MISSISSIPPI RIVER - GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS

SECTION 2 - DESIGN

INTRODUCTION

B.2.1. This section covers the Lock North of Claiborne site including the general lock geometry and IHNC site data, structural design and design criteria, and a description of features of the replacement lock. A description of the Government provided graving site is included in Section 3. Details of the replacement lock "float-in" type construction, foundation and installation are presented. Also, the bypass channels required to maintain navigation during construction including canal limitations and closure periods are described. The existing lock will be demolished upon completion of the replacement. The controlled demolition plan for the existing lock removal is given. The permanent IHNC channel and dredge disposal details are presented. Levee and floodwall revisions are included, as well as construction sequencing. St. Claude and Claiborne Avenue bridge retrofitting or replacement information is given, including temporary detour route details.

GENERAL LOCK AND IHNC SITE DATA

General

B.2.2. 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.

Lock Geometry

B.2.3. A precast, post-tensioned, float-in concrete lock design is presented. The top of the replacement lock wall is elevation 22.4 feet NGVD. The lock chamber sizes studied are 900 and 1200 feet usable length, and 110 feet in width. The final lock chamber size is being optimized in studies by the Economics Branch of CELMN-PD, included in Volume __ of this report. The lock has miter gates as shown on plates B-38 and B-39, and lock culvert (tainter) valves as shown on plates B-40N, B-40T, B-41N and B-41T. The lock culvert is 14.5 feet square for the 900-foot lock and 15 feet wide by 18.25 feet high for the 1200-foot lock. Emergency and maintenance bulkheads are as shown on plates B-36 and B-37.

B.2.4. Two lock depths were studied at this site: a replacement barge lock with a sill elevation (-)22 feet NGVD, and a replacement ship lock with a sill elevation (-)36 feet NGVD. A shallower, GIWW-type barge lock option, with a sill elevation (-)15 feet NGVD, was also considered in lock size optimization engineering studies, but was not adopted. The lock chamber depth must be twice the draft of the design vessel to maintain an adequate water cushion above the lock floor. This prevents excessive hawser forces on the lock walls during lockage.

B.2.5. The filling and emptying system analyzed, i.e., an interior, ported culvert and manifold system with 14.5 feet square culverts, will be used for the 22 and 36-foot lock depths, for the 900-foot usable chamber, as shown on plate B-29 for the barge lock. For the 1200-foot long usable chamber, the culvert size is 15-by-18.25 feet, as shown on plate B-19 for the ship lock.

B.2.6. The lock structure is pile-founded. The pile design is described in this section, and the pile soil capacity is detailed in Section 3. The amount of calculated differential settlement eliminated any consideration for soil founding the concrete structure. The pile foundation shall be grouted to the concrete base with tremie concrete.

STRUCTURAL DESIGN CRITERIA

Scope

B.2.7. The analysis and design concepts for the structural components are presented in the following text. A general layout of the structure is presented on plates B-19 and B-20 for the 1200' Ship Lock and plates B-29 and B-30 for the Barge Lock.

References

B.2.8. 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) ER 1110-2-1806, Earthquake Design and Evaluation for Civil Works Projects (Jul 95).

(11) ETL 1110-2-256, Sliding Stability for Concrete Structures (Jun 81).

(12) ETL 1110-2-307, Flotation Stability Criteria for Concrete Hydraulic Structures (Aug 87).

(13) ETL 1110-2-338, Barge Impact Analysis (April 93).

(14) ETL 1110-2-355, Structural Analysis and Design of U-Frame Lock Monoliths (Dec 93).

(15) SL-80-4, Strength Report of Reinforced Concrete Hydraulic Structures, Report 3 - T-Wall Design (Jan 82)

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.

Design Criteria

B.2.9. Design criteria shall be according to the referenced EM's, ETL's, and technical publications.

B.2.10. The following material weights were used in the design:

Material Weights

Item	PCF

Water	62.5	
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	

B.2.11. Design Stresses. The following are discussions of the design stresses used in the structural analysis for the various components of the lock design.

a. Concrete. The Strength Design Method is used in the design of conventionally reinforced concrete in accordance with the requirements of the ACI 318R-89 Building Code. Values contained in the Code will be modified as required in EM 1110-2-2104. The appropriate Load Factors are stated in para. (8). The transportation and setting load cases shall be analyzed by both the Strength Design Method (USD) and Working Strength Design (WSD) as directed in Section 4.4 of ACI 357. The WSD allowables for the reinforcement are specified in ACI 357. The concrete compressive stress for the temporary loadings shall not exceed 0.45 f'c. Crack width is controlled by limiting the stress in the tension reinforcement. The crack widths are calculated in accordance with formulas in ACI para. 10.6.4 Commentary. In that the structure is precast, the dead load of the structure was included in the calculation of flexural stresses at service load. The base section is prestressed in the longitudinal direction; tension is not permitted in the concrete, therefore, concerns for cracks are eliminated. In the lock upper wall the flexural stresses are low, and cracking will not be a problem. Although the reinforcement was increased to reduce stress, an upper limit was maintained to assure a ductile failure. The maximum steel ratio does not exceed 0.25pbal. (for members with compression steel pmax = 0.25pbal. + p'x(fs/fy)). Service load deflections and the effects of deformation loads (i.e. temperature) shall be investigated during the FDM. The base concrete will have a minimum design strength (f'c) of 6,000 psi at 28 days, the upper walls will be constructed of concrete with a minimum 28-day strength of 4,000 psi, and the ballast concrete will have a design strength of 3,000 psi.

