000 PSI)

This drawing has been reduced to half size

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JULY 2006

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CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

MISSISSIPPI RIVER-GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS
INNER HARBOR NAVIGATION CANAL LOCK REPLACEMENT
ORLEANS PARISH, LOUISIANA

LOAD DIAGRAMS-SET-DOWN

FILE NUMBER H-2-46087
DWG LD-GBI-2

EXHIBIT FIP-18

NOTES

1. EXISTING CONDITION FOR GRAVING SITE AT PARIS ROAD BRIDGE.

FIP – PROPOSED GRAVING SITE

US Army Corps of Engineers
Huntington District

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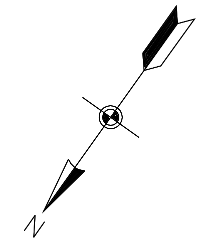
U.S. ARMY CORPS OF ENGINEERS
HUNTINGTON DISTRICT
HUNTINGTON, WEST VIRGINIA

MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
GRAVING SITE
EXISTING CONDITIONS

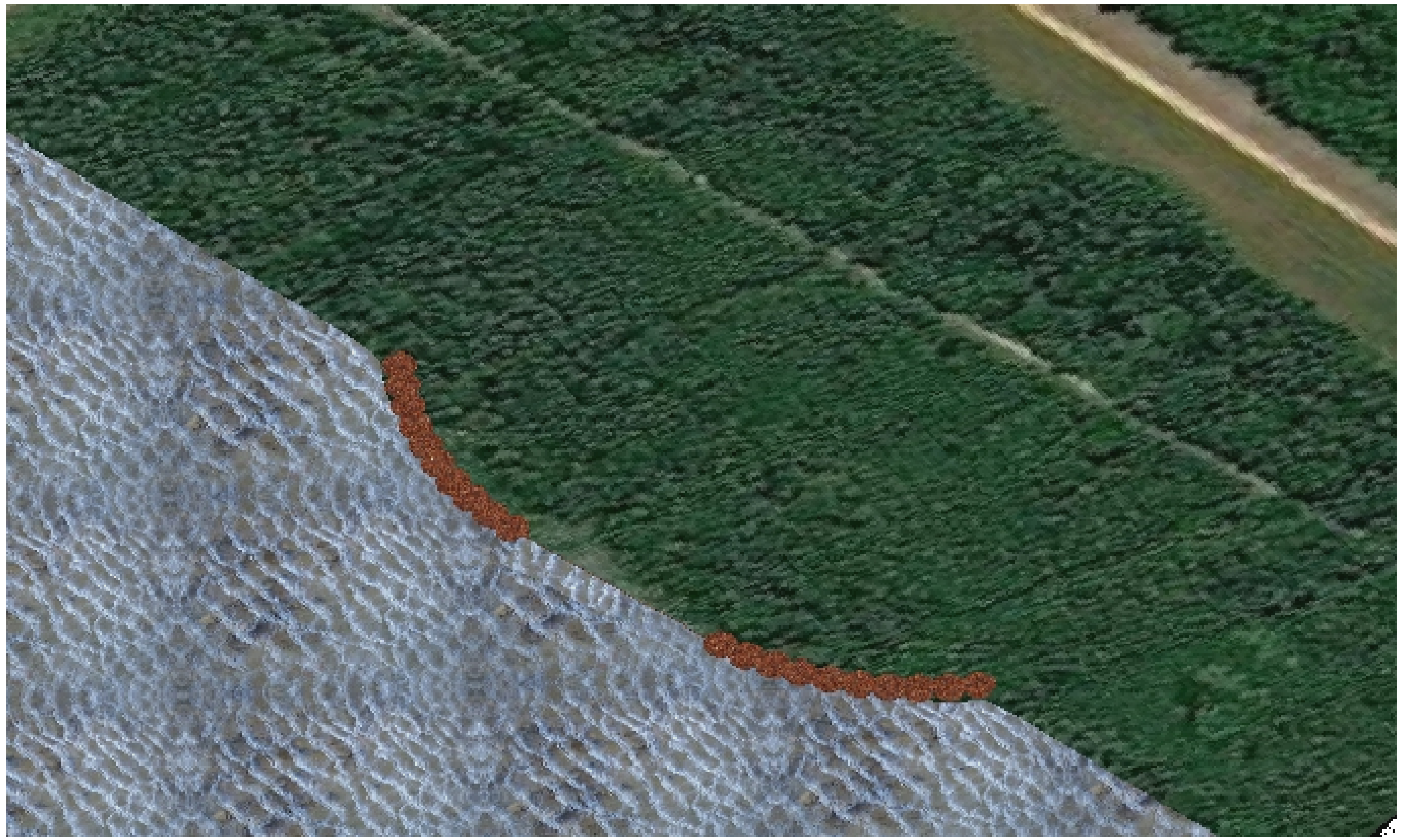
Sheet reference number:
GI901
Sheet 0 of

EXHIBIT FIP-19



NOTES

1. 30' DIAMETER CELLS FOR CLOSURE STRUCTURE.
2. 200' WIDE DIAPHRAM WALL WITH ROCK.



FIP – GRAVING SITE CLOSURE SYSTEM

US Army Corps of Engineers Huntington District

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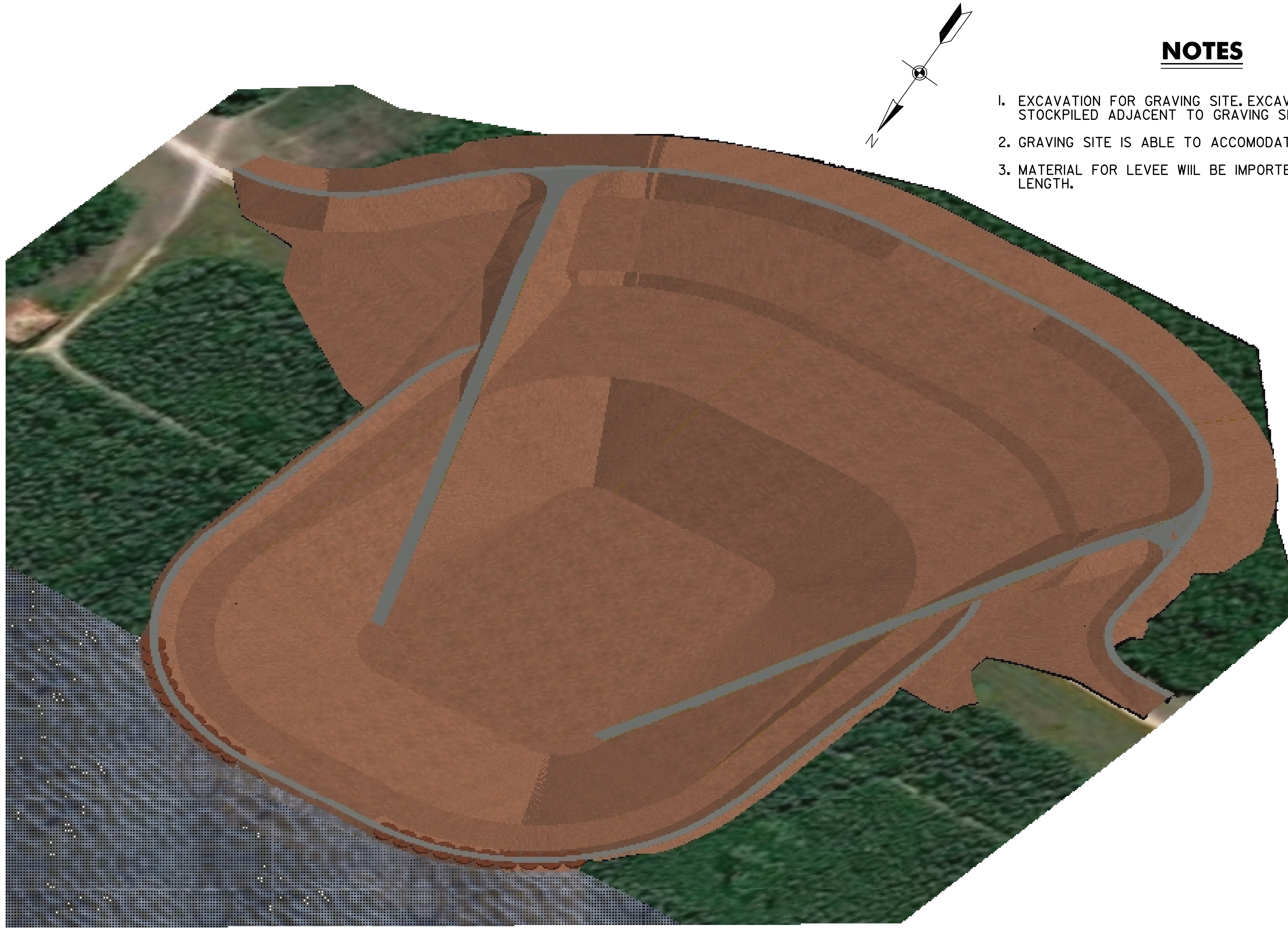
HUNTINGTON, WEST VIRGINIA

MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
GRAVING SITE
EXCAVATION FOR CELLS AND SHEET PILE

Sheet reference number:
GI902
Sheet 0 of

EXHIBIT FIP–20



NOTES

1. EXCAVATION FOR GRAVING SITE. EXCAVATION WILL BE STOCKPILED ADJACENT TO GRAVING SITE.
2. GRAVING SITE IS ABLE TO ACCOMODATE ONE MODULE.
3. MATERIAL FOR LEVEE WIIL BE IMPORTED FOR EXTENDED LENGTH.

FIP – GRAVING SITE EXCAVATED

US Army Corps of Engineers
Huntington District

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MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

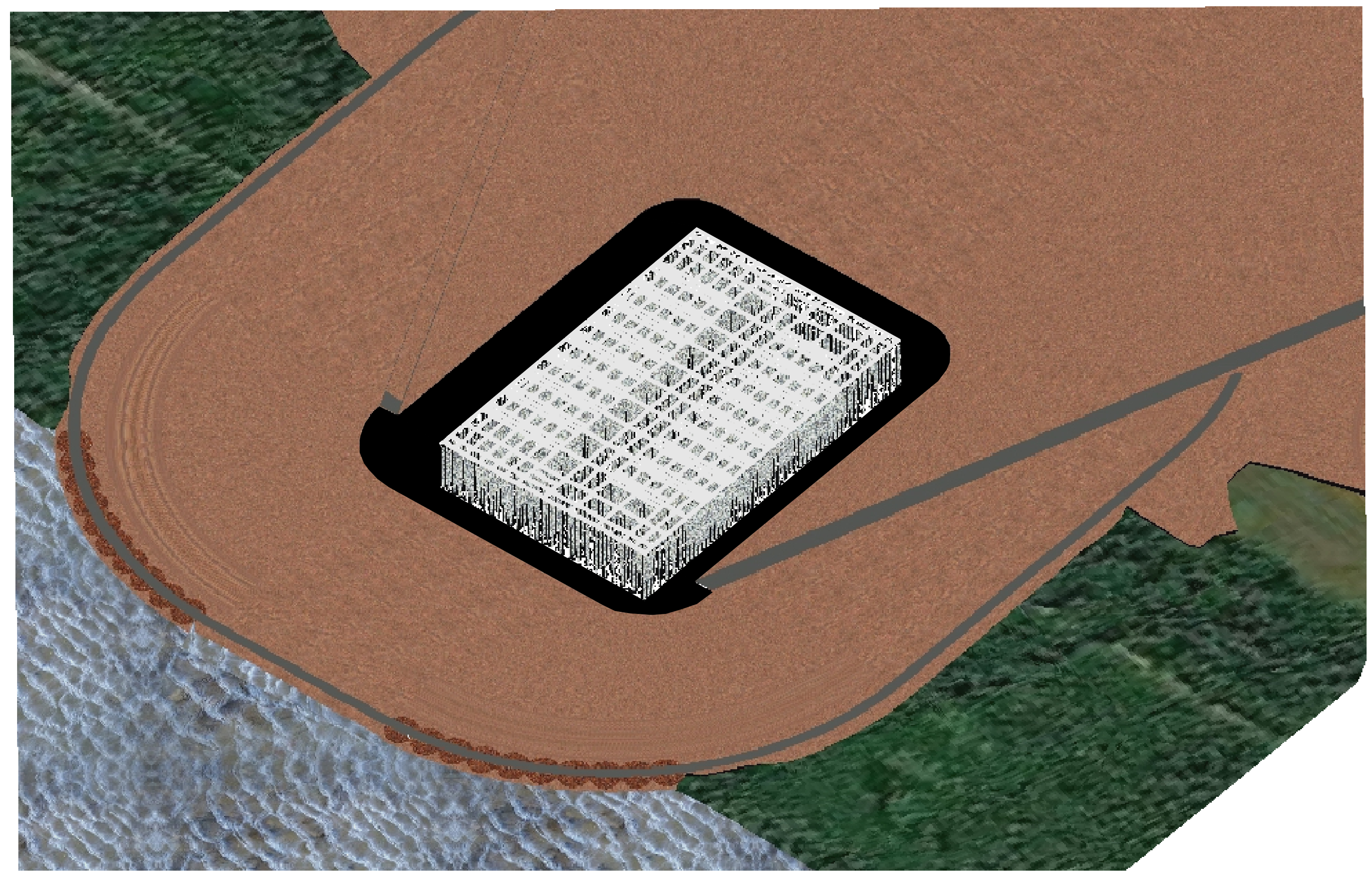
FLOAT IN PLACE
GRAVING SITE
GRAVING SITE EXCAVATION

Sheet reference number:
GI903
Sheet 0 of

EXHIBIT FIP–21

NOTES

1. PILING AND GRADE BEAMS INSTALLED



FIP – GRAVING SITE FOUNDATION

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HUNTINGTON DISTRICT
HUNTINGTON, WEST VIRGINIA

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MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

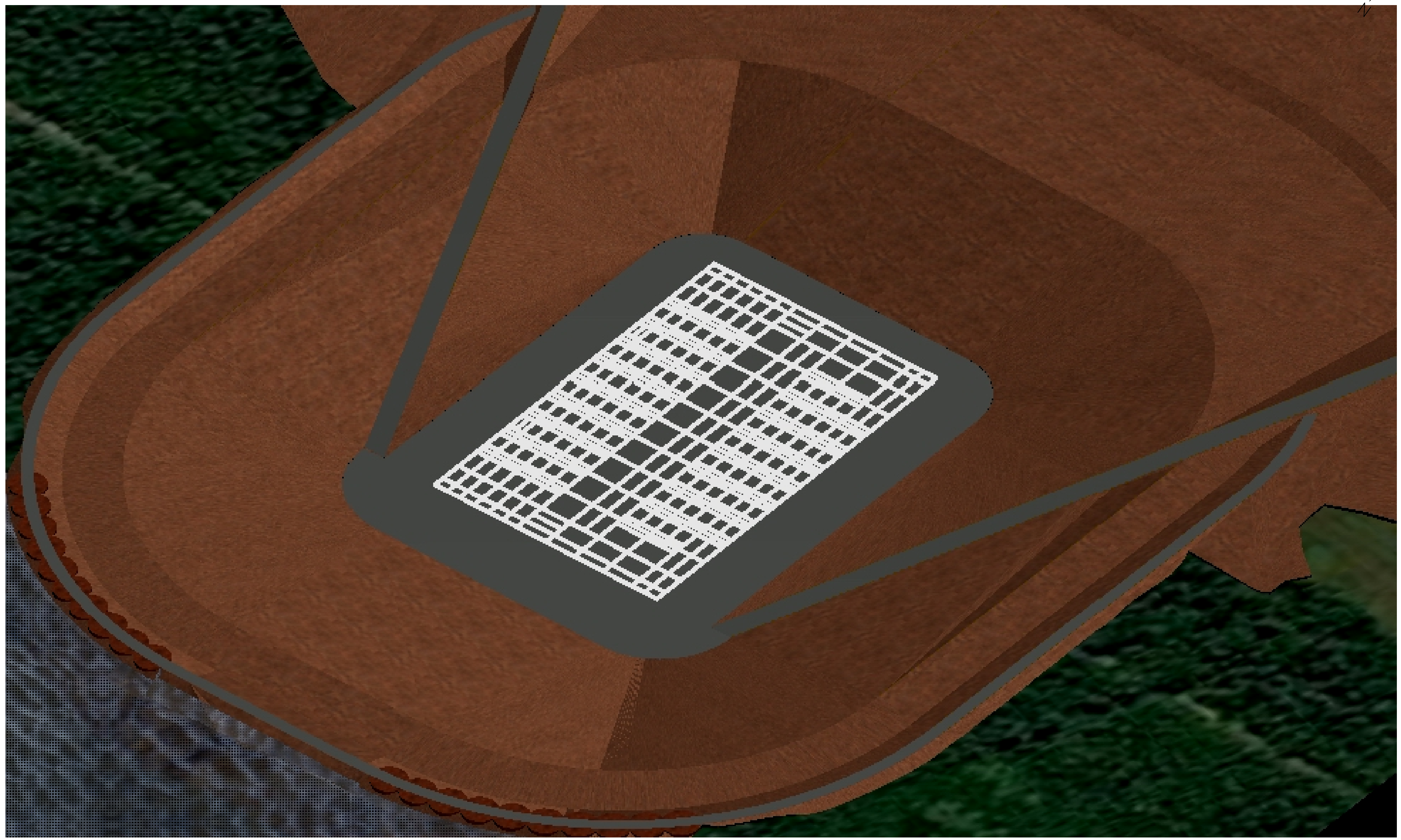
FLOAT IN PLACE
GRAVING SITE
FOUNDATION PILES/GRADE BEAMS

Sheet reference number:
GI904
Sheet 0 of

EXHIBIT FIP-22

NOTES

1. GRAVEL PLACED BETWEEN GRADE BEAMS FOR LEVEL SURFACE FOR CASTING MODULES



FIP – GRAVING SITE READY FOR MODULE CONSTRUCTION

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HUNTINGTON DISTRICT
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MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
GRAVING SITE
FINAL CONSTRUCTION

