C6. STRUCTURAL DESIGN.

C6.1 General.

C6.1.1 Structure Description. The replacement lock will be located northwest 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 earthen cofferdams, dewatering systems and cast-in-place concrete.

C6.1.2 Flood Protection. The new lock shall be tied into the existing East Atchafalaya Basin protection levee with the use of pile founded reinforced concrete I-walls and T-walls.

C6.2 References.

C6.2.1 Corps of Engineer Publications.

a. EM 1110-2-2000, Standard Practice for Concrete for Civil Works, Jul 94

b. EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures, Jun 92

c. EM 1110-2-2105, Design of Hydraulic Steel Structures, Mar 93

d. EM 1110-2-2502, Retaining and Flood Walls, Sep 89

e. EM 1110-2-2504, Design of Sheet Pile Walls, Mar 94

f. EM 1110-2-2602, Planning and Design of Navigation Locks, Sep 95

g. EM 1110-2-2703, Lock Gates and Operating Equipment, Jun 94

h. EM 1110-2-2906, Design of Pile Foundations, Jan 91

i. ETL 1110-2-307, Flotation, Aug 87

j. ETL 1110-2-355, Structural Analysis and Design of U-frame Lock Monoliths, Dec 93

C6.2.2 Technical Publications.

a. American Concrete Institute, Building Code and Commentary, ACI 318-99

b. American Institute of Steel Construction, Manual of Steel Construction, Allowable Stress Design, Ninth Edition, 1989.

c. American Association of State Highway and Transportation Officials, Standard Specifications for Highway Bridges, AASHTO, Sixteenth Edition, 1996

d. American Welding Society, Structural Welding Code, (AWS D1.1-96)

C6.2.3 Computer Programs.

a. CE Structural Analysis Program, "C-Frame", CASE Program No. X0030

b. Structural Analysis and Design Software, "STAAD-III", release 23W, Research Engineers

c. CE Pile Group Analysis Program, "CPGA", CASE Program No. X0080

d. CE Strength Analysis of Concrete Structural Elements, "CGSI", CASE Program No. X0061.

C6.3 Design Criteria.

C6.3.1 General. Design criteria shall be according to the referenced EM's, TL's, and technical publications.

C6.3.2 Material Weights.

Item	Unit Weight (PCF)
Water	62.5
Select Sand	122.0 ($K_O = 0.5$)
Semi-compacted Fill	110.0 ($K_O = 0.8$)
Silt	117.0
Stone	132.0 ($K_O = 0.5$)
Fully Compacted Fill	116.0 ($K_O = 0.8$)
Non-plastic Embankment	120.0
Concrete	150.0
Steel	490.0
Asphaltic Concrete	100.0

C6.3.3 Design Stresses.

C6.3.3.1 Concrete. The allowable stresses used in the design of conventionally reinforced concrete structures shall be according to the Building Code Requirements for Structural Concrete (ACI 318-99) and Commentary (ACI 318R-99). Values contained in the Code shall be modified by the strength requirements for reinforced concrete hydraulic structures as directed by EM 1110-2-2104.

C6.3.3.2 Reinforcement. The design strength of reinforcement shall be based on the use of ASTM A 615, Grade 60, steel having a yield strength of 60,000 psi.

C6.3.3.3 Structural Steel. The basic stresses for structural steel shall be according to the 9th Edition of the AISC, Manual of Steel Construction Allowable Stress Design, as modified by EM 1110-2-2101. This EM requires that all AISC allowable stresses be reduced by 17%, as a basis for design. EM 1110-2-2105, Design of Hydraulic Steel Structures shall also be used where applicable.

C6.3.3.4 Welds. Allowable stresses for the design of welds shall be according to the latest AWS Welding Code as modified by EM 1110-2-2101.

C6.3.3.5 Sheet Piling. Allowable stresses for permanent sheet piling will not be more than 50% of the yield strength in accordance with EM 1110-2-2504. Minimum thickness for corrosion control shall be 0.335 inches.