b. Reinforcement. The design strength of conventional reinforcement shall be based on the use of ASTM Grade 60 steel, having a yield strength of 60,000 psi. Development length shall be based on the full yield strength of 60,000 psi.

c. Prestressed Concrete. Criteria is based on the procedures presented in Section 9 of AASHTO Standard Specifications, the Post-Tensioning Manual and Chapter 18 of ACI 318. Prestressed members and composite members will be designed to satisfy both strength (Ultimate Strength Design-USD) and serviceability requirements (Working Strength Design-WSD). Allowable stresses are listed below. The Load Factors required in USD are specified in para. (8). The tendon system will be fully bonded. This Feasibility Design will only consider the threaded bar post-tensioning system. The concrete compressive strength f'c is 6,000 psi at 28 days; the concrete compressive strength at the time of transfer f'ci is 4,500 psi. The ultimate tensile strength (fpu) of the bars is 160 ksi.

ALLOWABLE STRESSES - PRESTRESSED CONCRETE

PRESTRESSING STEEL (Post-Tensioned Members)

Maximum Jacking	0.75 fpu
Maximum at Transfer	0.66 fpu
Effective (After Losses)	0.60 fpu**

** It is recognized that losses due to shrinkage, elastic shortening and creep of concrete as well as steel relaxation and friction should be considered. However, for this level of design an approximate value will be used, in the absence of a detailed analysis.

CONCRETE

AT TRANSFER (Before Losses)

Compression	0.55 f'ci
Tension	0

AT SERVICE LOADS (After Losses)

Compression	0.40 f'c
Tension	0

BEARING (Anchorage)

Compression	3,000 psi

d. Structural Steel. For this Feasibility Report all members shall be designed using Allowable Stress Design (ASD). Permissible stresses for structural steel members are specified in Chapter 4 of EM 1110-2-2105. The main steel components are classified by type, in para. g below. For Feasibility Report analysis, the miter gate was analyzed using ASD. The miter gate will be reanalyzed using LRFD as required in EM 1110-2-2105 during FDM design.

e. Welds. Allowable stresses for the design of welds shall be according to the latest AWS Welding Code as modified by EM 1110-2-2105.

f. Steel Pipe Piles. Allowable compressive stresses are given for both the lower and upper regions of the pile.

Axial Compression or Tension - Lower Region

Fa = 1/3 x Fy x 5/6

Combined Bending and Axial Compression - Upper Region:

$$\frac{fa}{Fa} + \frac{fbx}{Fb} + \frac{fby}{Fb} \leq 1.0$$

where:

fa = computed axial unit stress, kips per sq. in. (ksi)
Fa = 5/6 x 3/5 x Fy
fbx and fby = computed bending unit stress (ksi)
Fb = 5/6 x 2/3 x Fy (Compact Section)

g. Hydraulic Steel Component Classification. As described in paras. 4.4 and 4.8 of EM 1110-2-2105, the main components are classified by type as follows:

Steel Component	Type
1. Miter Gate	B

2. Culvert Tainter Valves A
3. Culvert Valve Bulkheads C
4. Emergency Bulkheads B
5. Temporary Module Closure Dam C

h. Ultimate Strength Design. Load factors for the ultimate strength design of the concrete structure are in accordance with EM 1110-2-2104 for hydraulic structure load conditions. For setting and transportation, the load factors were developed in accordance with ACI 357R-84.

Loading Condition	Load Factor
1. Normal Operating	Uh = (1.3) x 1.7(D + L)
2. Maximum Operating	Uh = (1.3) x 1.7(D + L)
3. Maintenance Dewatering (Unusual)	Uh = 1.7(D + L)
4. Hurricane (Unusual)	Uh = 1.7(D + L)
5. Setting	U = 1.2D + 1.6L + 1.3W U = .9(D + L) + 1.3W
6. Transportation	U = 1.2D + 1.6L + 1.3W U = .9(D + L) + 1.3W

where: D = dead load
L = live load
W = wind plus wave load
(where wind = 50 psf)

Loading Conditions

B.2.12. Six load cases will be investigated. Two uplift conditions will be applied to the Normal Operation and Maximum Operation, the conditions are described below. Only Uplift A is applied to the Maintenance Dewatering and Hurricane load cases.