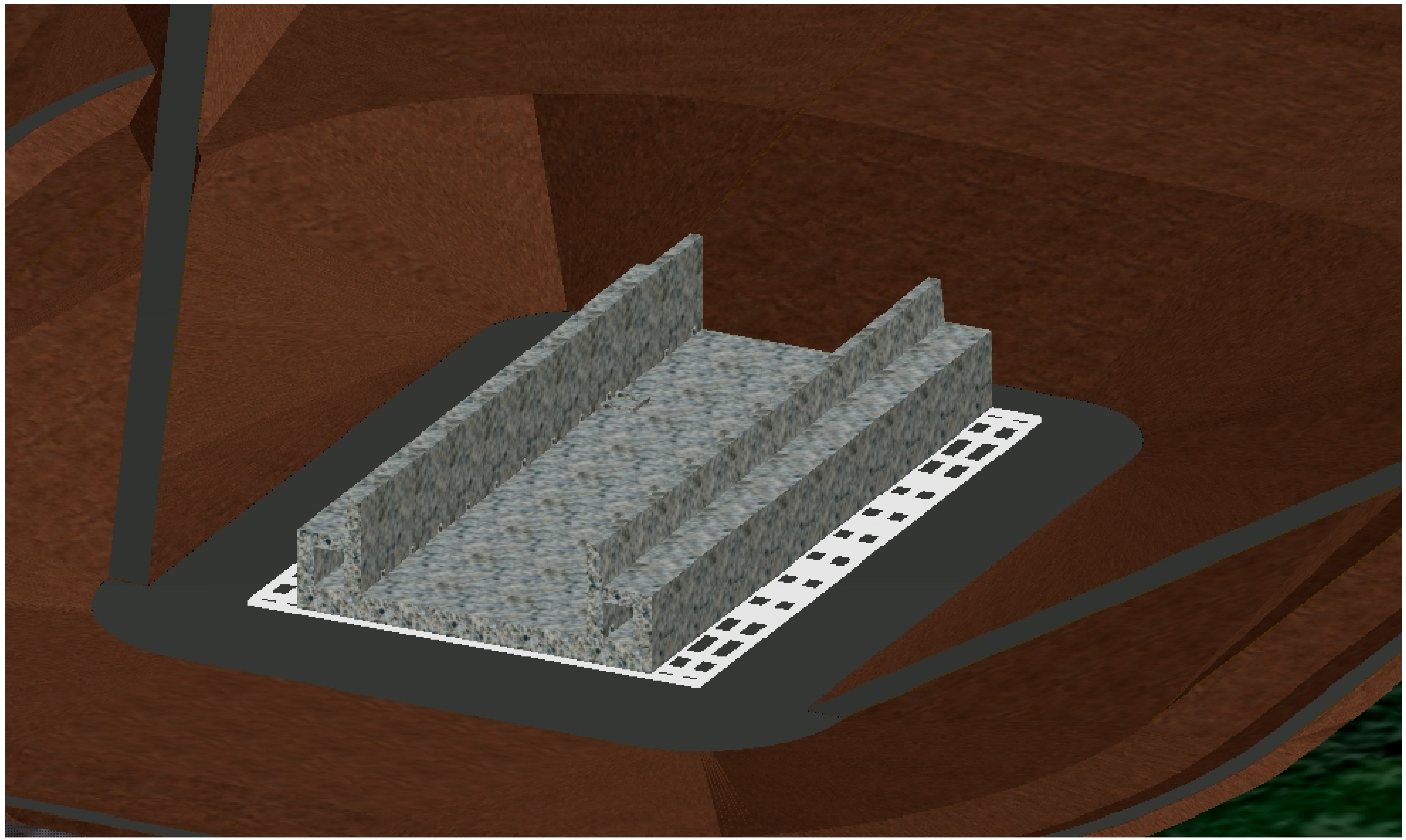
Sheet reference number: GI905

Sheet 0 of

EXHIBIT FIP–23

NOTES

1. CHAMBER MODULE FABRICATED



FIP – CONSTRUCTION OF CHAMBER MODULE

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Huntington District

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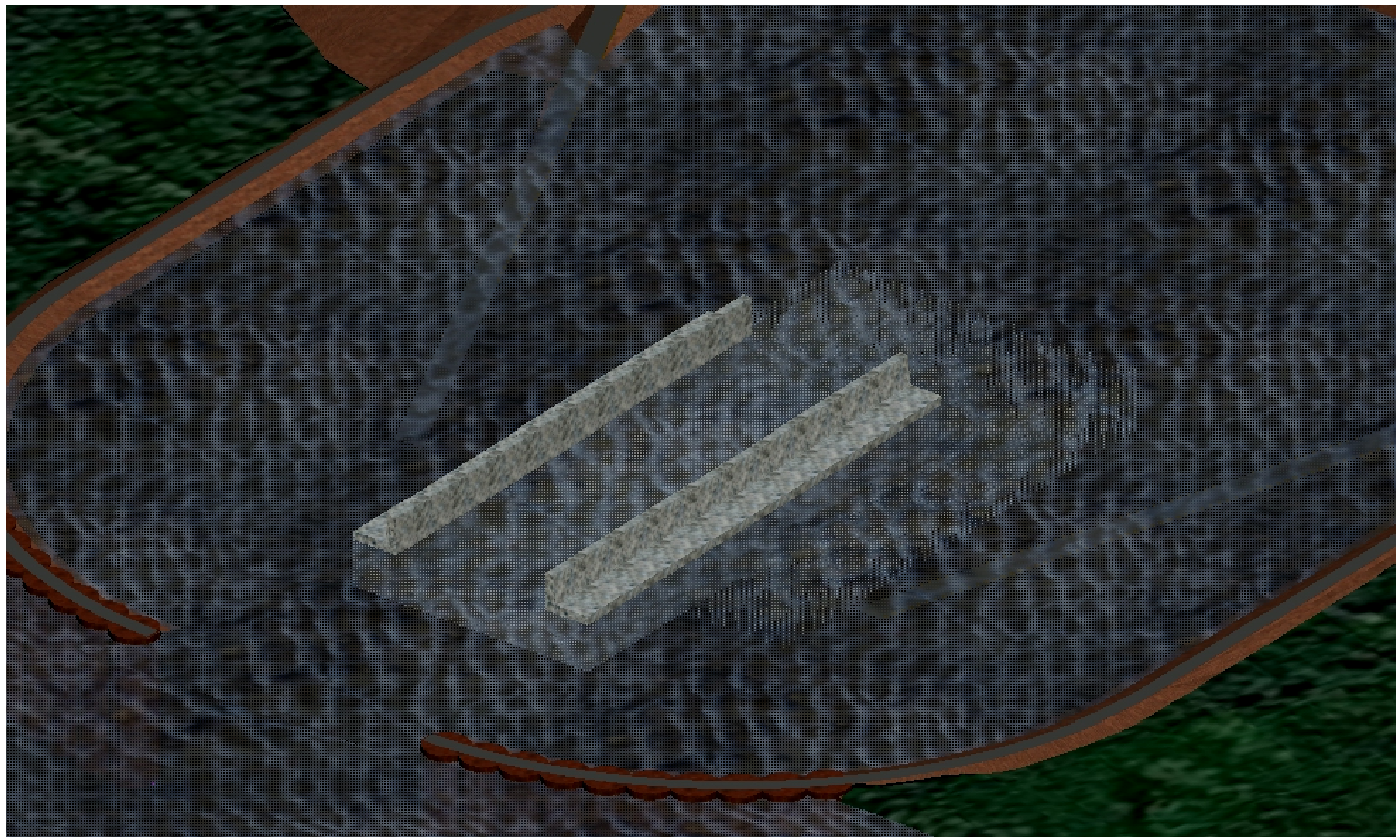
MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
GRAVING SITE
CHAMBER CONSTRUCTION

Sheet reference number:
GI906
Sheet 0 of

NOTES

1. WATER UP GRAVING SITE
2. FLOAT MODULE OFF
3. REMOVE CLOSURE



FIP – GRAVING SITE WATERED UP

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HUNTINGTON, WEST VIRGINIA

MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

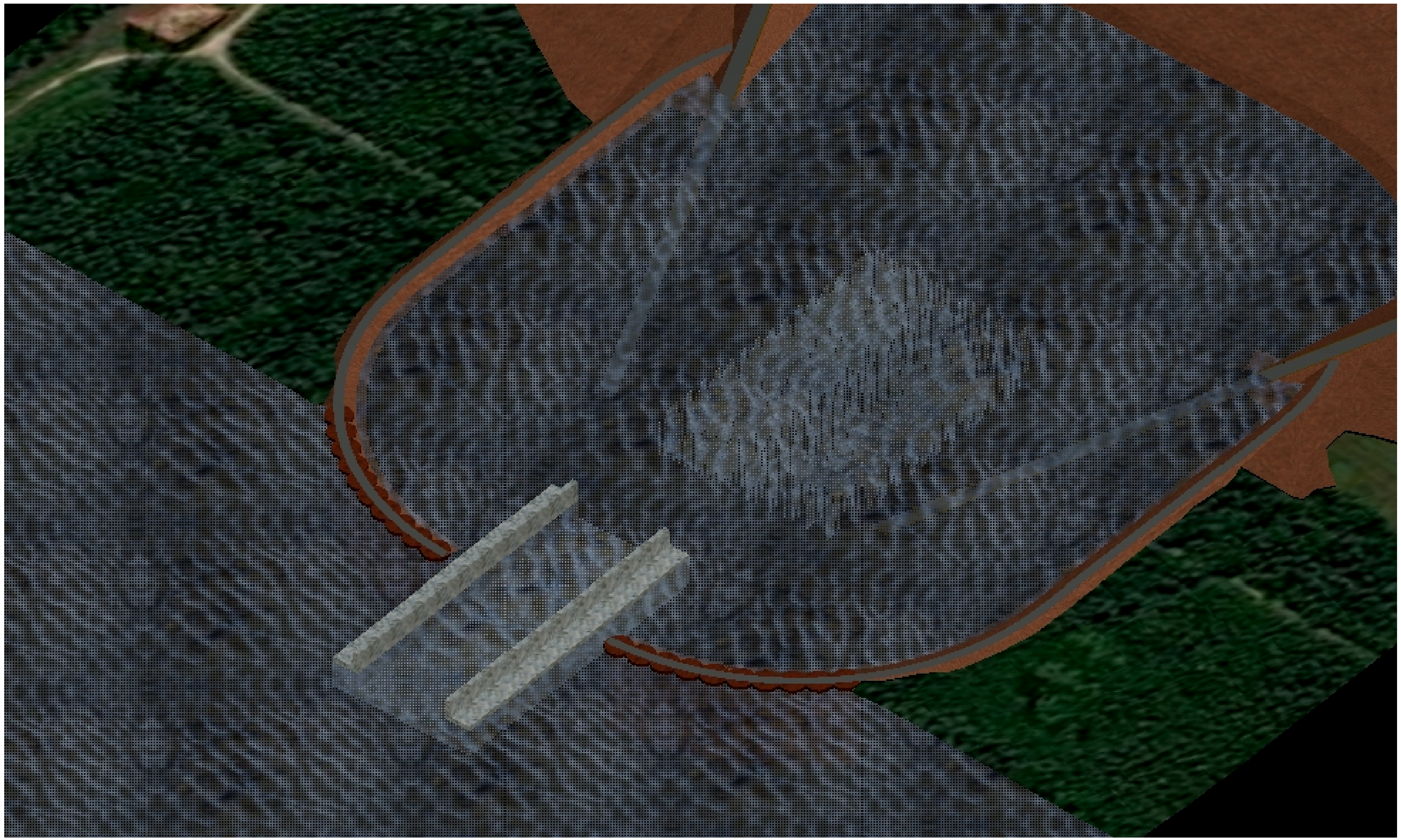
FLOAT IN PLACE
GRAVING SITE
RE-WATERED, CHAMBER AFLOAT

Sheet reference number:
GI907
Sheet 0 of 8

EXHIBIT FIP–25

NOTES

1. WITH CLOSURE REMOVED, MODULE IS FLOATED OUT OF GRAVING SITE.



FLOATING CHAMBER OUT OF GRAVING STIE

US Army Corps of Engineers Huntington District

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HUNTINGTON DISTRICT
HUNTINGTON, WEST VIRGINIA

MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
GRAVING SITE
RE-WATERED, CHAMBER IN GATE

Sheet reference number:
GI908
Sheet 0 of 0

EXHIBIT FIP-26

NOTES

1. EXISTING CONDITIONS
2. NOTICE TO PROCEED

FIP – EXISTING CONDITIONS

US Army Corps of Engineers
Huntington District

U.S. ARMY CORPS OF ENGINEERS

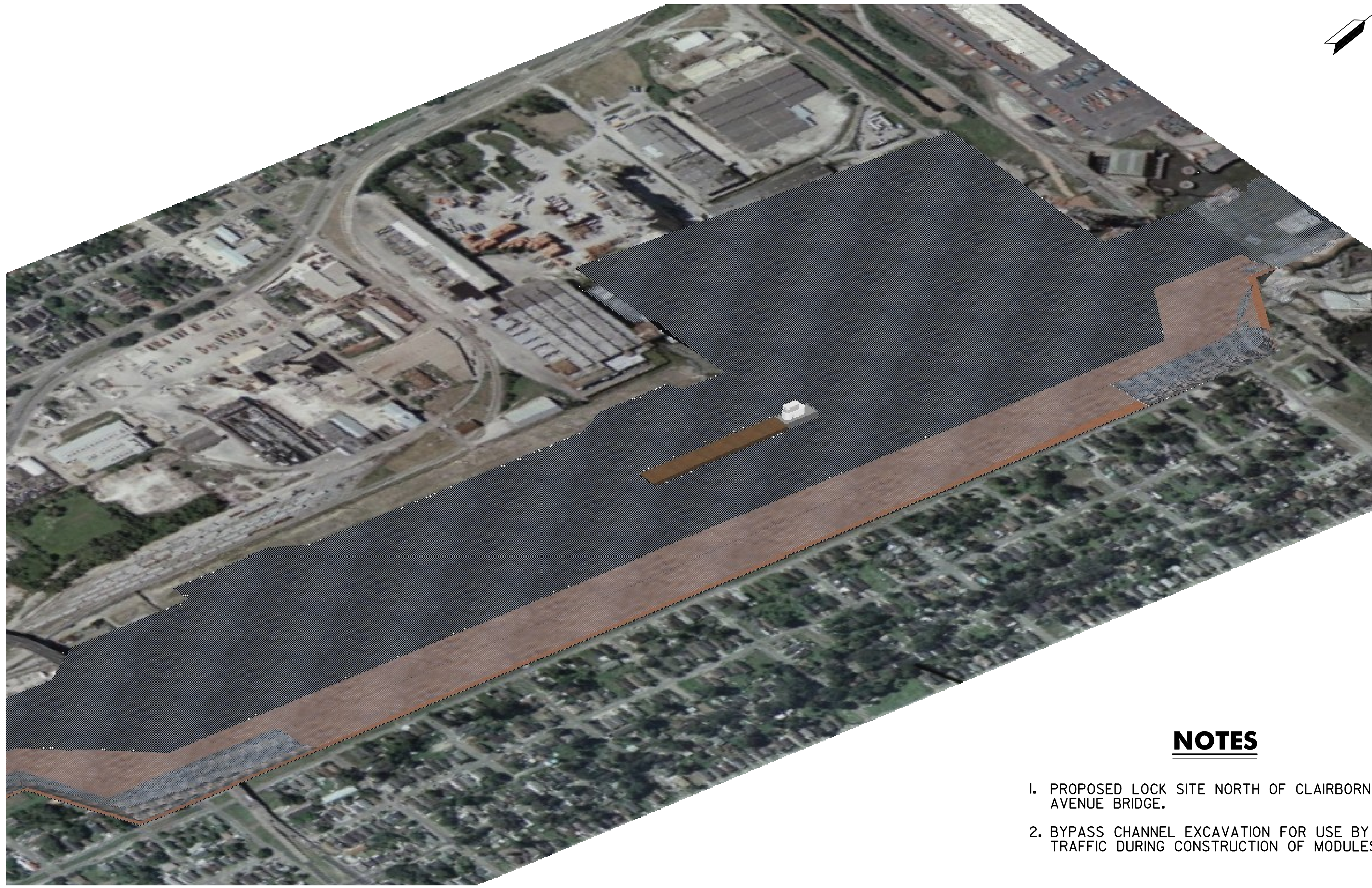
MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
EXISTING CONDITIONS

Sheet reference number:
GI603
Sheet 0 of 0

EXHIBIT FIP–27

US Army Corps of Engineers
Huntington District



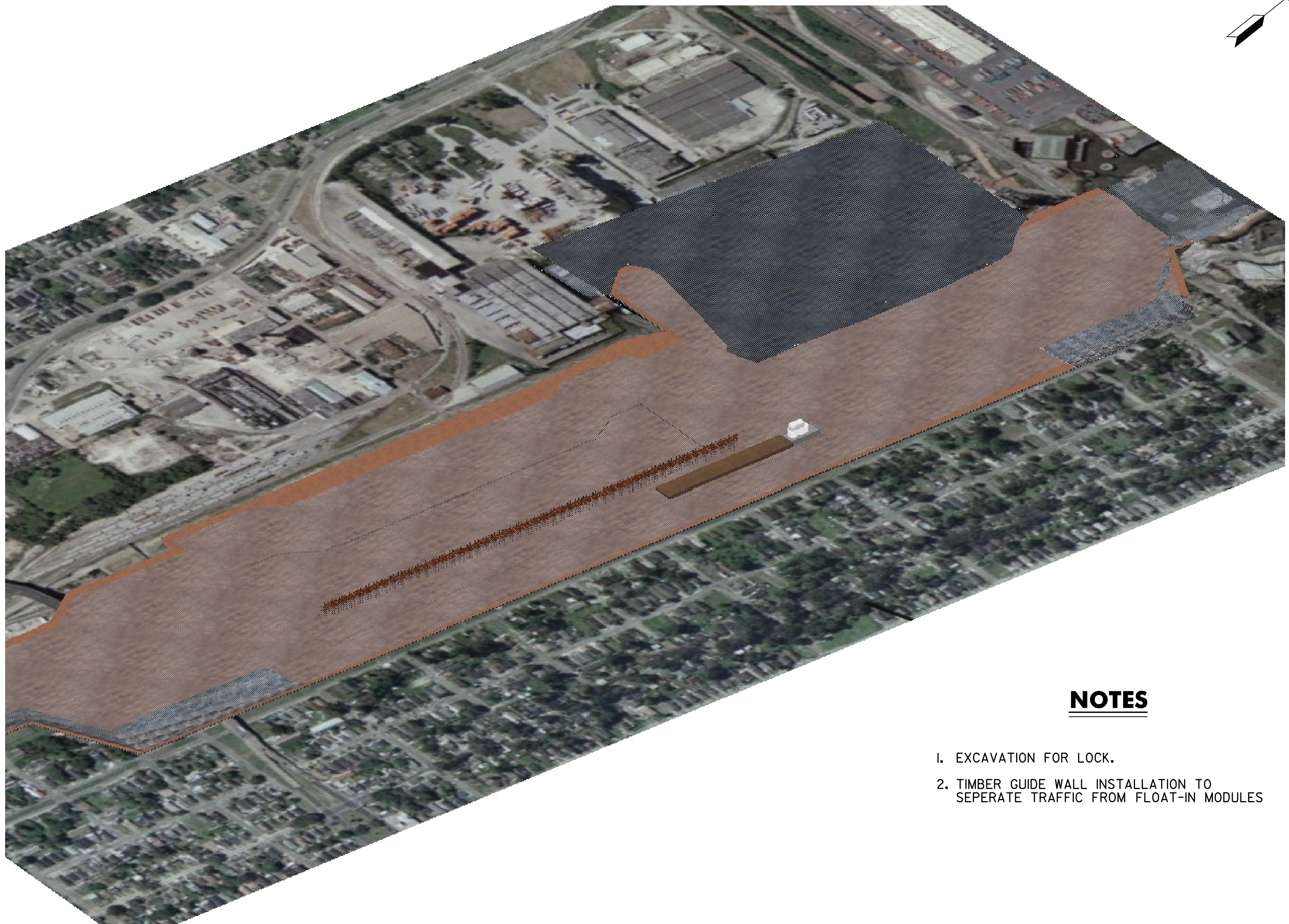
NOTES

1. PROPOSED LOCK SITE NORTH OF CLAIRBORNE AVENUE BRIDGE.
2. BYPASS CHANNEL EXCAVATION FOR USE BY TRAFFIC DURING CONSTRUCTION OF MODULES.