C6.3.3.6 Service Piling. The allowable concrete stresses used for prestressed concrete piling will be in accordance with EM 1110-2-2906.

Uniform Axial Tension	0
Bending (Extreme Fiber)	
Compression	$0.40\ f'_c$
Tension	0

For combined axial load and bending, the concrete stresses should be proportioned so that:

$$f_a + f_b + f_{pc} \leq 0.40\ f'_c$$

$$f_a - f_b + f_{pc} \geq 0$$

Where:

f_a = computed axial stress (tension is negative)

f_b = computed bending stress (tension is negative)

f_{pc} = effective prestress

f'_c = concrete compressive strength

C6.3.4 Allowable Overstress. The allowable overstresses used in the design of the structural components shall be as shown in the following table:

Loading Condition	Allowable Overstress
Construction	16-2/3 %
Maximum Operating	0 %
Project Flood	0 %
Project Flood + Freeboard	33-1/3 %
Dewatered Case	33-1/3 %
Temporary Loadings (Boat Impact, Hawser, Wind and Debris)	33-1/3 %

C6.4 Loading Conditions.

C6.4.1 General. A range of possible loading conditions will be considered for both the planned operation and construction of the gate monoliths. In addition to dead and live loads, the effects of superimposed temporary loading such as boat impact, wind and debris will be considered.

C6.4.2 Construction. Lateral earth pressures are developed considering the type of backfill, and where appropriate, will be considered as loading on various structural elements during the entire construction period.

C6.4.3 Hydraulics. The structure will be investigated for a range of possible hydraulic and operational modes, including the following:

a. Normal Operating (HW = El. 11.0, TW = El. 3.0)

b. Maximum Operating (HW = El. 23.8, TW = El. 0.0)

c. Project Flood (HW = El. 28.7, TW = El. 0.0) FS Gate *

d. Project Flood + Freeboard (HW = El. 30.7, TW = El. 0.0) FS Gate *

e. Maintenance Dewatering (HW = El. 5.7, TW = El. –0.3)

f. Maximum Reverse Head (HW = El. 7.9, TW = El. 0.0)

* No Lock Operation

C6.4.4 Uplift. Uplift pressure is comprised of position pressure and seepage pressure. Position pressure is based on the hydrostatic head across the base of the structure. Seepage pressure is a function of the equipotential flowline due to the difference in water elevation between headwater and tail water. To insure a conservative design, three uplift conditions will be used in the design for all monolith foundations. Uplift conditions 1 and 2 will be derived by assuming the sheet pile cut-off under the structure as effective. Uplift condition 3 will be derived by assuming the sheet pile cut-off as ineffective.

a. Uplift Condition 1 assumes the upstream sheet pile cutoff is impervious.

b. Uplift Condition 2 assumes the downstream sheet pile cutoff is impervious.

c. Uplift Condition 3 assumes-uniformly varying pressure between the upstream and protected side sheet piling cutoffs.

C6.4.5 Superimposed.

C6.4.5.1 Soil Drag. Drag due to differential settlement of the surrounding backfill with respect to the structure shall be incorporated into the design. In lieu of more accurate analysis drag shall be calculated as:

$$(P_{soil\text{-}at\text{-}rest}) \times 0.5 \times (\text{Tangent of Internal Angle of Friction})$$

C6.4.5.2 Debris Loads. A uniform load of 1.5 kips per foot in the direction of flow shall be applied 1 foot above the waterline.

C6.4.5.3 Wind Load. Wind load in accordance with the latest edition of ASCE 7, but not less than 50 psf shall be applied to the applicable structural components.

C6.4.5.4 Boat Impact. A boat impact of 125 kips shall be applied as a point load on the gate and impact zone of the adjoining concrete walls. In addition a uniform load of 5.0 kips per foot shall be applied 1 foot above the water line to the applicable structural component.

C6.4.5.5 Equipment Surcharge. A surcharge of 200 psf shall be applied to the applicable structural components.

C6.4.5.6 Walkways. A uniform live load of 150 psf shall be applied.