B.2.13. Uplift Condition. Uplift Condition A assumes the headwater sheet pile is fully effective. The uplift pressure is constant and equal to the tailwater pressure head.

B.2.14. Uplift Condition B. Uplift Condition B assumes the headwater sheet pile cutoff is ineffective. Uplift pressure varies uniformly between the headwater pressure and the tailwater pressure.

B.2.15. Loadings. The following load cases were investigated:

Load Case 1	Normal Operation	
	Riverside Headwater	El. 10.0
	Lakeside Tailwater	El. 1.0
	Chamber	El. 10.0
Load Case 2A	Maximum Operation	
	Riverside Headwater	El. 17.6
	Lakeside Tailwater	El. -2.0
	Chamber	El. 17.6

Load Case 2B	Riverside Headwater	El. 17.6
	Lakeside Tailwater	El. -2.0
	Chamber	El. -2.0
Load Case 3	Maintenance Dewatering	
	Riverside Headwater	El. 10.0
	Lakeside Tailwater	El. 1.0
	Chamber	DRY
Load Case 4	Hurricane	
	Lakeside Headwater	El. 13.0
	Riverside Tailwater	El. 0.0
	Chamber	El. 0.0
Load Case 5	Setting (Construction)	
	Static Head	El. 3.0
	Chamber	DRY
	No Backfill	
	Voids Grouted	
	Flotation F.S. = 1.05)	
Load Case 6	Transportation Case	
	Still Water	El. 1.0
	Wave Height	6.0 Feet
	Wave Length	400 Feet
	(Voids Empty with Short U-Frame)	

B.2.16. Additional Load Factors. The following additional load factors were considered and applied as described below:

a. Sand backfill is assumed throughout the wall height, except for a negligible clay blanket, for all load cases except Load Case 5.

b. The effects of siltation are considered negligible.

c. Drag on the outer walls was added when the effects were conservative. Drag Force = at rest lateral earth pressure x 0.5 x (Tangent of the internal angle of friction)

FLOTATION

B.2.17. Flotation will be evaluated in accordance with ETL 1110-2-307. For the Maintenance Dewatering Case, the dead weight of the structure alone will exceed buoyancy by a Factor of Safety equal to 1.05. Tension connections to an adequate number of piles will be utilized to develop the minimum required Factor of Safety. Drag force is neglected. The minimum factor of safety for load conditions will be as follows:

Load Condition	Minimum Factor of Safety
Normal Operation	1.5
Maximum Operation	1.5

Maintenance Dewatering	1.05 without tension piles
	1.30 with tension piles
Hurricane	1.3
Setting	N/A

SLIDING

B.2.18. Since this foundation is grouted to the base, a check for sliding will be performed. The analysis will conservatively assume no embedment of the piles into the U-Frame base. The shear strength of the pile tension connections is also neglected. The sliding resistance considered is developed by the friction between the concrete base and the tremied 4 foot pile cap. The factor of safety will be calculated in accordance with ETL 1110-2-256. A friction factor of 0.60 is used. A minimum factor of safety equal to 2.0 is required for all load conditions. Deep seated failure is discussed in Section 3.

PILE FOUNDATION DESIGN

Design Criteria

B.2.19. The pile foundation shall be designed in accordance with EM 1110-2-2906. The minimum factors of safety, based on performing pile tests in the future, for the loading conditions are as follows:

Loading Condition	Category	Soil Case	Pile Capacity Factor of Safety Compression	Tension
Normal Operation	Usual	S	2.0	2.0
Maximum Operation	Usual	Q	2.0	2.0
Maintenance Dewatering	Unusual	Q	1.5	1.5
Hurricane	Unusual	Q	1.5	1.5
Setting (Constr.)	Unusual	Q	1.5	1.5

Pile Selection

B.2.20. The structure foundation for the replacement lock was designed as a pile-founded lock. The initial design was a soil-founded steel hull structure. Extensive work was done justifying the use of a soil foundation in spite of the fact that 10 inches of differential settlement was predicted. With the ballast concrete added, the weight of the steel hull and precast concrete structure is similar. However, the 10 inches of differential settlement was considered to be detrimental to the less ductile concrete lock. The possibility of excessive cracking created by the settlement, compounded by stress-induced cracking, eliminated the use of a soil foundation. The selected piles are 48-inch diameter by 5/8-inch wall steel pipe piles. The piles are 160 feet long for the ship lock and 140 feet long for the barge lock. Ultimate compression and tension pile capacity versus tip elevation were developed for 36-inch, 42-inch and 48-inch diameter steel pipe piles. The 48-inch was selected as the most economical pile.