FIP – BYPASS CHANNEL EXCAVATION

FLOAT IN PLACE
BYPASS EXCAVATION

Sheet reference number:
GI909



NOTES

1. EXCAVATION FOR LOCK.
2. TIMBER GUIDE WALL INSTALLATION TO SEPERATE TRAFFIC FROM FLOAT-IN MODULES

FIP LOCK EXCAVATION

US Army Corps of Engineers Huntington District

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HUNTINGTON DISTRICT
HUNTINGTON, WEST VIRGINIA

MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
EAST EXCAVATION
TIMBER WALL

Sheet reference number:
G1910
Sheet 0 of 0

EXHIBIT FIP-29

NOTES

1. CONSTRUCT 8 PROTECTION CELLS AND A SINGLE BARGE PROTECTING THE LOCK SITE.
2. 48" PIPE PILES 120' LONG INSTALLED
3. TRAFFIC DIVIERTED TO BYPASS CHANNEL

FIP – COMPLETED PILING AND END CELLS

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Huntington District

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HUNTINGTON, WEST VIRGINIA

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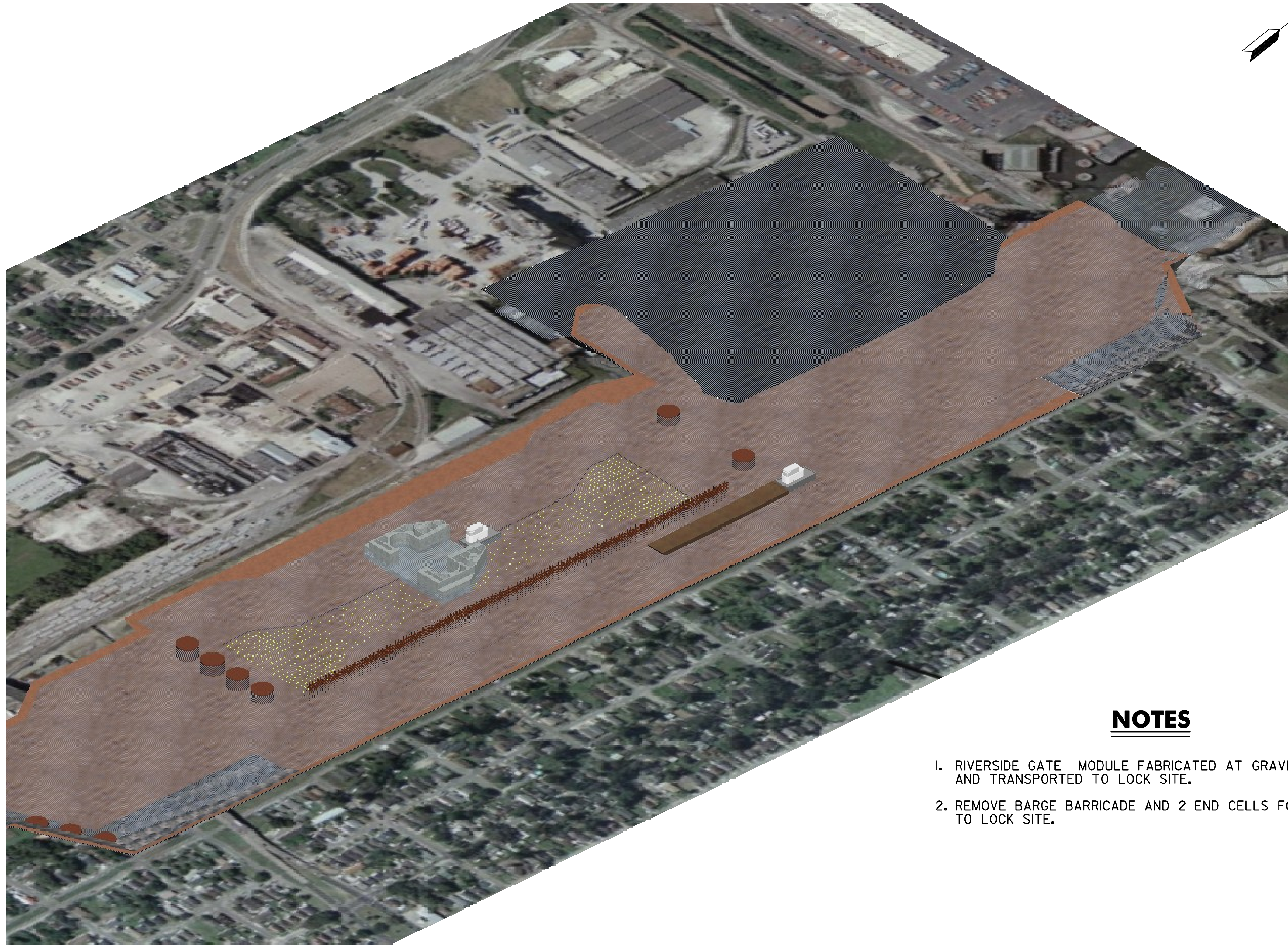
MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
CELL IN PLACE
PILING

Sheet reference number:
G1911
Sheet 0 of

EXHIBIT FIP–30



NOTES

1. RIVERSIDE GATE MODULE FABRICATED AT GRAVING SITE AND TRANSPORTED TO LOCK SITE.
2. REMOVE BARGE BARRICADE AND 2 END CELLS FOR ACCESS TO LOCK SITE.

FIP – FLOAT IN RIVER SIDE GATE BAY MODULE

US Army Corps of Engineers
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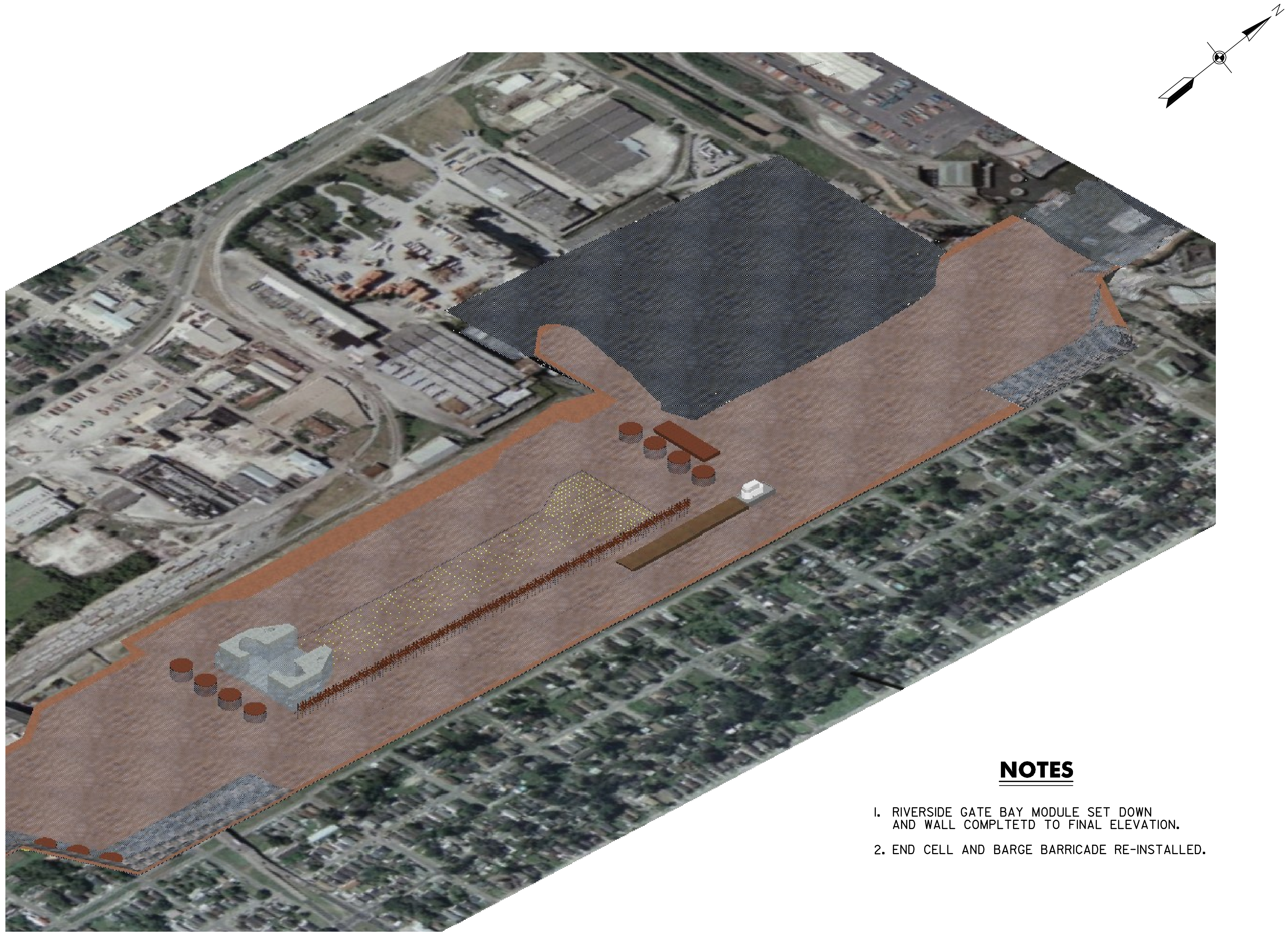
HUNTINGTON DISTRICT
HUNTINGTON, WEST VIRGINIA

MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
FLOAT RSBG

Sheet reference number:
G1912
Sheet 0 of 0

EXHIBIT FIP–31



NOTES

1. RIVERSIDE GATE BAY MODULE SET DOWN AND WALL COMPLTETD TO FINAL ELEVATION.
2. END CELL AND BARGE BARRICADE RE-INSTALLED.

FIP – RIVER SIDE GATEBAY IN PLACE

US Army Corps of Engineers Huntington District

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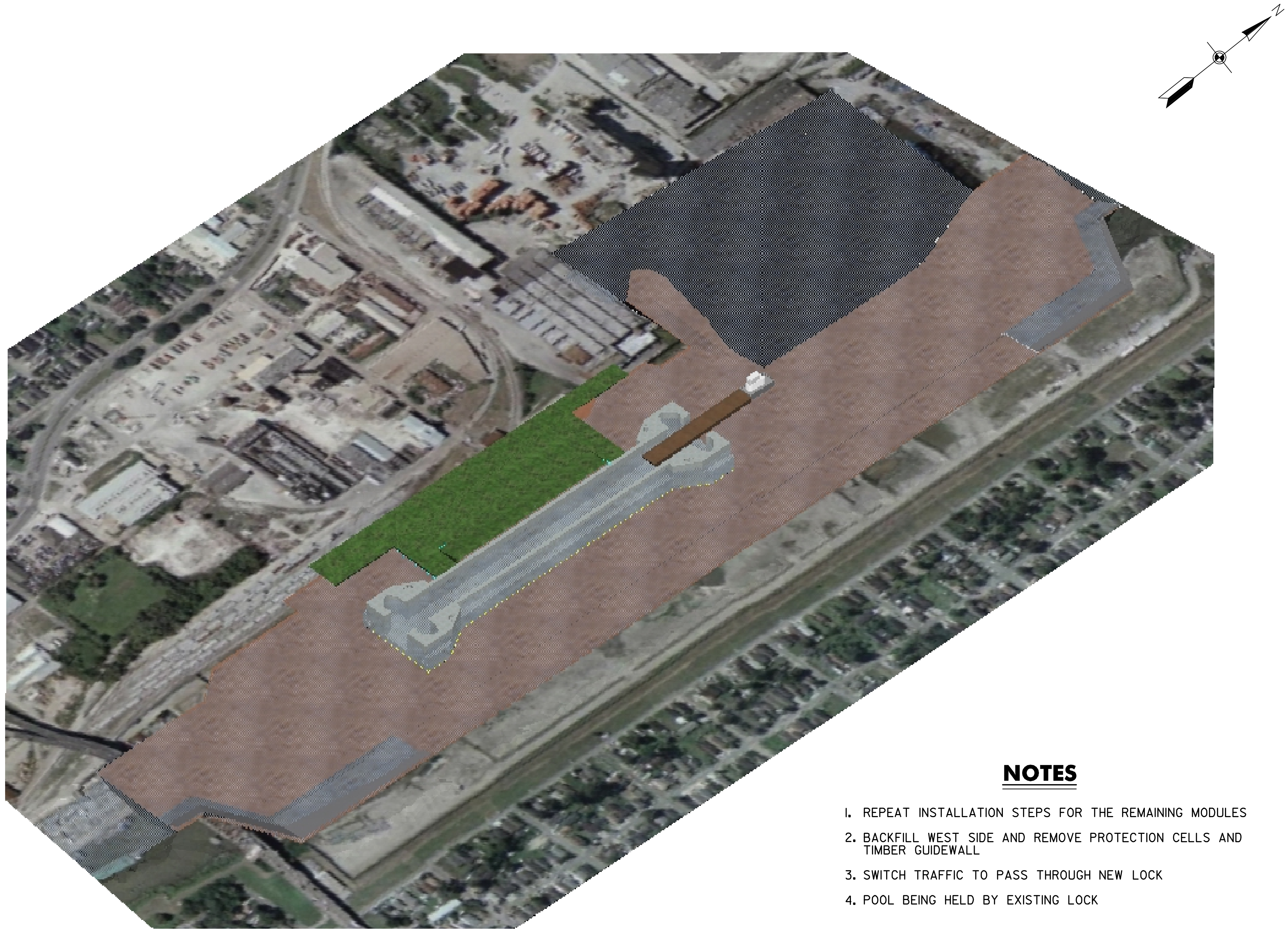
HUNTINGTON DISTRICT
HUNTINGTON, WEST VIRGINIA

MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
RIVER SIDE
GATE BAY

Sheet reference number:
G1913
Sheet 0 of

EXHIBIT FIP–32



NOTES

1. REPEAT INSTALLATION STEPS FOR THE REMAINING MODULES
2. BACKFILL WEST SIDE AND REMOVE PROTECTION CELLS AND TIMBER GUIDEWALL
3. SWITCH TRAFFIC TO PASS THROUGH NEW LOCK
4. POOL BEING HELD BY EXISTING LOCK

FIP – MODULE CONSTRUCTION COMPLETED

US Army Corps of Engineers
Huntington District

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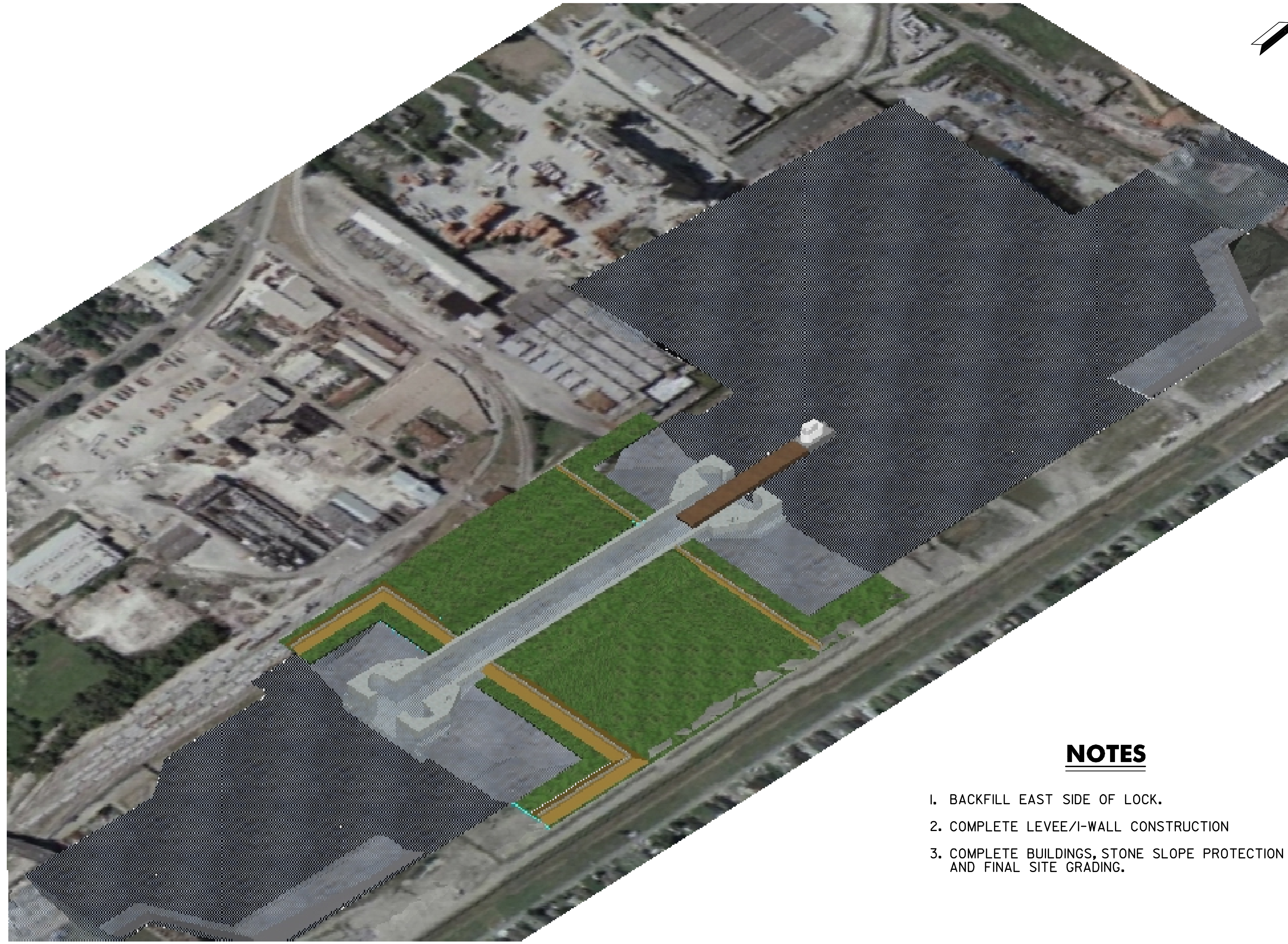
HUNTINGTON, WEST VIRGINIA

MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
LOCK CONSTRUCTED

Sheet reference number:
G1914
Sheet 0 of

EXHIBIT FIP–33



NOTES

1. BACKFILL EAST SIDE OF LOCK.
2. COMPLETE LEVEE/I-WALL CONSTRUCTION
3. COMPLETE BUILDINGS, STONE SLOPE PROTECTION AND FINAL SITE GRADING.