C6.4.5.7 Hawser Loads. A pull of 160 kips shall be used for the design of line hooks and check posts.

C6.5 General Description of Structural Elements (Recommended Plan).

C6.5.1 Pile Foundations. The design Factors of Safety comply with EM 1110-2-2906. The pile capacity used considers that a pile test shall be performed. Concrete piles were selected based on capacity, economics, and availability. Pile loads and stresses shall be determined using the rigid base analysis program CPGA. For the lock, vertical piles will provide all of the lateral resistance. Tension anchors in the form of embedded reinforcing bars in the tops of the piling will be provided where required to resist tension.

C6.5.2 Sector Gates.

C6.5.2.1 General. Structural design will be in accordance with Corps engineering guidance and applicable industry standards. The Corp criterion is specified in EM1110-2-2105 and EM 1110-2-2703. The sector type gates will be all welded structural steel construction. The gates will have a central angle of approximately 75□. The radius to the inside of the skin plate will be 36 feet 3 inches. Loads applied to the gates skin plate will be transmitted to horizontal girders through vertical tees. The girder load will then be transmitted through four vertical trusses and horizontal framing to a hinge and pintle. 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. See plate S11 for details.

C6.5.2.2 Gate Support. The gate frames are 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 is used in the hinge to compliment the spherical pintle. Horizontal reactions are transferred to the lock wall through the bronze bushings. All vertical loads are 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.

C6.5.3 Lock Chamber Monoliths. Lock chamber monoliths will enclose the lock between the upper and lower gate bay monoliths. The proposed lock chamber is designed to be constructed with 27 each – 40 foot monoliths, that in conjunction with the gate bay monoliths, will provide a chamber 75 feet wide by 1200 feet long (useable length). The lock chamber floor will be set at El. –15.0 (NGVD) with the top of the wall set at El. 26.8 (NGVD). See plate S1 for details. All of 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 plate S10 for details.

C6.5.4 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. –15.0 (NGVD), with the south top of wall set at El. 31.7 (NGVD) and the north top of wall set at El. 26.8 (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 plates S4 through S9 for details.

C6.5.5 Guide Walls and Dolphins. Parallel guide walls, to guide the long tows into the lock and to provide mooring facilities, will be provided at each end of the lock. The guide walls will be composed of braced pile bents faced with horizontal composite marine timber wales. Guide walls will be provided on both sides of the lock approach. At both the north and south west ends of the lock 1200-foot guide walls will be provided, while both the north and south east ends of the lock will have 400-foot guide walls. Pile supported, steel sheet pile, concrete filled dolphins will be provided at the end of each timber guide wall. See plates S3, S13 and S14 for details.

C6.5.6 Tie-In Floodwalls. Reinforced concrete T-Walls and I-walls will connect the gate bay monolith to the existing East Atchafalaya Basin Protection Levee west of the new lock.
The T-walls and I-walls east of the new lock will connect the gat bay monolith to a new levee that ties into the lock closure. The T-walls will be supported on prestressed concrete piling and the I-wall on Z-shaped sheet piling. See plates S15 through S17 for details.

C6.5.7 Tie-In Levee. New earthen levees will be constructed to connect concrete T-Walls and I-walls to the Lock Closure and connect the lock closure to this existing East Atchafalaya Basin Protection Levee. The new levees will become part of the existing East Atchafalaya Basin protection Levee. The majority of the new levees will fall on top of the existing earthen chamber for old lock.

C6.5.8 Closure Dam. An earthen closure dam at El. 33.0 will close the existing lock earthen chamber, after completion of the replacement lock. See plates S18 and S19 for details.

C6.6 Other Structures Investigated.

C6.6.1 General. For project optimization two alternatives were investigated for the replacement of the existing lock, a 1,200-foot earthen chamber with T-wall, and a 1,200-foot concrete U-shaped chamber. Each of these alternatives involved different lock widths including, 110 feet, 90 feet, 84 feet and 75 feet. In addition, replacing the lock in kind with a 770-foot long by 56-foot wide lock was also investigated.