General

B.2.21. The use of large diameter pipe piles was first recommended by EBASCO, Inc. in their work with the steel-hull lock. The use of larger pipe piles (versus steel H-Piles) with greater capacity was also considered given the difficulty of aligning piles that will be driven 50 feet underwater. The piles were analyzed for Load Cases 1 thru 4 as is specified in para. B.2.15. Pile tests will be performed, to determine capacity and noise levels, during the FDM phase. The pile reactions were developed assuming a rigid base. In the transverse direction, the reactions on the piles beneath the lock walls were increased to 125% of the uniform load. This assumption was based on the results of monitoring at the Port Allen Lock. The monitoring is reported in Technical Report S-68-7, by Sherman and Trahan, dated Sept 1968. This assumption was in close agreement with the pile reactions generated by the CASE program "CWFRAM". "CWFRAM" treats piles as elastic elements that develop resistance proportional to the displacement of the pile. The resistance is calculated from the axial and lateral stiffness of the soil. Additionally, the foundation was analyzed considering the effects of lateral and eccentric loadings using the CASE program "CPGA". Only two loadings, the Maximum Operation and Maintenance Dewatering Cases, which develop the most significant lateral forces, were considered. The combined bending and axial stress factor was low, and bending stress was a small portion of the total. The pile size and length needed for the maximum load were used throughout the foundation. A minimum spacing of three pile diameters required no reduction for group effects. The pile group effect was calculated in accordance with EM 1110-2-2906.

Pile Layout

B.2.22. The layout of the ship lock foundation is shown on plate B-24A; the barge lock foundation is shown on plate B-34A. The piles, all the same size, fall into three categories. The majority of piles act only as compression piles. In each module, three setting pads are required. Basically, the piles located beneath the pads are compression piles that are driven to a more stringent tolerance. In addition to the setting and compression piles, there are piles that offer a tension resistance by being anchored to the U-Frame. These piles provide tension capacity when the lock is dewatered; for all other load cases they act in compression. Design considerations for each of the three pile types are as follows:

COMPRESSION ONLY PILES

B.2.23. The required pile lengths were governed by the compression piles beneath the lock walls. The maximum combined stress factor is low, however, the 5/8 inch wall was needed to comply with the local buckling requirements as specified in AISC Table B5.1. The piles are connected to the structure base by the 4-foot thick tremie concrete cap.

SETTING PILES

B.2.24. The setting pads are required for module alignment. The tripod support was recommended by the Board of Experts as optimum for level placement and full contact with support pads. The selected longitudinal location developed the lowest moment in the U-Frame section with the fewest required number of piles. The ultimate pile capacity was reduced by the permitted factor of safety equal to 1.5. The setting piles withstand an applied load that is 5% greater than buoyant forces with the water stage at El. 3.0 NGVD. This load was increased by 2 feet of water head assuming that the water stage could drop to El 1.0 NGVD. The pile capacity was limited to that which could be provided by the length required for the governing compression only piles. Details for the setting piles are shown on plate B-24B.

TENSION PILES

B.2.25. For the Maintenance Dewatering load case, the modules are ballasted to achieve a factor of safety against flotation of 1.05 without tension pile anchors. With the tension piles, the factor of safety exceeds the required 1.3 minimum. The use of tension piles permits a reduction in the overall structure weight, thus permitting a shorter pile length. The tension connection also reduce the shear and moment of the base slab in the transverse direction. There are three tension piles per transverse row.

Future Study

B.2.26. In order to assure a more economical engineering solution, the following recommendations will be studied in the FDM:

a. The use of larger pipe piles will be investigated. Our design considered only pipe pile sizes commonly used in offshore oil structures. We will investigate 60-inch, 72-inch and 96-inch diameter pipe piles as was recommended by the Board of Experts. Noise levels and economics will dictate the pile size.

b. The use of multiple pile sizes will be considered. Using larger piles beneath, or in lieu of, the setting pads will be explored. Additionally, we will investigate varying sizes between the lock wall area versus the chamber area.

c. Pile designs will be optimized by using a flexible base analysis with bracketed soil coefficients. These results will be confirmed by finite element analysis.

Precast, Post-Tensioned Float-In Concrete Lock
Structural Design Methodology

General

B.2.27. The precast float-in concrete lock will be constructed in four pieces, as shown on plates B-19 for the ship lock and B-29 for the barge lock. All modules incorporate the U-Frame structure. The base section and lock walls achieve their structural integrity from a continuous series of I-Beams. In the base section the I-Beams span both directions with the bottom slab and floor slab acting as the flanges. The lock wall I-Beam utilizes the chamber wall and outer wall as the flanges. The ship lock floor is at El. -40; the barge floor/sill is at El. -22. The lock wall top elevation, El. 22.4, meets the river protection design grade. Beyond the northernmost gate the lock wall is reduced to the lakeside hurricane protection grade. Each module base acts independent of the others. The completed lock plan is shown on plates B-17 and B-18 for the ship lock and B-27 and B-28 for the barge lock.