FIP – FINAL – LOCK OPERATIONAL

US Army Corps of Engineers
Huntington District

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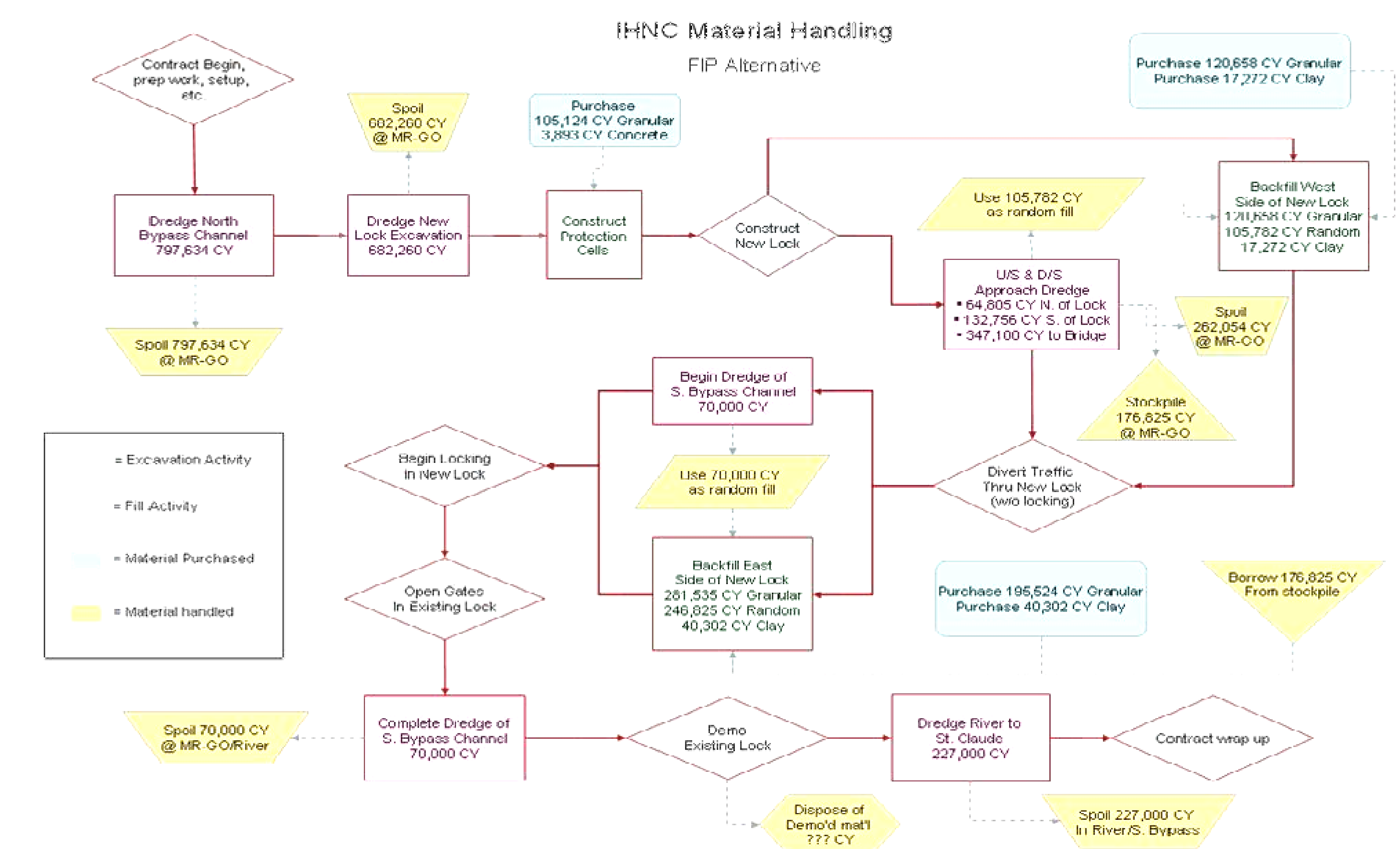
HUNTINGTON, WEST VIRGINIA

MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
FINAL

Sheet reference number:
GI915
Sheet 0 of

EXHIBIT FIP-34



FIP – MATERIAL HANDLING FLOWCHART

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Huntington District

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HUNTINGTON DISTRICT
HUNTINGTON, WEST VIRGINIA

MISSISSIPPI RIVER – GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

FLOAT IN PLACE
MATERIAL HANDLING

Sheet reference number:
GI601

EXHIBIT FIP-35

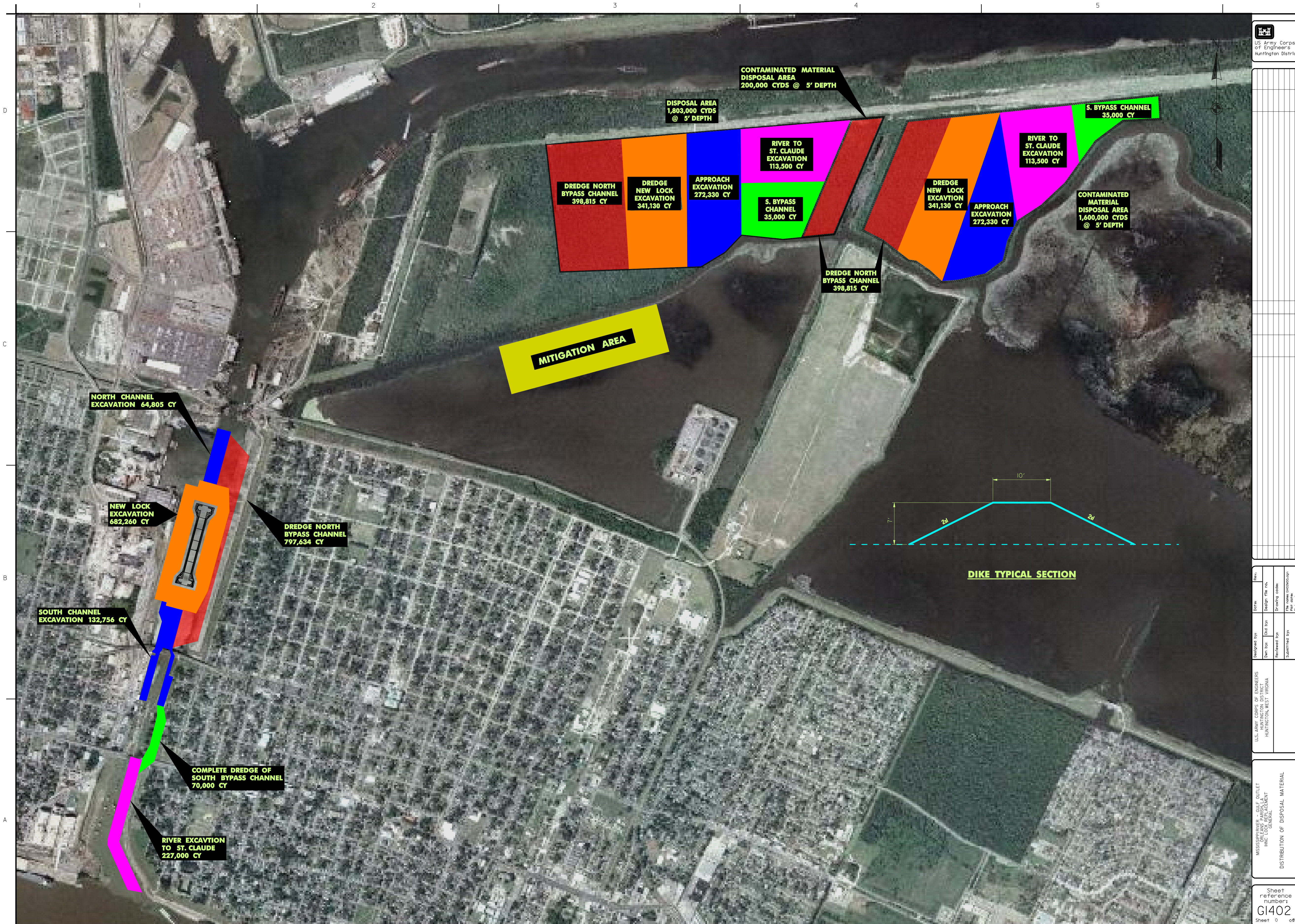
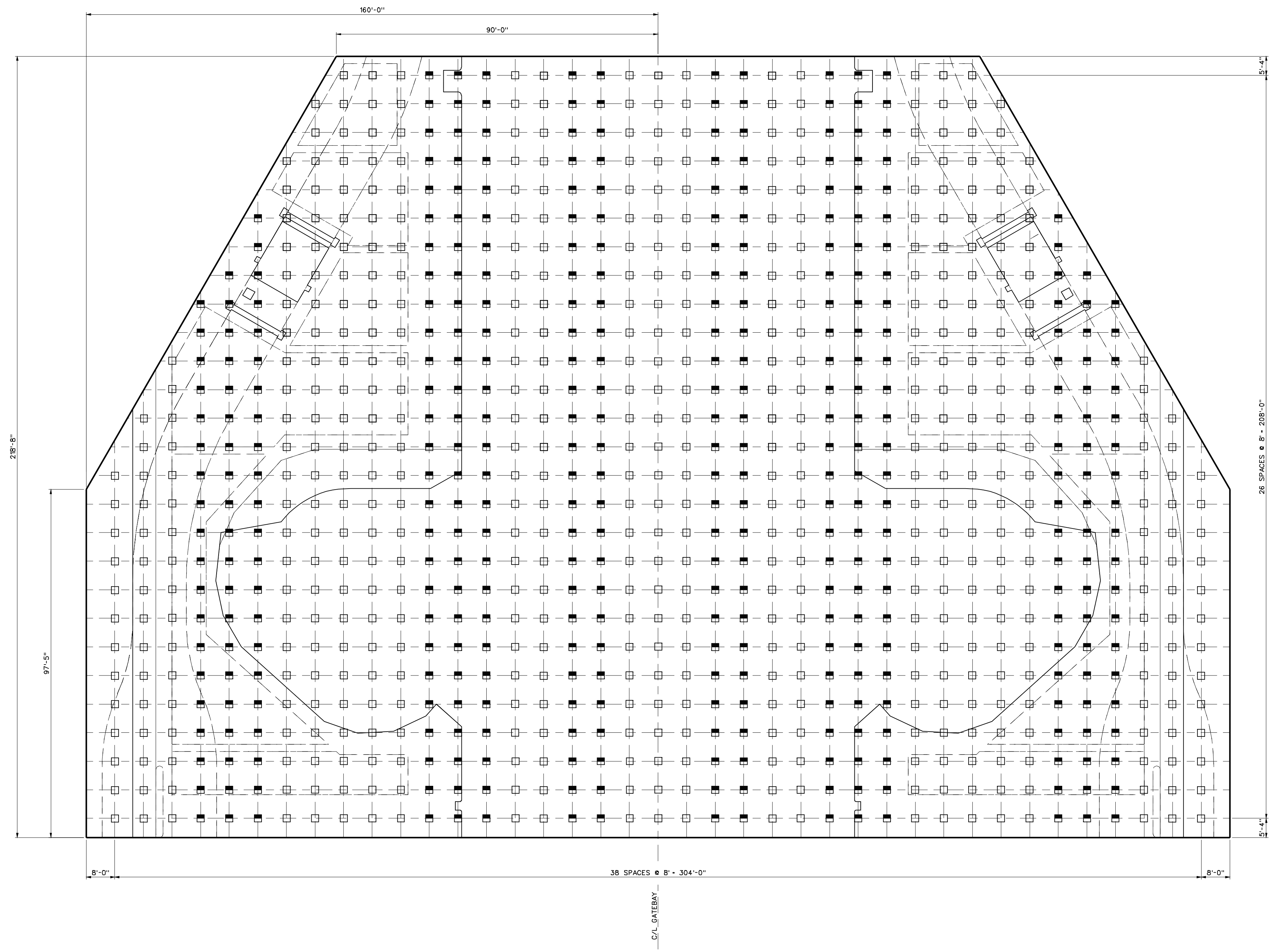


EXHIBIT FIP-36



RIVER SIDE
PILE LAYOUT

NOTES:

1. ALL PILES ARE 24" SQUARE PPC PILES WITH 18, 1/2" 0.7 WIRE, LOW-RELAXATION STRAND, GR 270.

GATEBAY STRUCTURE PILE SCHEDULE						
MONOLITH	TOTAL NUMBER OF PILES	PILE BATTER	PILE TIP ELEVATION	PAYMENT LENGTH WITH 12" EMBED	SERVICE LOAD	
					COMP. (K)	TENSION (K)
RIVER SIDE	541	VERTICAL	–185	131 FT	520	–200
RIVER SIDE	392	12V ON 1H	–185	131.5 FT	520	–200
LAKE SIDE	541	VERTICAL	–185	131 FT	520	–200
LAKE SIDE	392	12V ON 1H	–185	131.5 FT	520	–200

PILE LEGEND

- -12V ON 1H BATTER PILES
- -VERTICAL PILES
- 21 -PILE NUMBER

NOTES:

1. PILE C/L DIMENSIONS TAKEN AT EL. -54.0.
2. PILE LAYOUT SYMMETRICAL ABOUT C/L.
3. LAKESIDE GATEBAY SIMILAR.

16' 8' 8' 0 16'
SCALE: 3/32" = 1'-0"

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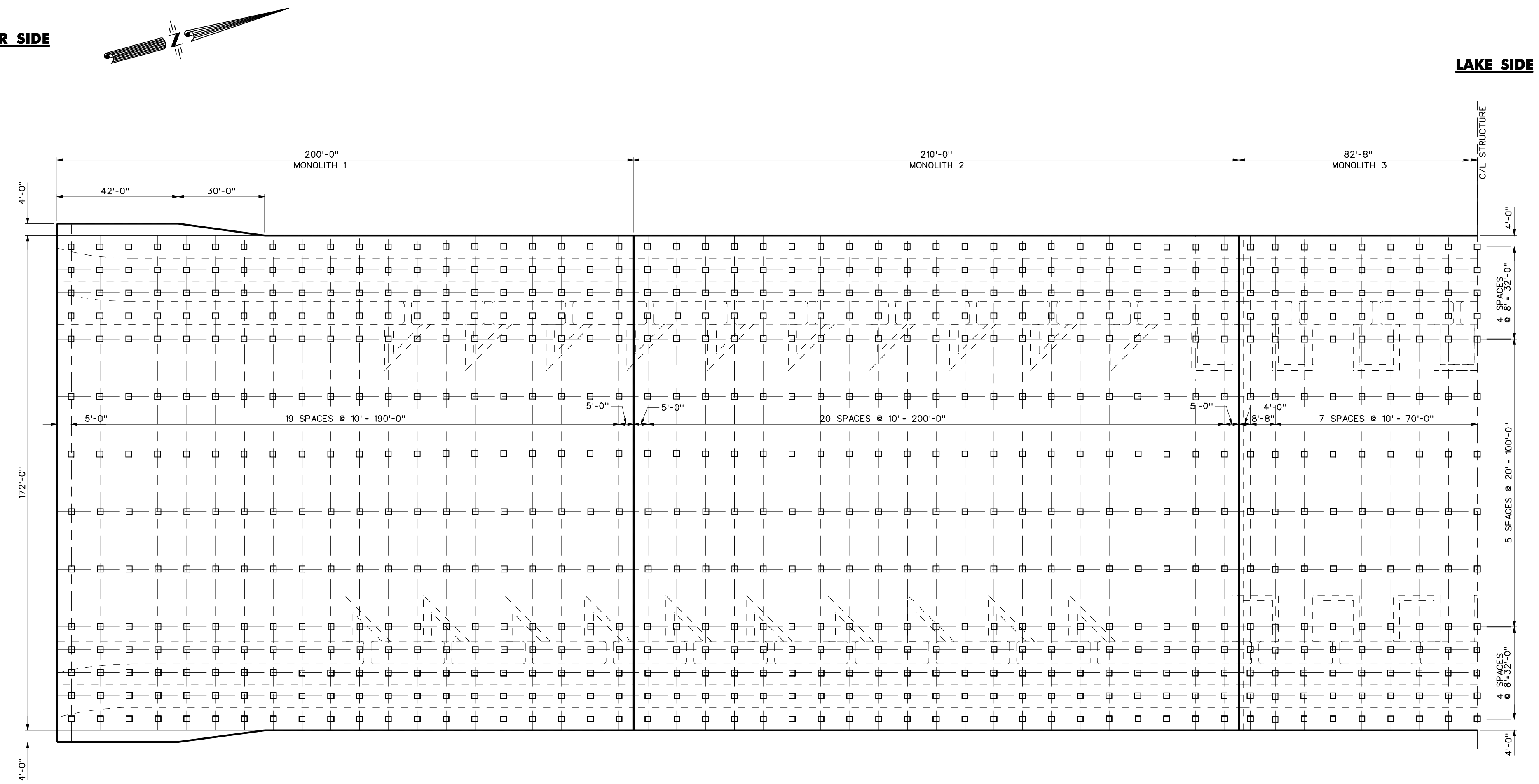
MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

CAST - IN PLACE
RIVERSISE GATEBAY MONOLITH
PILE LAYOUT

Sheet reference number:
S-301
Sheet 0 of 0

EXHIBIT CIP-1



PILE AND SHEETPILE LAYOUT

CHAMBER MASONRY PILE SCHEDULE

MONOLITH	TOTAL NUMBER OF PILES	PILE TIP ELEVATION	PAYMENT LENGTH WITH 12" EMBED	SERVICE LOAD COMP (K)	SERVICE LOAD TENSION (K)
1	280	-185	133 FT	520	200
2	294	-185	133 FT	520	200
3	238	-185	133 FT	520	200
4	294	-185	133 FT	520	200
5	280	-185	133 FT	520	200

NOTE:

1. PILE LAYOUT FOR CHAMBER IS SYMMETRIC ABOUT C/L.
2. PILE C/L DIMENSIONS TAKEN AT EL. -52.0.
3. ALL PILES ARE 24" SQUARE PPC PILES W/18, 1/2" DIA. 7 WIRE, LOW-RELAXATION STRAN. GR 270.