C6.6.2 In-Kind Replacement. A base line estimate was prepared for replacing the existing lock with a lock of the same useable length and width (770 feet x 56 feet). Essentially, the structure consisted of two pile founded gate bay monoliths, with the south top of wall set at El. 31.7 (NGVD) and the north top of wall set at El. 26.8 (NGVD), housing the sector gates and machinery, a riprapped earthen chamber used in conjunction with a pile supported reinforced concrete T-wall, composite timber faced guide walls and steel sheet pile dolphins. Structural features incorporated current design criteria to meet current flood control objectives. The lock chamber floor was set at El. –15.0 (NGVD) with the top of the wall set at El. 26.8 (NGVD). Pile founded reinforced concrete T-Walls and I-walls were used as tie-in floodwalls.

C6.6.3 Earthen Chamber with T-wall. Estimates were prepared for replacing the existing lock with locks having a useable length of 1,200 feet and varying widths including 110 feet, 90 feet, 84 feet and 75 feet. Essentially, the structure consisted of two pile founded gate bay monoliths, with the south top of wall set at El. 31.7 (NGVD) and the north top of wall set at El. 26.8 (NGVD), housing the sector gates and machinery, a riprapped earthen chamber used in conjunction with pile supported reinforced concrete T-walls, composite timber faced guide walls and steel sheet pile dolphins. Before the T-walls can be constructed atop the new chamber/levee fill, the area will require preloading to eliminate settlements and any additional load on the piling due to negative skin friction. Two methods of preload construction were investigated. The first involved simply allowing the preload section to sit for two years with no other construction activities occurring. The second involved the use of wick drains, geotextiles, circular celled walls and concurrent construction activities to reduce the total construction time. Structural features incorporated current design criteria. The lock chamber floor was set at El. –15.0 (NGVD) with the top of the wall set at El. 26.8 (NGVD). Pile founded reinforced concrete T-Walls and I-walls were used as tie-in floodwalls.

C6.6.4 U-shaped Chamber. Estimates were prepared for replacing the existing lock with locks having a useable length of 1200 feet and varying widths including 110 feet, 90 feet, 84 feet and 75 feet. Essentially, the structure consisted of two pile founded gate bay monoliths, with the south top of wall set at El. 31.7 (NGVD) and the north top of wall set at El. 26.8 (NGVD), housing the sector gates and machinery, pile supported reinforced concrete U-shaped monoliths, composite timber faced guide walls and steel sheet pile dolphins. Structural features incorporated current design criteria. The lock chamber floor was set at El. –15.0 (NGVD) with the top of the wall set at El. 26.8 (NGVD). Pile founded reinforced concrete T-Walls and I-walls were used as tie-in floodwalls.

C6.6.5 Float–In Lock Structure. A float-in lock structure was considered and not adopted. This type of structure is typically most beneficial where rights-of-way limitations, cofferdam and graving site construction and flotation dredging, produce excessive costs.

C6.6.6 Float-In Sector Gate Structure. So that the flood control componenet of the tentative plan could be more clearly identified, a float-in sector gate type structure that would provide flood protection in front of the existing lock was investigated. The float-in concept was necessary for this alternative, so that lock down time would be minimized. The structure consisted of a pile founded, post tensioned and reinforced concrete sector gate monolith with the floor set at El. –15.0 (NGVD) and the top of walls set at El. 31.7 (NGVD). The monolith would house the sector gates and machinery that would provide the same 56 foot wide opening for tows as the existing lock. The structure would be constructed at an adjacent graving site surrounded by an earthen cofferdam, to provide flood protection during construction. Upon completion, the cofferdam would be breached to allow the structure to be floated out and positoned above its foundation. Once lowered into place, the pile foundation would be grouted to the structure's concrete base. To keep the tows aligned, the existing guide wall would be extended between the existing lock and the new structure, as well as a new composite timber faced guide wall and sheet pile dolphin constructed on the approach to the new structure. To complete the line of flood protection, approximately 240 linear feet of pile supported reinforced concrete T-wall and I-wall would be constructed to tie into the existing East Atchafalaya Basin protection levees.