Base Section

B.2.28. The base section will be constructed of 6,000 psi strength concrete at the graving site. The higher concrete strength was used to accommodate prestressing and to resist the high shear stress in the transverse web. The effective flange width complies with ACI 8.10. The flexural stresses could be resisted by conventional reinforcement, however, it was decided to post-tension the longitudinal direction to minimize cracking that could be created in the 434-foot direction during transport. This was the recommendation of the Board of Experts; Japanese Rules for prestressed barges require prestressing for lengths in excess of 300 feet. The maximum moment is created by the six-foot design wave. The wave-induced moment was calculated by methods recognized by the American Bureau of Shipping (ABS). The threaded bar post tensioning system is used, and the use of higher strength strand will be investigated in the FDM. Conventional reinforcement, capable of resisting the wave moment was also included as a "belt and suspenders" design. There are numerous other more economical methods that will be investigated (See Future Studies below). Strength in the longitudinal direction is only critical during the construction phase. In the transverse direction conventional reinforcement was used even though the shear force and bending moment are larger than the post-tensioned longitudinal direction. A significant amount of steel was needed to limit the tensile stresses in an effort to control cracking. Note that it is the recommendation of the Board of Experts' A-E members to use prestress in the transverse direction also. The moments created by lock operations, were the larger of flexible and rigid base conditions (see Pile Foundation Design). Shear and moment diagrams were created considering both rigid and flexible base pile design. The reinforcement required by the governing moment was conservatively used in both flanges and extended throughout the chamber span. Moments created by the heavier end walls (cantilevered about the center) during transport and setting were limited to that of normal operation by installing a strut. The strut will be a threaded post-tensioning bar. A second row of bars will be installed as the upper wall is added. The axial load was too small to use to increase the concrete strength for both concrete shear and flexural capacity. The shear force on the I-Beam web required increasing the web thickness and required a significant amount of stirrup reinforcement. The maximum stirrup size is limited to a No. 8 bar by ACI para. 7.1.3 Commentary. In addition to the I-Beam member design, each component was analyzed as a fixed plate. Obviously, the fixed plate analysis was only needed for loadings prior to ballasting. In addition to the shear and flexural requirements, the reinforcement meets or exceeds Corps criteria for temperature and shrinkage steel and ACI para. 10.6.7 requirements for web horizontal steel.

Lock Walls

B.2.29. The culvert walls are constructed of 6,000 psi concrete. The longitudinal direction is post-tensioned and acts monolithically with the base. The transverse section was considered as a rigid frame fixed at the floor with the culvert roof thick enough to provide rigid links in the corners. The culvert walls act as 10 foot deep I-Beams, 14 feet O.C., resisting the axial force due to wall weight and the overturning moment created by lateral forces. As with the base slab, the I-Beam components were checked as rectangular plates experiencing hydrostatic load prior to ballasting.

Module Joints

B.2.30. The joints act as expansion joints with no transfer of load. An adjustable J-Bulb waterstop assembly is mounted to the perimeter during construction. This waterstop is only needed to form an outer seal during the construction of the inner seal and seals around the culvert and gallery. The inner seal embeds a 3-Bulb waterstop in each module. The exposed joint surface is protected by a UHMW pad that permits movement.

Future Studies

B.2.31. The design presented is conservative. During FDM, alternate designs/systems will be explored to provide a more economical end product. Some revisions to be considered are:

a. Accept the use of bonded prestressed systems for the intended design life. The use of post-tensioning in both directions of the base has been recommended by both A-E members on the Board of Experts. Prestressing the base will permit a smaller section and will eliminate flexural cracking.

b. Investigate the corrosion potential for a bonded high strength strand. Strand provides greater capacity and is easier to install than the threaded bar.

c. Consider the use of partial prestress, utilizing the prestress to develop the needed strength for normal operation and resisting the difference for rare loadings with conventional reinforcement.

d. Increase the shear strength of the transverse web by:

- Post-tensioning the web in the vertical direction.

- Utilize the strength of the ballast concrete. Use corrugated forms to transfer shear or insert dowels in the web that extend into the ballast concrete forming a composite web.

- Use an anchor plate in lieu of the 90 degree hook to develop adequate bond for stirrups larger than No. 8 (limited by ACI 7.1.3).

e. As described in the Pile Foundation Design, para. B.2.22, use a flexible base analysis to take some conservatism out of the base design.

f. Use pneumatic seals in lieu of the conventional J-Bulb and 3-Bulb Waterstops.