16' 8' 0 16'
SCALE: 1/16" = 1'-0"

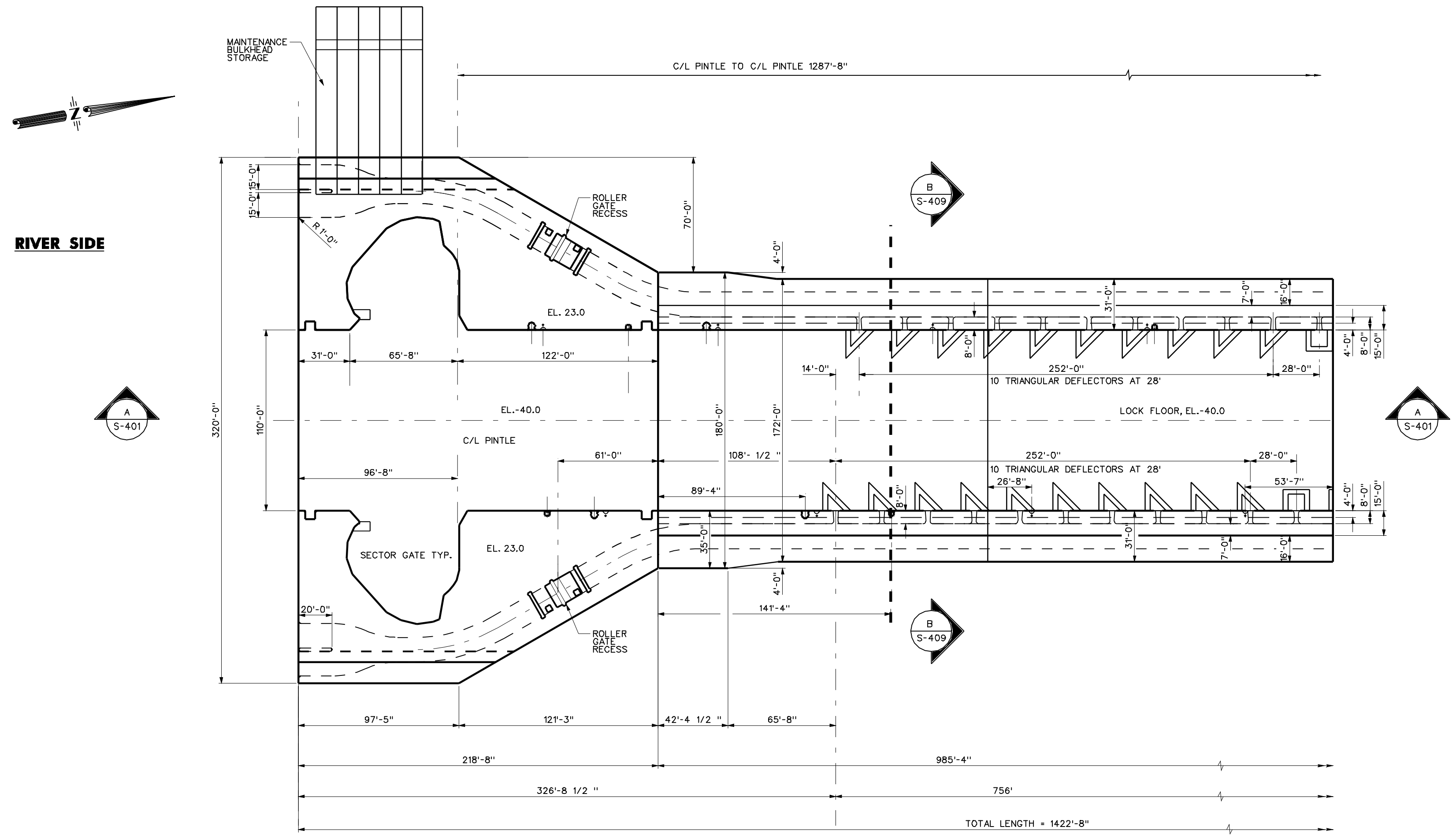
US Army Corps of Engineers
Huntington District

MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

Sheet reference number:

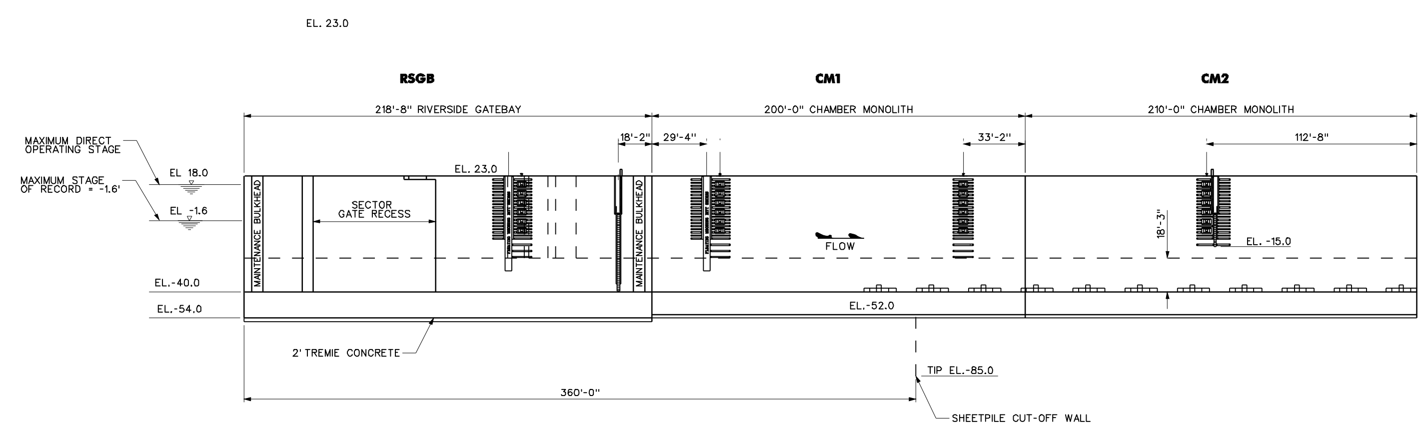
Sheet 0 of 0

EXHIBIT CIP-2



LOCK MONOLITHS PLAN

SCALE: 1" = 30'



LONGITUDINAL CHAMBER

SECTION A

30' 20' 10' 0 30'

SCALE: 1" = 30'-0"

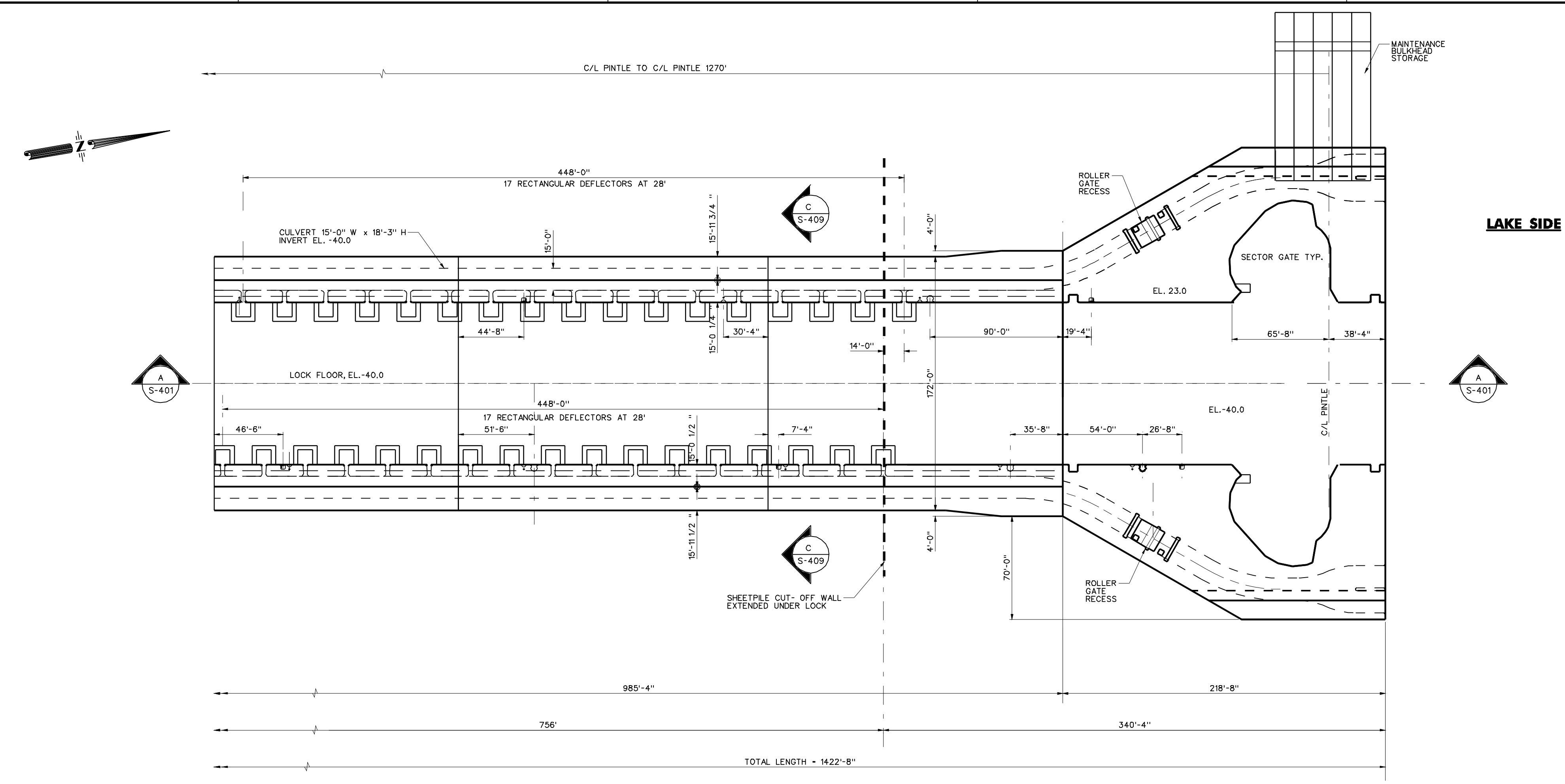
US Army Corps of Engineers
Huntington District

MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

Sheet reference number:

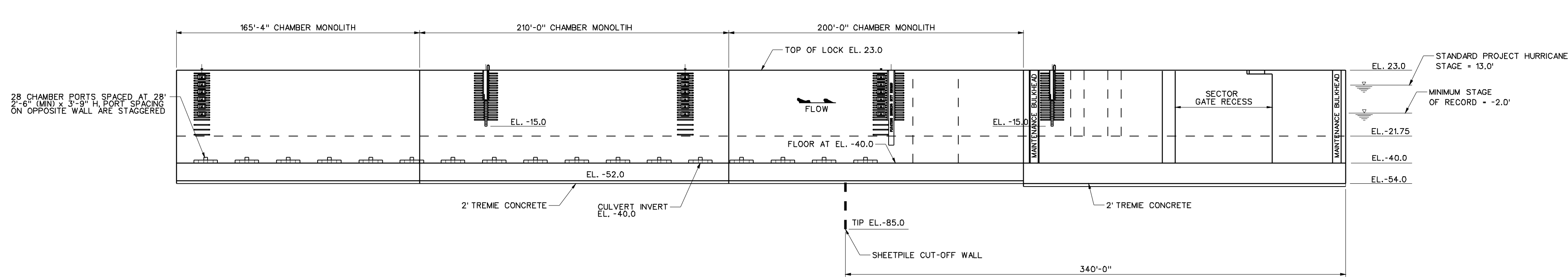
Sheet 0 of 0

EXHIBIT CIP-3



LOCK MONOLITHS PLAN

SCALE: 1" = 30'



LONGITUDINAL CHAMBER

SECTION A

SCALE: 1'' = 30'-0"

US Army Corps of Engineers
Huntington District

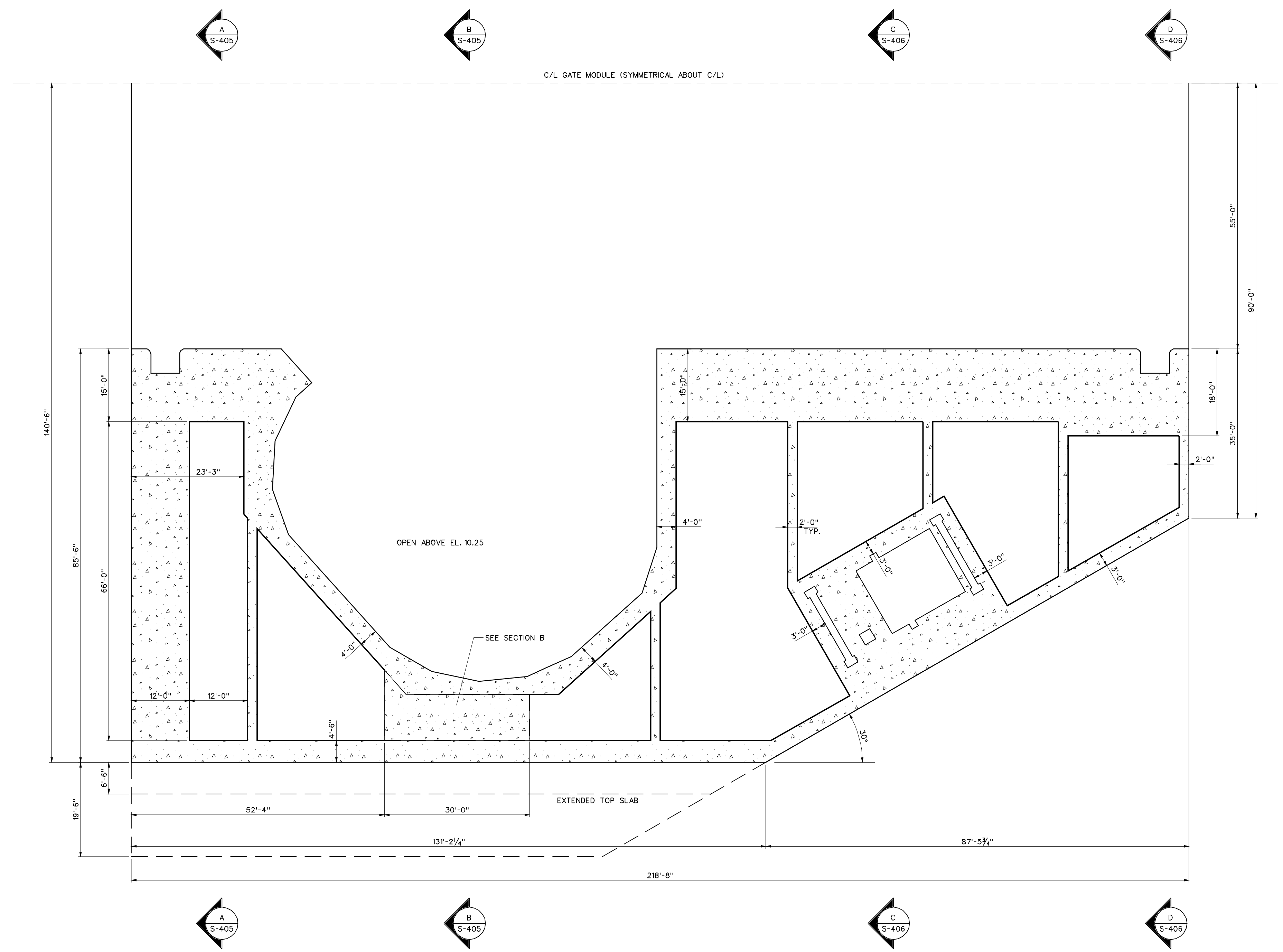
MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

Sheet reference number:

Sheet 0 of 0

EXHIBIT CIP-4

RIVER SIDE GATEBAY



MASONRY PLAN
ABOVE CULVERT

SCALE: 1/8" = 1'-0"

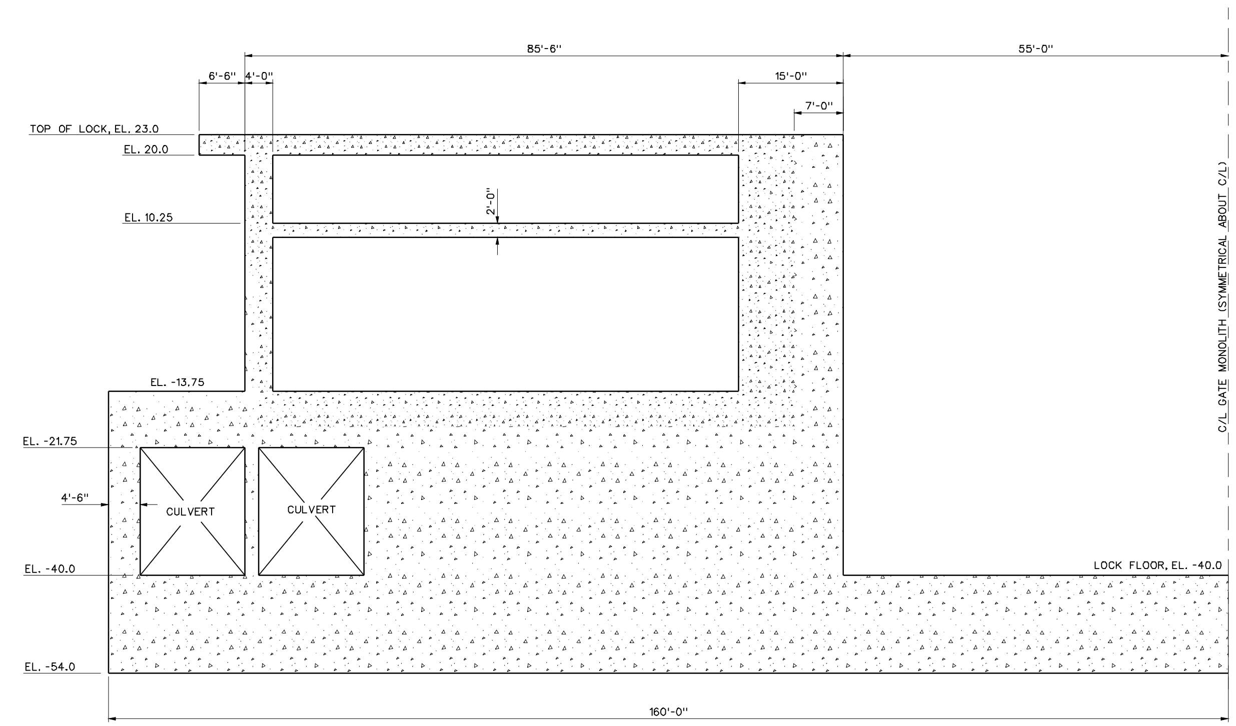
US Army Corps of Engineers
Huntington District

MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

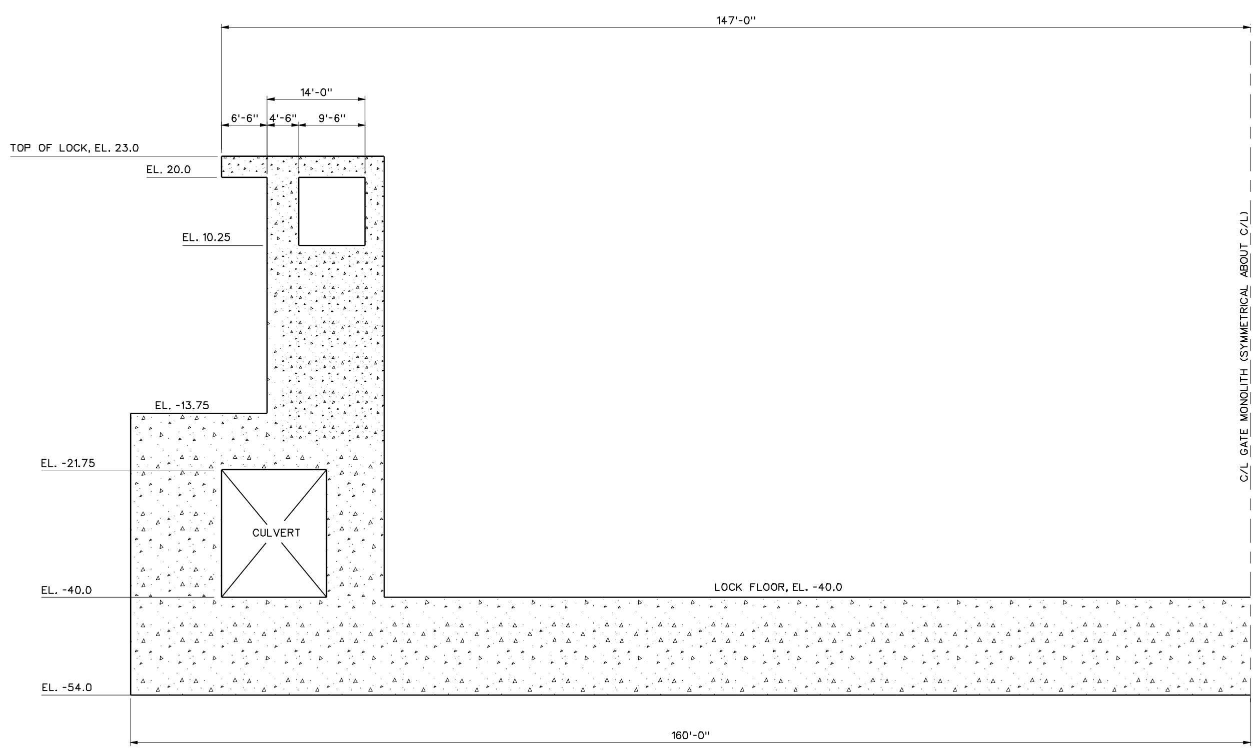
Sheet reference number:

Sheet 0 of 0

EXHIBIT CIP-5



SECTION A / S-404



SECTION B / S-404

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HUNTINGTON DISTRICT
HUNTINGTON, WEST VIRGINIA

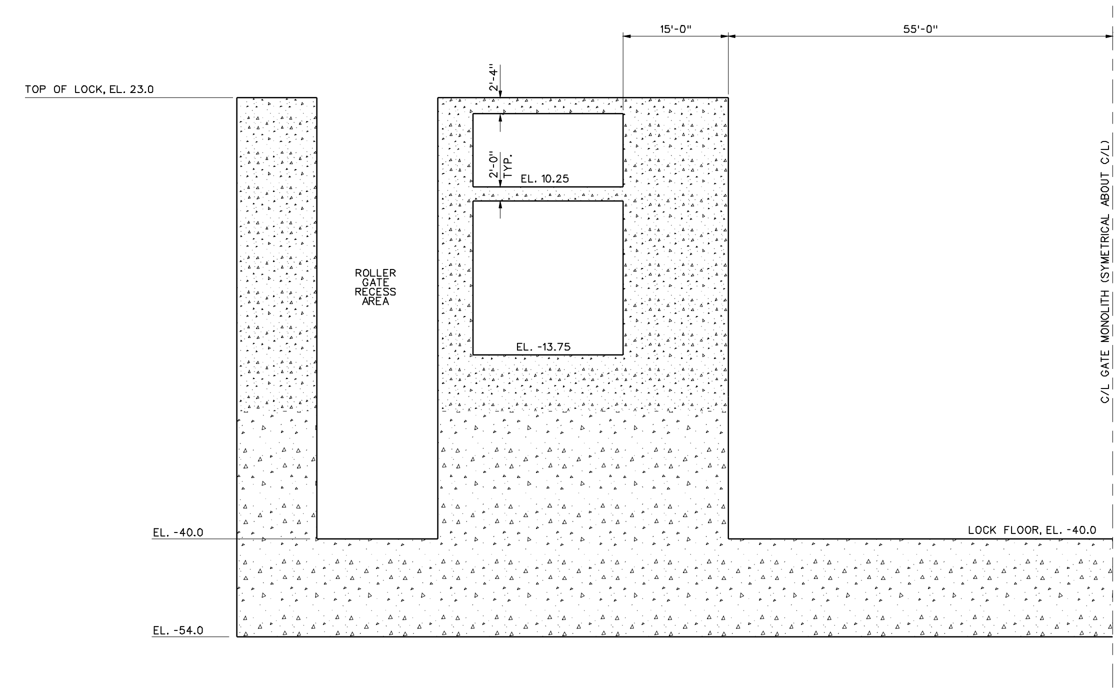
MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

CAST - IN PLACE
GATEBAY MASONRY SECTIONS

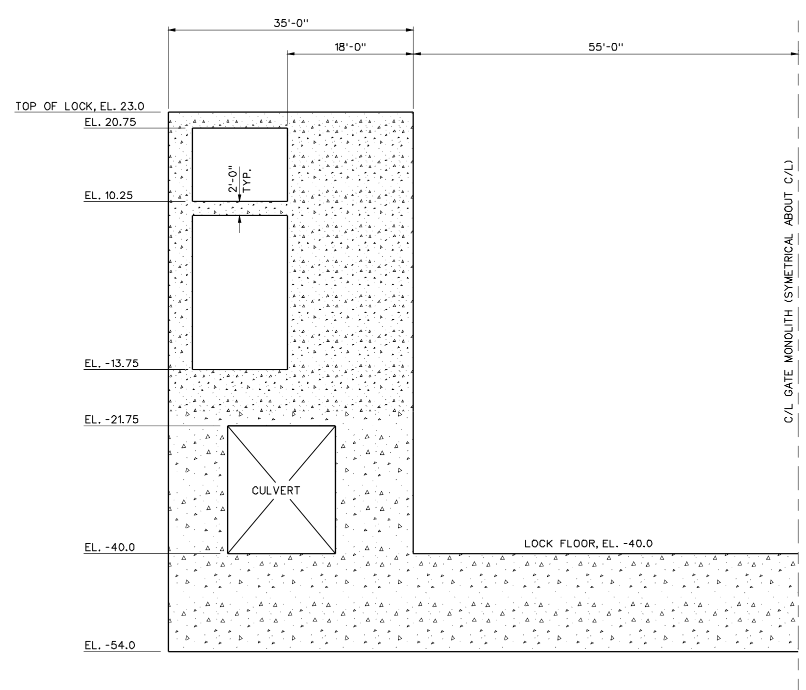
SCALE: 1/8" = 1'-0"

Sheet reference number:
S-405
Sheet 0 of 0

EXHIBIT CIP-6



SECTION C / S-404



SECTION D / S-404

SCALE: 1/8" = 1'-0"

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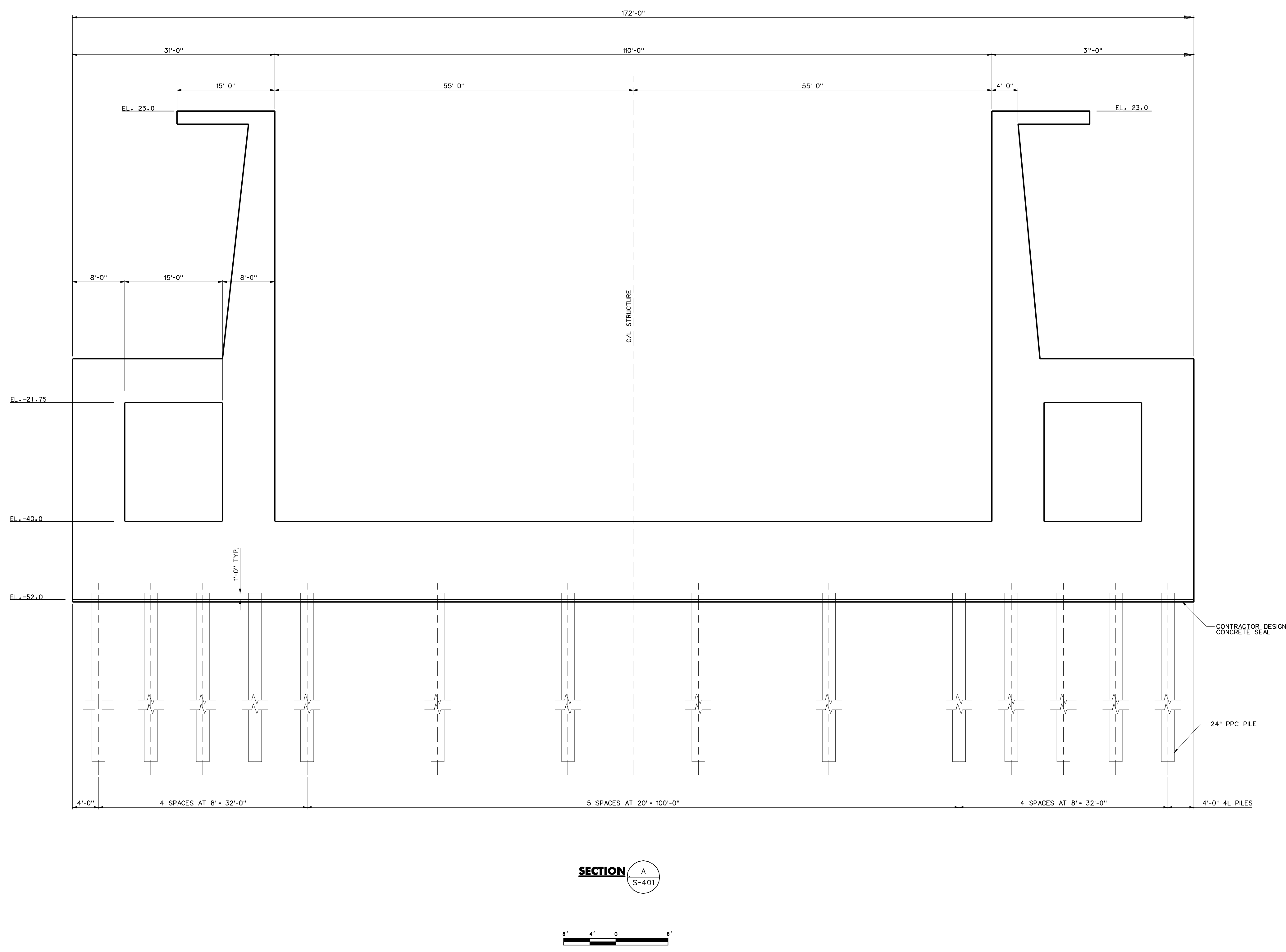
HUNTINGTON, WEST VIRGINIA

MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

CAST - IN PLACE
GATEBAY MASONRY SECTION

Sheet reference number:
S-406
Sheet 0 of 0

EXHIBIT CIP-7



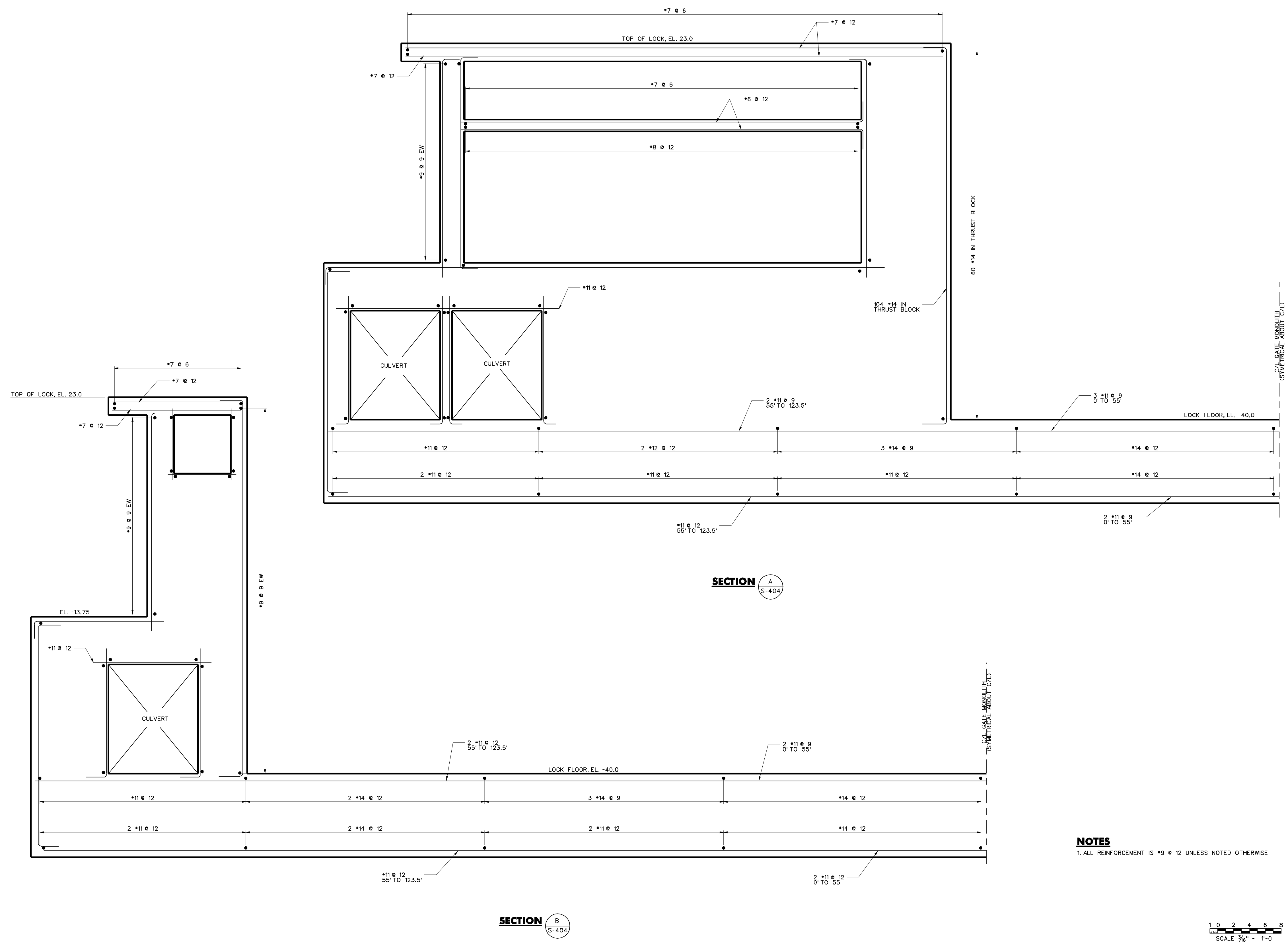
US Army Corps of Engineers
Huntington District

MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

Sheet reference number:

Sheet 0 of 0

EXHIBIT CIP-8



NOTES

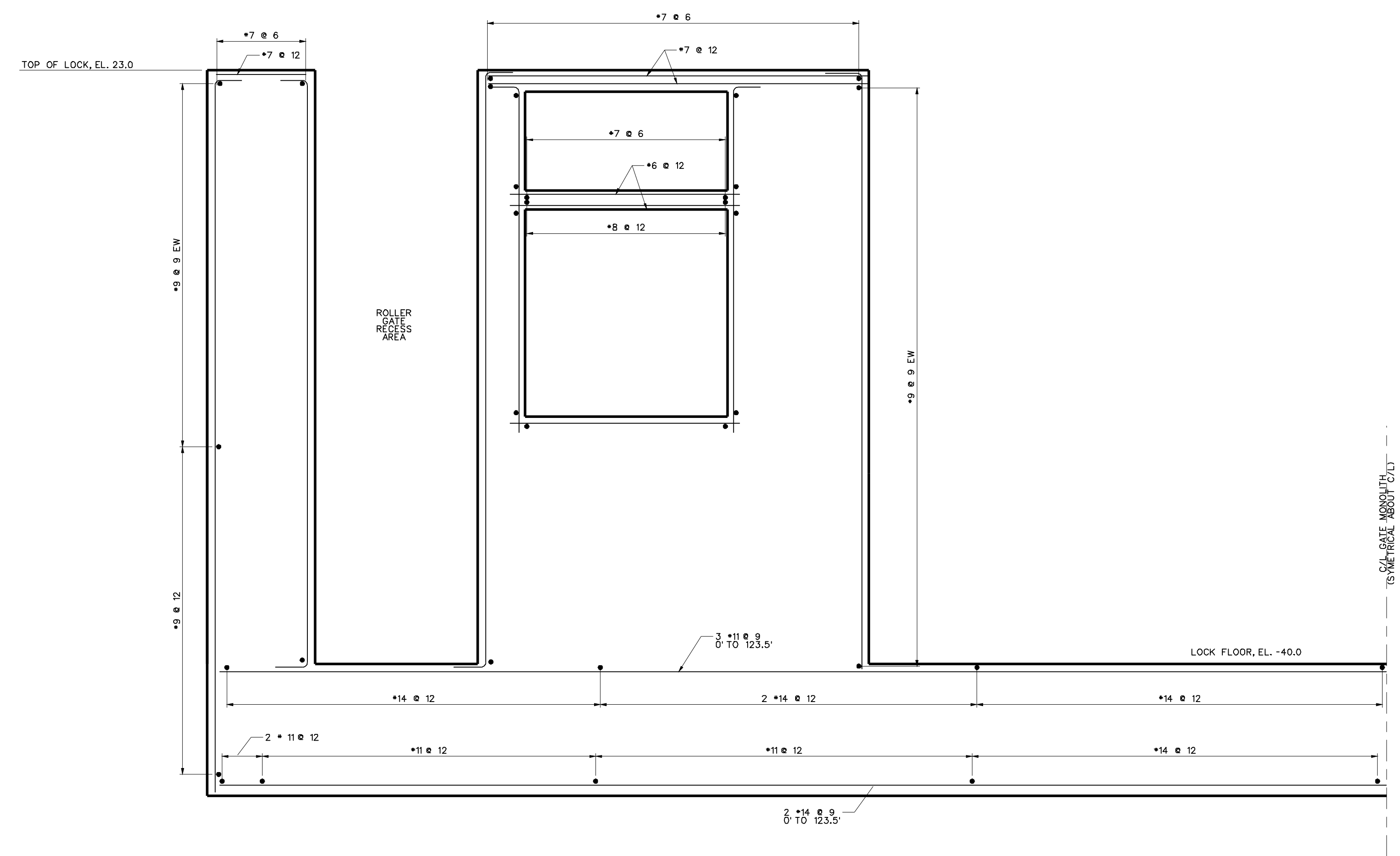
1. ALL REINFORCEMENT IS #9 @ 12 UNLESS NOTED OTHERWISE