C6.7 Analysis of Existing Structure.

C6.7.1 Review of Flood Protection. A structural stability study of the existing south gatebay structure, completed in 1980, indicated that it would be overstressed for an upper pool of El. 23.5 (NGVD) and a lower pool of El. 4.00 (NGVD). If temporary flood protection is provided, above the existing top of structure El. 24.0 (NGVD), it would require modifications to both the gates and the gatebay monolith. The lock would have to be closed to traffic, so that the chamber pool could be controlled to offset any increased head.

C6.7.2 Review of Structural Design Criteria. A comparison of the latest concrete design criteria, with that in effect during the locks design, indicates that the design is still adequate for flexure; however, the new design requirements for shear are much more stringent than those used in the past and would indicate some overstress. The latest ACI criteria for development and splice lengths has increased significantly, indicating that the existing bars do not meet current standards. Current allowable stresses for structural steel elements are slightly lower than those used during the time of the design, which would indicate some overstress. Also, it is unlikely that the gates were designed for the 125 kip impact force required by the latest steel gate criteria. The structure was not designed for the latest seismic loading; however, it is unlikely that it would govern any aspect of the design, due to the allowable overstress for this loading condition. While a review of current design criteria indicates that the structure does not meet several current design requirements for both the concrete and steel elements, they are deemed adequate for their original design loads based on their successful performance.

C6.8 Risk Analysis. Reliability assessment procedures for existing navigation structures typically require rigorous analysis and reporting with the main purpose of identifying project features which are in need of major rehabilitation to improve their reliability or operational efficiency. Since this structure is being evaluated for lockage capability and not structural reliability, CECW-PC memorandum dated Aug 96 stated, "A reliability analysis is required but the form of the reliability analysis should be streamlined. The report will present the age and condition of all other locks in the system." Therefore, the information presented in Annex 7, includes a general description of each lock; information on inspection, maintenance and repair histories, including downtime; a discussion of the present condition of each lock and any known deficiencies; major maintenance, improvements, and repair activities anticipated through the operation and maintenance program; and a discussion on the potential for major rehabilitation.

C6.8.1 Historical Data. Information on the repair and maintenance history of each lock, including downtime, comes from three sources: summaries of the Periodic Inspection Reports for the locks, summaries of repairs made during dewaterings of the locks, and information gathered from the Summary of Monthly Reports of Operations prepared by the lockmasters. The amount of data presented and to what detail was included for each lock varies greatly for each lock because of the number of reports found for the individual locks and the degree of details recorded by the lockmasters in these reports.

C6.8.1.1 Repair and Maintenance Histories. This information concerns repairs to the lock and its appurtenances and not minor routine maintenance items (i.e., periodic lubrications) and repairs on facility buildings and other non-lock items. A listing of closures due to maintenance and repair, and a summary of maintenance and repair operations for each lock can be furnished upon request.

C6.8.1.2 Downtime Histories. The information presented includes only the downtimes of the locks that are lock-related and not items such as weather delays, oil slicks, maritime incidents and debris in the lock. The phrase "lock maintained/repaired" is shown for some of the closures since the Summary of Monthly Reports of Operations only mentions that the lock maintenance crews were working in the lock without specifying further details. Although some locks have minimal downtime histories presented, it can be seen that there has been minimal unscheduled downtime and the lock is in satisfactory operating condition.

C6.8.2 Existing Lock Condition. Information on the present condition of each lock comes from summaries of the Periodic Inspection Reports for each lock. All locks were found to be safe, structurally stable, well maintained and in satisfactory operating condition.

C6.8.3 Anticipated Major Maintenance, Improvements, or Repairs. Information on anticipated major maintenance, improvements, or repairs for each lock comes from summaries of the Periodic Inspection Reports and from the Lock Repair and Closure Schedule developed by Operations Division.

C6.8.4 Assessment of Potential for Major Rehabilitation. Based on the information gathered, the potential for major rehabilitation of the lock is discussed.