DESCRIPTION OF LOCK NORTH OF CLAIBORNE AVENUE STUDIES

<u>General</u>

B.2.32. Two construction techniques were studied:

a. A precast, post-tensioned, float-in concrete lock seated on a prepared foundation (i.e., "float-in" construction), and

b. A conventional cast-in-place lock construction with a dewatered excavation (i.e., construction "in the dry").

Either technique will be a miter-gated lock, with direct head and reverse head gates. The "in the dry" plan was not adopted due to excessive costs and rights of way required for construction cofferdamming to maintain navigation in a two-way navigation bypass channel for the full construction period. Thus, the "float-in" plan was adopted, and is presented herein.

<u>PRECAST FLOAT-IN CONCRETE LOCK STRUCTURE</u>

B.2.33. The precast, concrete float-in lock structure requires a graving site. The Government furnished graving site is located on the MRGO approximately six miles from the lock site. The graving site will consist of an earthen excavation and closure berm. A pile-founded work platform is provided with a slab elevation of EL. -26 NGVD. The voided lock module base section will be fabricated and floated to the Galvez Street staging area. Onsite, the north bypass channel, lock foundation (excavation and pile-driving) and Galvez Street staging area will be constructed. The pile foundation will be installed concurrent with the construction of the first module. The staging area will be used to finish the upper lock walls. Completed lock modules will be installed by positioning, partially ballasting and then lowering the module onto the setting pads. After proper alignment is obtained on the pads, the base grouting and lock wall ballasting will be completed. The lock chamber will remain dry throughout construction. Lock module joints will be completed. The lock will be opened to navigation traffic as a pass-through only. Lock backfill will be placed and levee tie-ins completed. Lock guidewalls will be completed, and the lock will be opened to navigation. It is estimated that the 900' barge lock will take approximately 4.5 years to construct; the 1200' ship lock will require 5.5 years. A bar chart for the construction sequence will be included in the Project Management Plan (PMP) for this project. Details of the lock construction are given below, in the general order of the construction sequence.

CONSTRUCTION DESCRIPTION

<u>Graving Site Construction</u>

B.2.34. 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 23 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. -30 feet NGVD (or deeper), which is sufficient draft for transporting all modules. The graving site and details are shown on plates B-12, B-12B and B-12C. 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.

B.2.35. 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.

B.2.36. Two cofferdam cells and a sheet pile I-wall (see plate B-12B) must be driven before excavation. The cofferdam cells and I-wall will provide a removable closure dam. The upper 20 feet of the cells will be filled with lightweight stone. Natural ground can be used in the closure dam for the first closure; subsequent closures will be reconstructed of stone. The initial excavation will be done in the wet, using land-based equipment. Of the 225,000 cubic yards of material excavated, about 83,000 cubic yards will be used to construct the hurricane protection and tie-in levees. All levee work will be shaped, but not compacted. The levees will be overbuilt to account for settlement. The remaining excavated material will be hauled adjacent to the graving site (approx. 1/4 mile) and stockpiled.

B.2.37. 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.

B.2.38. Once dewatered, the excavation will be lowered another 16 inches. A 4-inch perforated concrete stabilization slab will be placed above a 12-inch compacted gravel base. Concrete piles, 12-inches by 12-inches and approximately 70 feet long, with a two-by-three foot reinforced concrete pile cap each, will be used to support the lock module base section during construction. The 4-inch perforated concrete stabilization slab and pile caps will be covered with a 6-mil polyethylene bond breaker.

B.2.39. After the project is completed, the graving site will be turned over to the Locals for their use as a dock facility. Therefore, the Contractor will only be required to remove the tie-in levee I-Wall and cell sheet piling and stone closure, and backfilling the site is not necessary.

North Bypass Channel Construction

B.2.40. 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, as shown on plates B-14N and 15N for the barge lock and plates B-14T and 15T for the ship lock. 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 1860 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.

B.2.41. 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 a period of 4-1/2 years of lock construction.

Lock Foundation Construction

B.2.42. Once the bypass channel is opened, lock foundation excavation will commence, as shown on plates B-16 for the ship lock and B-26 for the barge lock. 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. A barge mounted dragline will be used to obtain the required slopes. For estimating purposes, it was assumed that two feet was removed along the sloped portion of the lock excavation for approximately 2000 feet in length. Prior to pile driving, the Contractor will complete the 78-foot diameter protection cells, located at both ends of the excavation, as shown on plate B-12A.

B.2.43. The 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.

B.2.44. The lock piles, 48-inch diameter steel pipe piles, 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 setting 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 and precast concrete setting pads will be installed by divers; after leveling, the pads will be grouted into place as shown on plate B-24B for both the ship lock and the barge lock.

B.2.45. 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.

Galvez Street Staging Area Construction

B.2.46. Concurrent with north bypass channel dredging (or sooner), the existing Galvez St. Wharf must be demolished and removed.