SCALE 3/16" = 1'-0

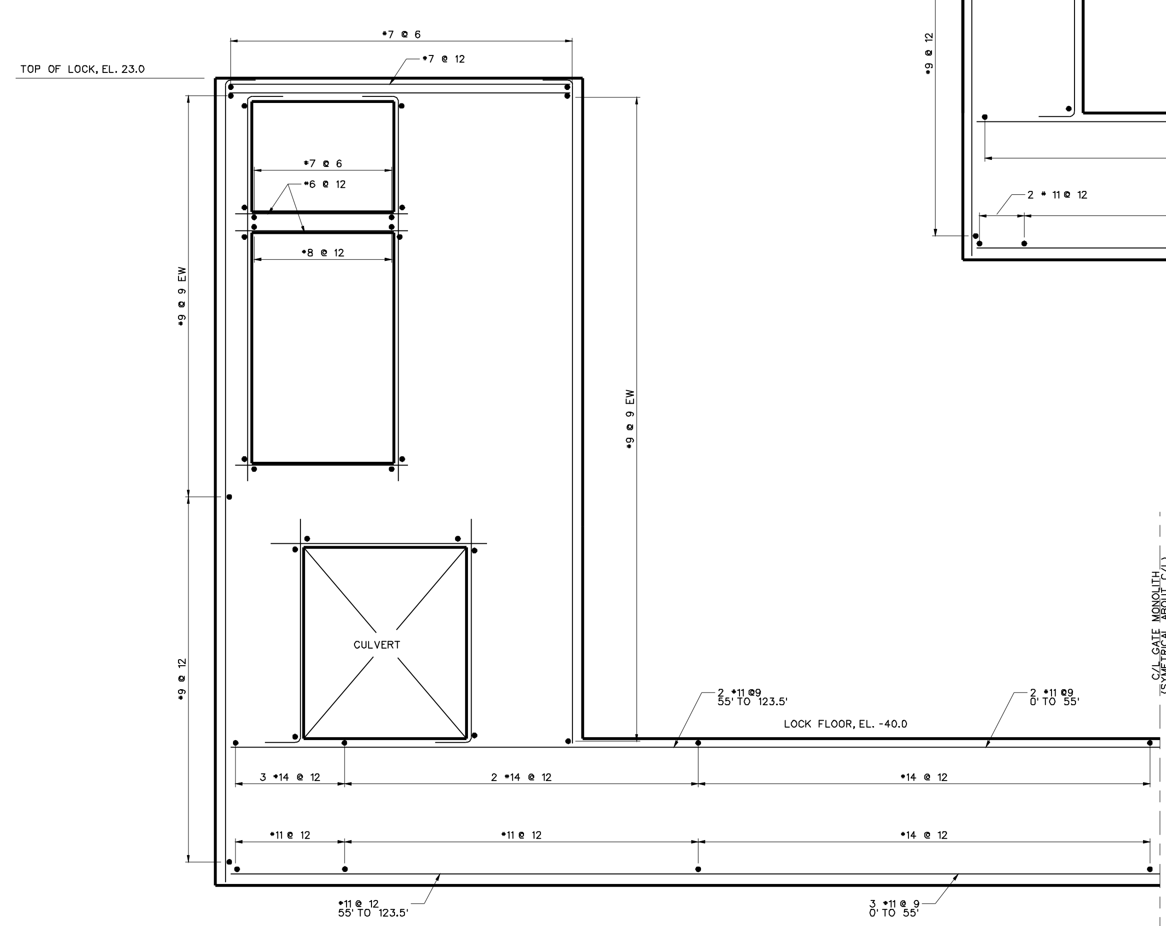
US Army Corps of Engineers Huntington District

MISSISSIPPI RIVER - GULF OUTLET ORLEANS PARISH, LA IHNC LOCK REPLACEMENT GENERAL

EXHIBIT CIP-9



SECTION C / S-404



SECTION D / S-404

NOTES

1. ALL REINFORCEMENT IS #9 @ 12 UNLESS NOTED OTHERWISE

SCALE 3/16" = 1'-0

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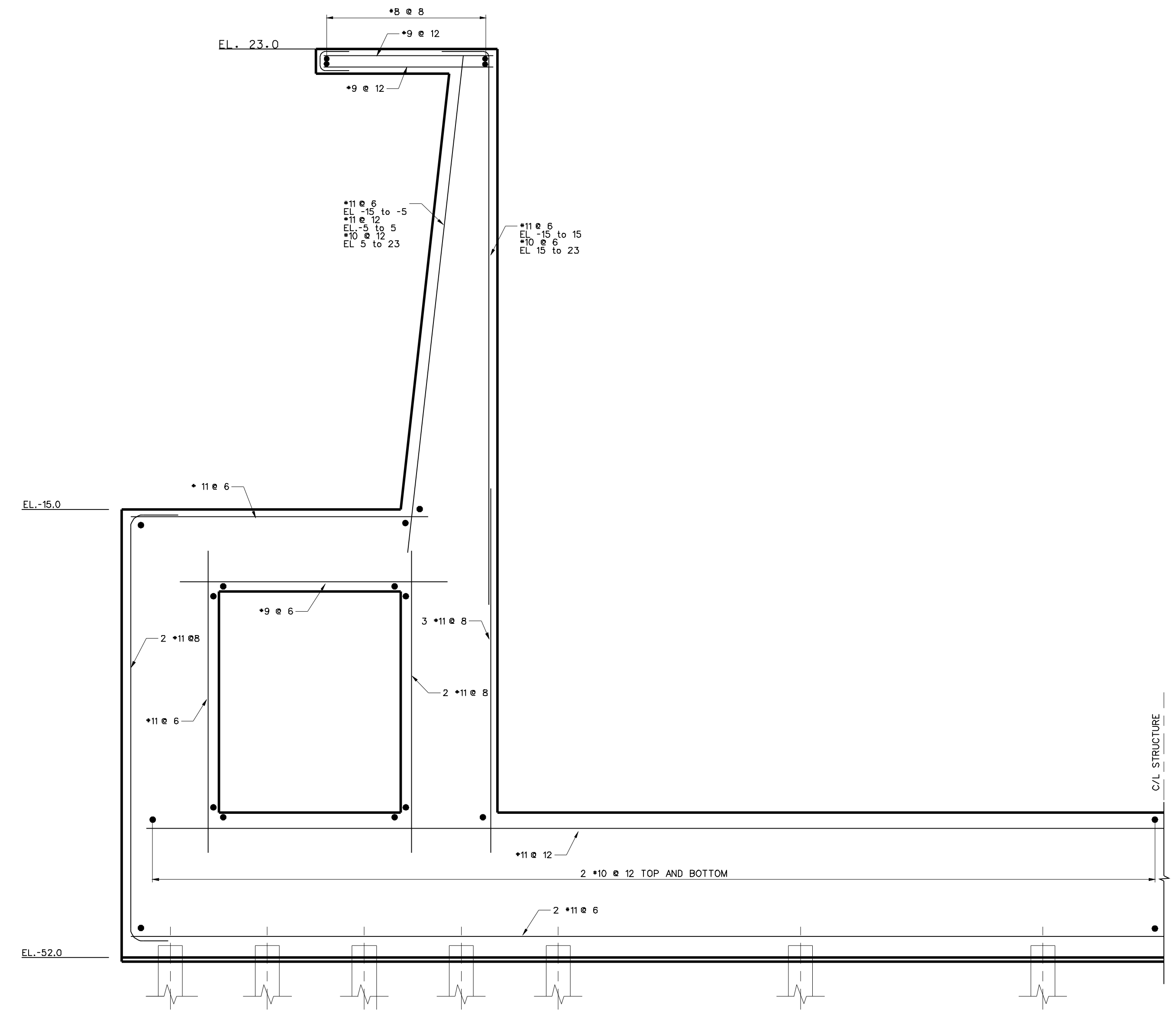
HUNTINGTON, WEST VIRGINIA

MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

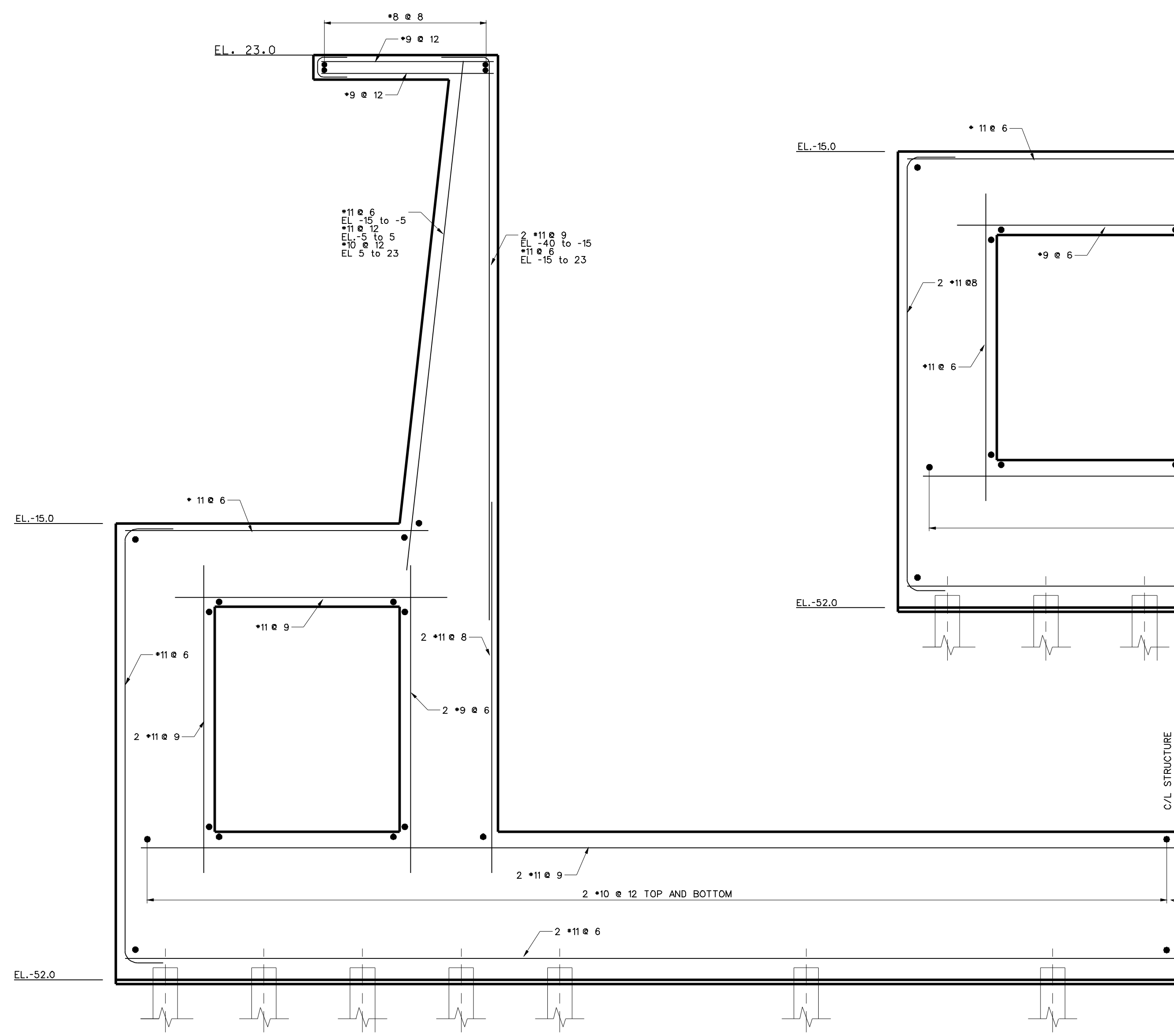
CAST - IN PLACE
GATEBAY REINFORCEMENT
SECTION 2 OF 2

Sheet reference number:
S-502
Sheet 0 of 0

EXHIBIT CIP-10



RIVER SIDE CHAMBER B S-401



LAKESIDE CHAMBER AND CENTER CHAMBER C S-402

NOTES

1. ALL LONGITUDAL STEEL IS #9 @ 12 UNLESS OTHERWISE NOTED.
2.

4' 2' 0 4'
SCALE: 1/4'' = 1'-0"

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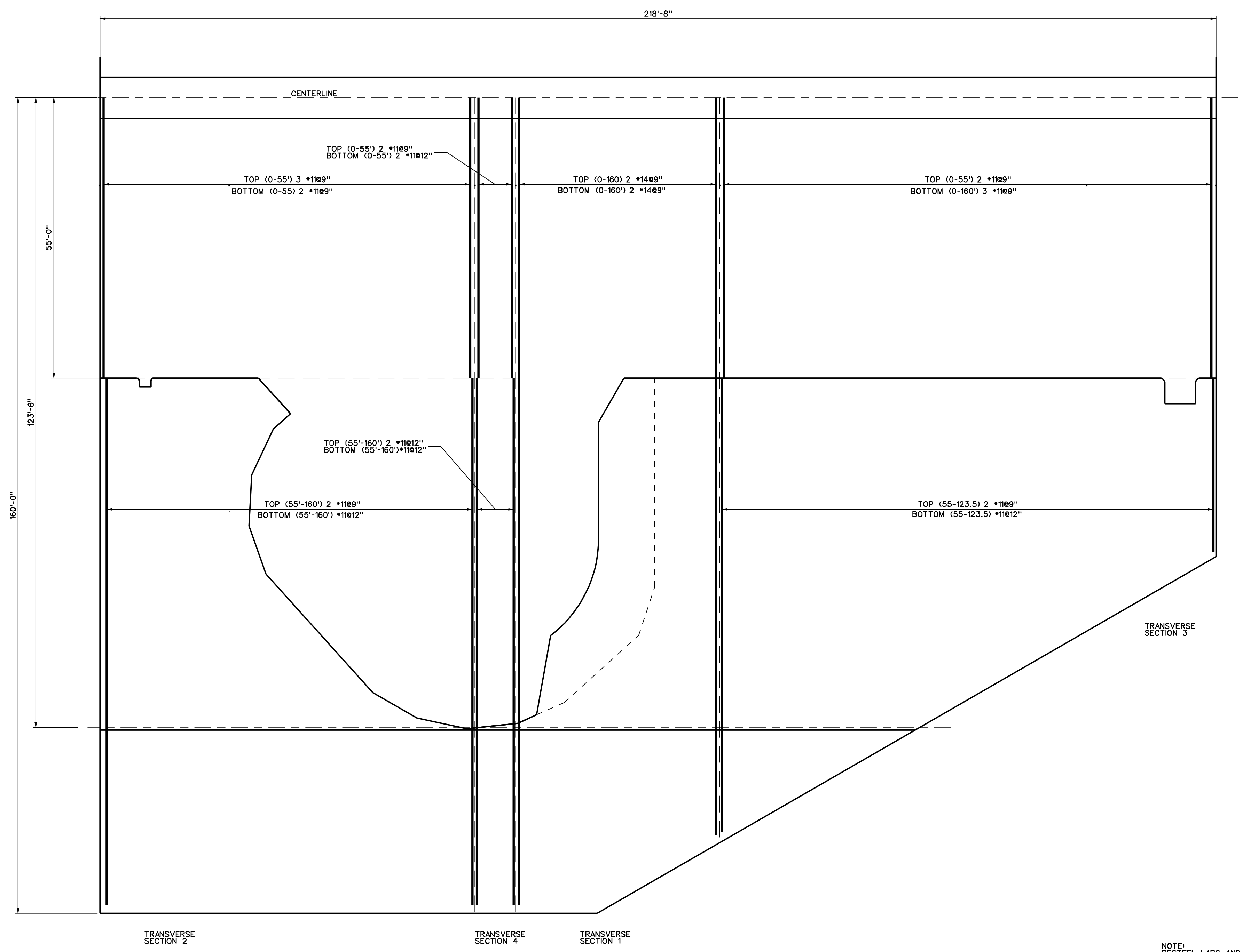
MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

Sheet reference number:

Sheet 0 of 0

EXHIBIT CIP-11



NOTE:
RESTEEL LAPS AND MECHANCIAL SPLICES ARE NOT SHOWN FOR CLARITY.

BASE SLAB RESTEEL

N.T.S.