B.2.47. The Contractor is anticipated to need a work platform to finish the upper lock walls. Note: This work platform is not mandatory; the Contractor may elect to use another method and finishing location. The Contractor may construct a work platform at any time after the Galvez St. Wharf is removed and before the first lock module (GB-1) is completed. The work platform, that is included in the cost estimate, will be a concrete deck, 60-feet by 400-feet in base area, and supported on steel H-Piles. The work platform will act as a deck, and as a mooring facility for each lock base module under construction. A sufficient number of battered piles must be installed to resist the wind and wave lateral load imposed by the moored module. Sheet piling will be driven and anchored to the landslide of the work platform. A void, 160-feet by 400-feet in base area, between the work platform and the west side floodwall, will be filled with earth, and will act as an access ramp and storage area. We assume that the Contractor will use a crawler-mounted crane with a minimum 25-ton capacity, and a 180-foot boom on the deck, and at least one barge mounted crane.

B.2.48. The western protection cell, at the north end, 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 Galvez Street staging area.

B.2.49. A clear area of 350-feet by 350-feet is available less than one-half mile away for the Contractor's batch plant and stockpile area. The batch plant must be capable of producing at least 125 cubic yards of concrete per hour. This production rate will require two portable batch plants.

Construct Lock Module Base Section

B.2.50. The south (riverside) gatebay module (GB-1) must be constructed first. The entire concrete base section will be constructed from 6,000 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.

B.2.51. The embedded metals required for cutoff piling and module joints and waterstops will be positioned during forming as shown on plates B-23 for the ship lock and B-33 for the barge lock. Each module will be post-tensioned in the longitudinal direction. The 1-3/8 inch diameter threaded post-tensioning bars will be coupled at 60-foot intervals. A larger duct is required at the coupler. The threaded bar will be fully bonded with a high strength grout. Transfer of stress to the bonded bars will not occur until the grout has obtained its full compressive strength. The base section culvert walls will be constructed with slip forms. The main steel reinforcement details are shown on plates B-21 and B-22 for the ship lock; B-31 and B-32 for the barge lock.

B.2.52. Concurrent with lock base module construction, the permanent emergency bulkheads and temporary end dam bulkheads will be fabricated. The end dam closure, needed for flotation at the graving site, will consist of one emergency bulkhead (10 feet high) at the lowest position and one temporary bulkhead 10 feet high (for construction only). The end dams will be installed just prior to flooding. The culvert openings will be sealed with steel bulkheads. Two modules will be floating concurrently, thus, four lower temporary bulkheads are required at this phase of construction. Three upper bulkheads per end dam (9 feet high each) are added as the upper wall is constructed at the Galvez Street staging area. (a total of five bulkheads per end dam). The maximum bulkhead weight is 130 tons.

B.2.53. Tension struts are needed to counteract the moment induced by the heavy lock walls during transport, as shown on plates B-21 for the ship lock and B-31 for the barge lock. The struts will span the transverse direction at the culvert top elevation. Post-tensioning bars 1-3/8 inches in diameter will be used as the struts, which will be located at each bulkhead wall (14 feet on centers). The struts will be encased by standard weight pipe sleeves. These struts will remain in place until the module is set and grouted to the pile foundation.

<u>Transport Lock Base Modules</u>

B.2.54. Prior to transport of each module, the graving site will be control-flooded and the I-Wall and berm closure 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.

B.2.55. It is anticipated that six tug boats are needed to pull the module along the six mile route to the Galvez Street staging area. The MRGO will be closed to marine traffic during the one-day haul. To complete transport, each module will be moored to the work platform.

<u>Construct Upper Wall Section</u>

B.2.56. The upper walls will be constructed of conventionally reinforced concrete, and no vertical or transverse post-tensioning is required. The concrete in the upper walls will have a minimum compressive strength of 4,000 psi at 28 days.

B.2.57. Wall armor and joint seals will be installed. Additional end dam bulkheads will be installed as wall construction progresses. A second row of temporary tension struts is needed to counteract the weight of the upper wall. The upper row of tension struts are the same as ones placed at the graving site. All temporary tension struts will be removed after the module is properly positioned on the setting pads.

B.2.58. Miter and culvert gates and machinery will be installed at the work platform. A barge-mounted crane with a 200-ton capacity is needed to install the miter gates. The base slab will be ballasted with concrete while moored at the Galvez Street staging area work platform. Ballast concrete will have a minimum compressive strength of 3,000 psi at 28 days.

Lock Module Installation

B.2.59. Positioning guides shall be installed while the upper wall is nearing completion. The completed concrete module is then transported to the foundation site.

B.2.60. Mooring lines will be attached to the structure. The structure position is horizontally adjusted by tensioning the guide lines. Tolerances of less than 3 inches have been achieved on similar structures.