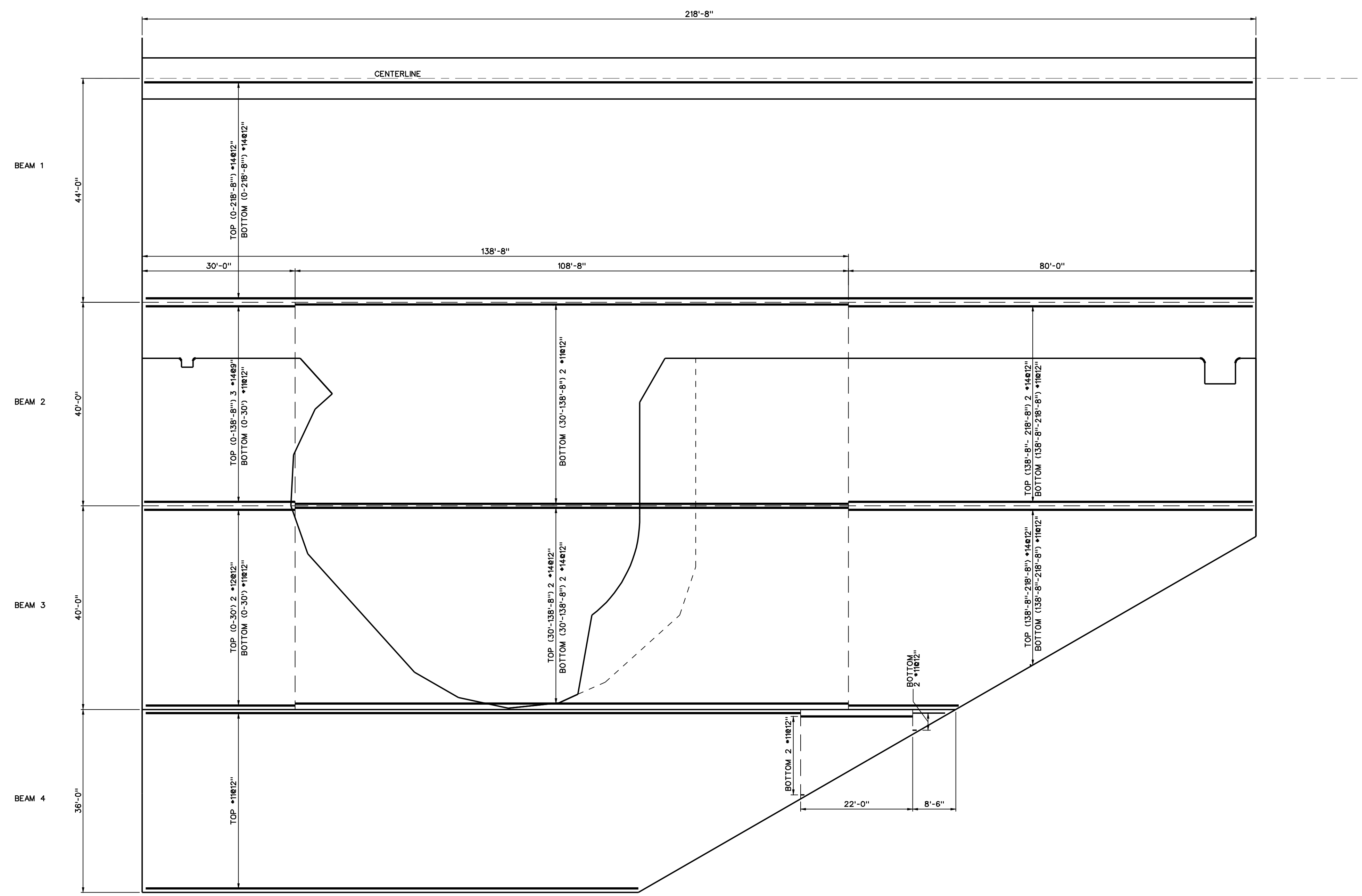
US Army Corps of Engineers
Huntington District

MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

GATEBAY MONOLITH
TRANSVERSE RESTEEL

Sheet reference number:
S504
Sheet 0 of 0

EXHIBIT CIP-12



BASE SLAB RESTEEL
N.T.S.

NOTE:
RESTEEL LAPS AND MECHANCIAL SPLICES ARE NOT SHOWN FOR CLARITY.

US Army Corps of Engineers
Huntington District

MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

GATEBAY MONOLITH
LONGITUDINAL RESTEEL

Sheet reference number:
S505
Sheet 0 of 0

EXHIBIT CIP-13

CIP – EXISTING CONDITIONS

NOTES

1. EXISTING CONDITIONS
2. NOTICE TO PROCEED

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MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

CAST IN PLACE

EXISTING CONDITIONS

Sheet reference number:
GI604
Sheet 0 of

EXHIBIT CIP-14

NOTES

1. BYPASS CHANNEL EXCAVATION.
2. TRAFFIC RUNNING NORMALLY.

CIP – BYPASS CHANNEL EXCAVATION

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MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

CAST IN PLACE
BYPASS
GRAVING EXCAVATION

Sheet reference number:
GI916
Sheet 0 of 8

NOTES

1. TRAFFIC DIVERTED TO BYPASS CHANNEL.
2. COFFERDAM CONSISTING OF 67.3' DIAMETER CELLS WITH ROCK BERM INSTALLED.
3. LOCK EXCAVATION COMPLETED IN THE WET.
4. DEWATERING SYSTEM INSTALLED/OPERATING.
5. CUTOFF WALLS INSTALLED.

CIP – COFFERDAM INSTALLATION

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MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

CAST IN PLACE
CELL CONSTRUCTION
LOCK EXCAVATION

Sheet reference number:
G1917
Sheet 0 of

EXHIBIT CIP-16

NOTES

1. 24" x 24" PRECAST, PRETENSIONED PILES (PPC) 120' LONG INSTALLED ON APPROXIMATELY 8' CENTERS.
2. RIVERSIDE GATE BAY MODULE SET DOWN AND WALL COMPLETED TO FINAL ELEVATION.

CIP – COMPLETED PILINGS

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MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

CAST IN PLACE PILING

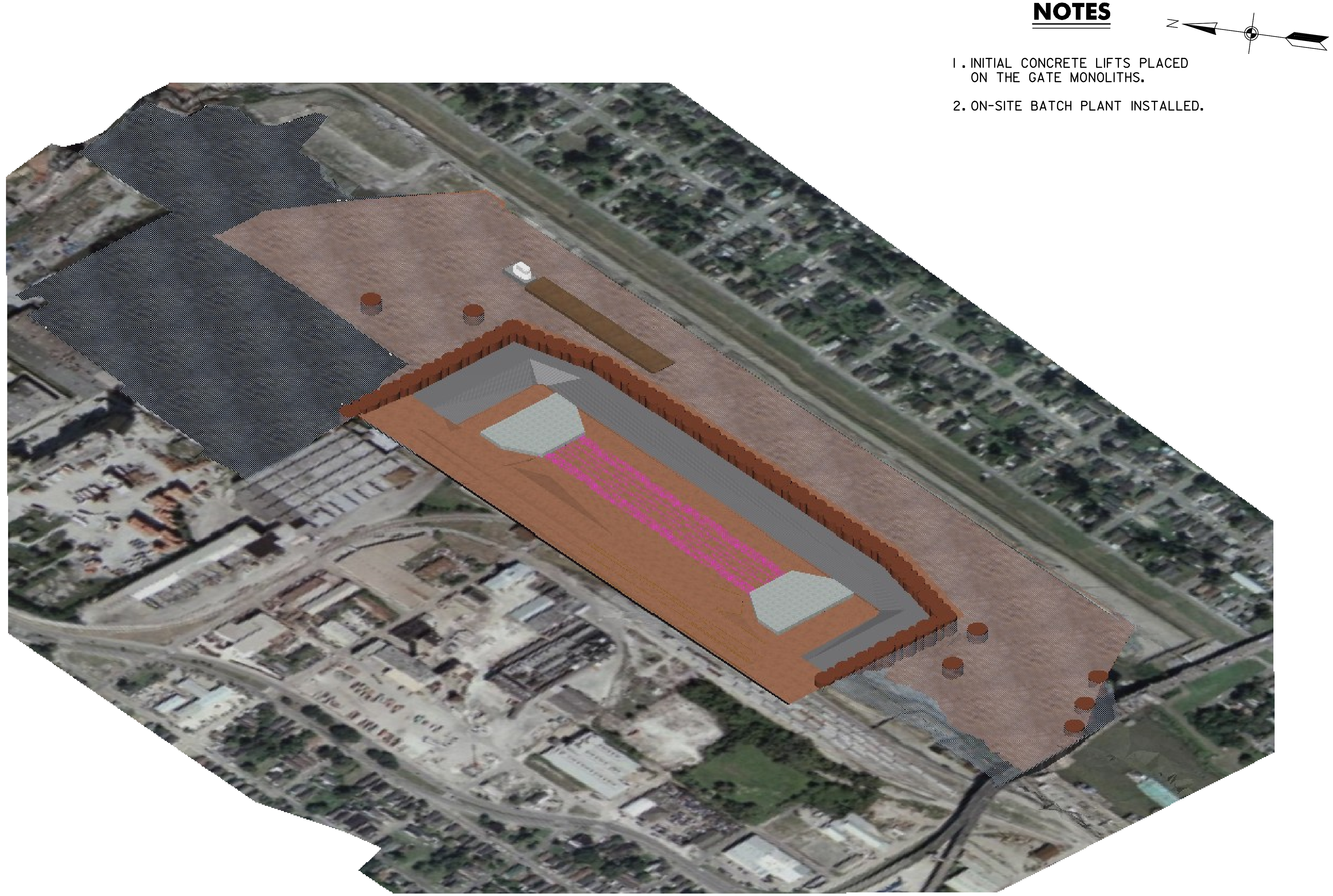
Sheet reference number:
G1918
Sheet 0 of

EXHIBIT CIP–17

NOTES

1. INITIAL CONCRETE LIFTS PLACED ON THE GATE MONOLITHS.

2. ON-SITE BATCH PLANT INSTALLED.



CIP – GATE FLOOR CONSTRUCTION

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HUNTINGTON DISTRICT
HUNTINGTON, WEST VIRGINIA

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MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

CAST IN PLACE
GATES
CHAMBER FLOOR

Sheet reference number:
G1919
Sheet 0 of

EXHIBIT CIP-18

NOTES

1. GATE MODULE CONCRETE COMPLETE.
2. CHAMBER MODULE INITIAL CONCRETE LIFTS STARTED..

CIP – GATE FINAL AND CHAMBER CONSTRUCTION

US Army Corps of Engineers
Huntington District

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Mark	Description	Date	Appr.

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HUNTINGTON, WEST VIRGINIA

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MISSISSIPPIRVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

CAST IN PLACE
GATE FINAL CONSTRUCTION
CHAMBER FLOOR CONSTRUCTION

Sheet reference number:
GI920
Sheet 0 of

EXHIBIT CIP-19

NOTES

1. LOCK CONCRETE COMPLETE.

CIP – LOCK CONCRETE COMPLETE

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Huntington District

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HUNTINGTON DISTRICT
HUNTINGTON, WEST VIRGINIA

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MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

CAST IN PLACE
FINAL LOCK

Sheet reference number:
GI921
Sheet 0 of

EXHIBIT CIP–20

NOTES

1. ENDS OF COFFERDAM REMOVED.
2. WEST SIDE BACKFILLED.
3. OPEN UP LOCK AS A PASS THRU.
4. POOL BEING HELD BY EXISTING LOCK.

CIP – PASS TRAFFIC THRU LOCK

US Army Corps of Engineers Huntington District

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HUNTINGTON DISTRICT
HUNTINGTON, WEST VIRGINIA

MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

CAST IN PLACE
WEST FILL
PULL CELLS AND ROCK

Sheet reference number:
GI922
Sheet 0 of

EXHIBIT CIP–21

NOTES

1. COMPLETE EAST SIDE BACKFILL.
2. COMPLETE LEVEE TIE-IN.
3. COMPLETE BUILDINGS, ETC.

CIP – LOCK OPERATIONAL

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HUNTINGTON, WEST VIRGINIA

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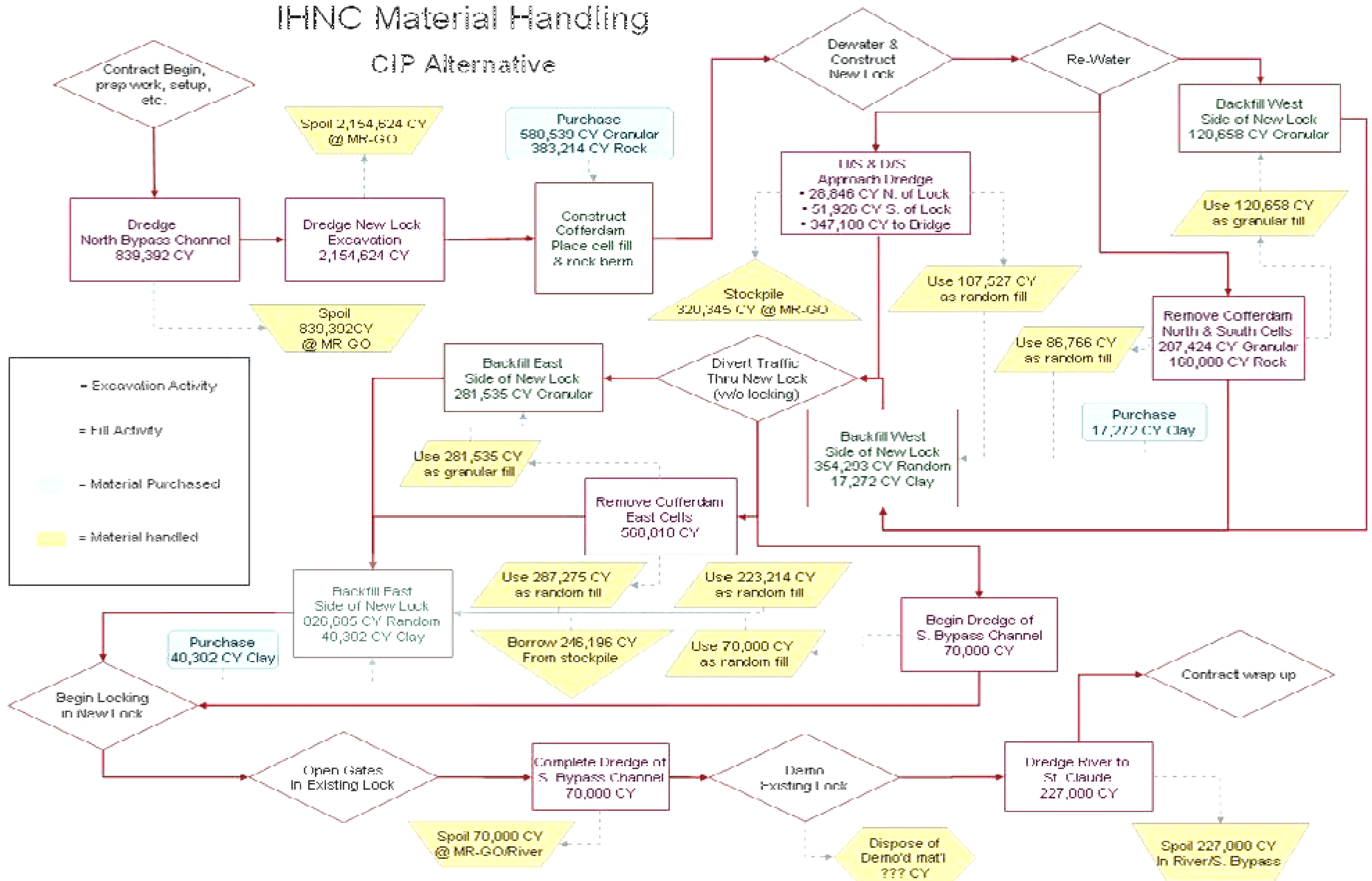
MISSISSIPPI RIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

CAST IN PLACE
FINAL CONSTRUCTION

Sheet reference number:
GI923
Sheet 0 of

EXHIBIT CIP-22



CIP – MATERIAL HANDLING FLOWCHART

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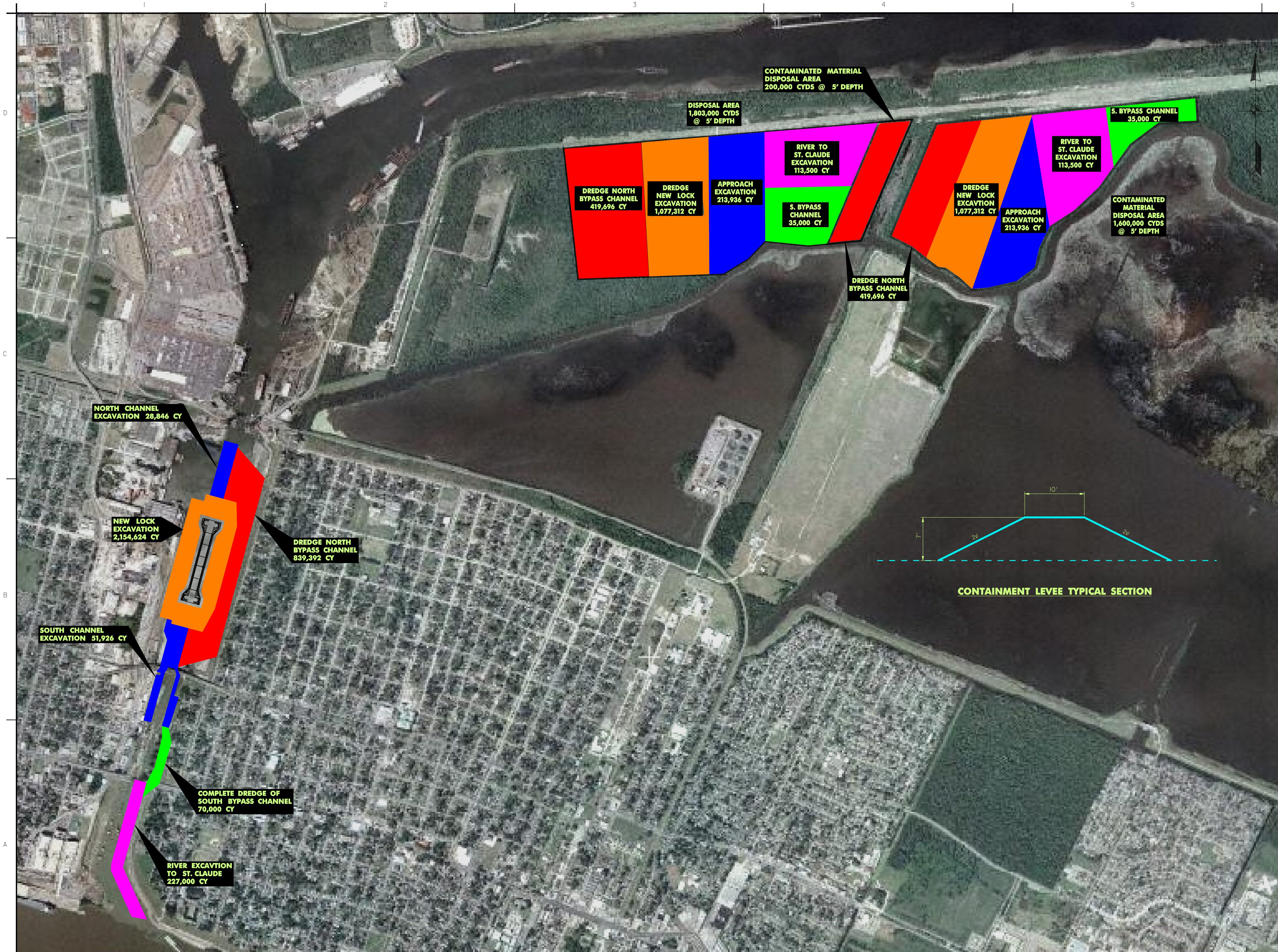
MISSISSIPPI RIVER - GULF OUTLET ORLEANS PARISH, LA IHNC LOCK REPLACEMENT GENERAL

CAST IN PLACE MATERIAL HANDLING

Sheet reference number: GI602

Sheet 0 of 0

EXHIBIT CIP-23



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Huntington District

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HUNTINGTON, WEST VIRGINIA

MISSISSIPPIRIVER - GULF OUTLET
ORLEANS PARISH, LA
IHNC LOCK REPLACEMENT
GENERAL

DISTRIBUTION OF DISPOSAL MATERIAL

Sheet reference number:
GI403
Sheet 0 of

EXHIBIT CIP-24

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