B.2.61. The chamber will remain dry until the entire lock is completed. The base and outer culvert wall voids will be filled with ballast concrete prior to setting. With this partial amount of ballasting, the module is still buoyant. The final lowering will be accomplished by ballasting the lock wall mid-height compartments with water. Should the first attempt be out of tolerance, the module can readily be refloated and lowered again. A ballasting scheme shall be provided to the contractor to insure that each module is lowered in a trim position. Once properly positioned on the setting pads, sufficient ballast concrete will be added to achieve a minimum flotation factor of safety of 1.05.

B.2.62. Exterior containment sheet piling will be driven three to five feet into the underlying clay strata. The foundation tremie concrete will then be injected through the base via prepositioned four-six inch diameter grout pipes. Each tube will be valved. A set of pipes is located in each base compartment and along the containment piling at 14' O.C.. Each set will contain an intake pipe, a spare intake pipe, and an outlet pipe that will extend above the piezometric head. (Installation notes are included on plate B-25 for the ship lock and B-35 for the barge lock). Concurrent with the base grouting operation will be the installation of the tension anchors. Each tension pile will be connected by four tension anchors. An outer pipe sleeve and O-ring seal, compressed by the weight of the lower base, prevents water and tremie concrete from entering the tension pile. The pile cap and anchors are then grouted. The installation procedure is included on plate B-24B.

B.2.63. The remaining lock wall ballast will be pumped in after the foundation concrete and tension anchors have cured.

B.2.64. Complete all wall armor. Construct lock wall control houses. Complete module mechanical and electrical installation.

B.2.65. Install the southwest guidewall and dolphin after the riverside gatebay (GB-1) is complete.

B.2.66. Repeat steps 4 thru 7 for the next three modules, CM-2, CM-3 and GB-4, respectively.

B.2.67. Construct the module joint once the next module is installed and fully ballasted. Alternately, the joint may be constructed prior to tremieing the foundation 4 foot void of the following module. By aligning the module on the setting pads, and dewatering the joint void, the water pressure will compress the free module against the grouted module forming a tight seal. The outer seals are temporary and only needed to permit the installation of the inner seals. The outer vertical seals will be adjusted by divers to form a tight seal. Once the outer seal is adjusted, the space between the adjoining end dams will be dewatered. The inner wall, culvert and gallery 3-bulb waterstops will be grouted to the abutting module. The exposed joint surface will be protected with ultra-high-molecular-weight (UHMW) pads. The UHMW pad is slotted on one side to permit movement in the joint.

B.2.68. After the module joint is complete, the inner module end dams will be removed and transported to the graving site for use with the third (CM-3) and fourth (GB-4) modules.

B.2.69. Install the northwest guidewall after the last module (GB-4) is complete.

Open Lock as Pass-Through Lock

B.2.70. Test all machinery and flood lock chamber.

B.2.71. 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).

B.2.72. Remove the bypass timber fender. The three south end bypass channel protection cells and riprap will remain.

B.2.73. 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

B.2.74. 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).

B.2.75. 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.

B.2.76. 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.

B.2.77. Complete site work.

REQUIRED CLEARANCES AT FLORIDA AVENUE FOR THE FLOAT-IN LOCK PLAN

B.2.78. The float-in lock construction is contingent upon removal by local interests of the width restriction of about 90 feet at the existing Florida Avenue Bridge. Plans for replacing this bridge are complete, and construction of the new low level vertical lift bridge is scheduled for 1997. The replacement railroad bridge includes two at-grade lanes for vehicular traffic. The new bridge provides 156 feet of vertical clearance and 300 feet of horizontal clearance. A total of $11 million has been appropriated for construction of this bridge. Additional funding is expected to complete bridge construction. If construction of this bridge fails to be realized, the alternate measure is to construct smaller modules. Removal of existing timber fendering at Florida Avenue widens the opening to 105 feet. The smaller units would be substantially more time-consuming and expensive. The alternate lock construction would require fourteen units (in lieu of four).

B.2.79. Note also that 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 feet clear (with fenders). It must also be removed for the four-piece float-in plan to be workable. Removal of the siphon is a local interest responsibility in conjunction with replacement of the Florida Avenue bridge. A replacement siphon is not required.

B.2.80. The Louisiana Department of Transportation and Development (LDOTD) was designing (in mid-1993) a high-level, four-lane vehicular crossing with a 300-foot minimum horizontal clearance to suit United States Coast Guard requirements. The 300-foot clearance meets and actually exceeds the Corps' requirements for the float-in lock plan. However, the LDOTD has stopped work on this project and turned it over to the City of New Orleans. The project's final status is undetermined, but local discussion is still on going. Although somewhat impacting detour plans, the bridge status does not affect available clearance for the float-in modules.