C3. GEOTECHNICAL INVESTIGATIONS AND DESIGN.

C3.1 General. This section includes the soils investigations and foundation design for the navigation lock. The lock consists of I-walls, levees, T-walls, pile supported sector gates and pile supported U-frame chamber.

C3.2 Geology.

C3.2.1 Physiography. The study area is in the Mississippi River alluvial plain. Specifically, it is in the Atchafalaya River Basin along the Gulf Intracoastal Waterway at Bayou Sorrel Lock. The main physiographic features consist of distributaries, abandoned distributaries, and backswamp overlain by natural levees adjacent to active distributaries. This is an area of low relief averaging approximately +6 feet NGVD on the natural levee to +1 foot NGVD on the backswamp areas.

C3.2.2 General Geology. Only recent geologic events are pertinent to this project. Approximately 5,000 years ago sea level reached its present level after being lowered by the last glaciation, and the Mississippi River began to migrate across the alluvial valley depositing several different deltaic lobes. Several cycles of deposition and erosion have occurred in the project area as the Mississippi River shifted back and forth across the deltaic plain. Approximately 2,000 years ago the Mississippi River shifted west and began building the Lafourche Delta sequence. The Mississippi River shifted eastward approximately 1,000 years ago to its present Plaquemines Delta sequence. After this eastward shift additional deposition occurred in the project area when the Atchafalaya River and associated distributaries occupied the basin abandoned by the Mississippi River. Construction of levees and other development has eliminated further deposition of fluvial/deltaic sediments in this area.

C3.2.3 Subsidence. Regional subsidence and geosynclinal downwarping have been occurring since the end of the Pleistocene epoch. The long-term relative subsidence rate in the project area is <1 foot per century. In addition, man induced subsidence of the ground surface has occurred in reclaimed swamp due to the shrinking of highly organic soils after drainage.

C3.2.4 Investigations Performed. Fourteen borings taken in 1994 and three borings taken in 1995 were used in this report and accompanying cross-sections. All borings were taken in and around the present location of Bayou Sorrel Lock, as well as the proposed location for the new lock. These borings exceeded 90 feet in depth with the deepest being 120 feet. In addition, other reports, maps, and publications were used for the interpretation of physiography and the surface and subsurface conditions of the area.

C3.2.5 Subsurface Conditions. The project area is overlain by natural levee deposits and in the sections from distance 0 to 1875 feet and from distance 4144 feet to the end of the sections (see Plates). Natural levee deposits consist of interbedded soft to stiff fat and lean clays with occasional lenses of silt. These deposits average 4 feet thick and range in elevation from +8 to -2 feet NGVD. Backswamp deposits underlie the natural levee deposits and in the sections occur at the surface from distance 1875 to 4144 feet. Backswamp deposits consist of interbedded very soft to stiff fat clay with occasional layers and lenses of very soft to stiff lean clay and silt and lenses of silty sand. These deposits average 95.2 feet thick and range in elevation from +6 to -109 but extend to an unknown depth where the borings do not penetrate the entire thickness of the backswamp deposits. Lacustrine deposits underlie the backswamp deposits from distance 2325 feet to 5568 and within the backswamp deposits from distance 5794 to the end of the project. Lacustrine deposits consist of interbedded silty sand, silt, and soft to stiff clays. These deposits average 20 feet thick and range in elevation from -60 to -86 feet NGVD and from -16 to -37 feet NGVD. Substratum sands underlie the lacustrine and backswamp deposits and consist of interbedded sand and silty sand with occasional lenses of silt and clays. The surface of the substratum sands averages -86 feet NGVD, and these deposits extend to an unknown depth. Although, none of the borings penetrate the Pleistocene surface, the top of the Pleistocene deposits in the project area is estimated to be -325 feet NGVD. The soils in this area are gray to brown clayey alluvium deposited as natural levees from distributaries of the Mississippi River. These soils are level, poorly drained, frequently flooded, and very fertile.

C3.2.6 Ground Water. The first aquifer encountered in the subsurface is located between approximately –20 and –40 feet NGVD. This aquifer is discontinuous and composed of silt and silty sand. A piezometer placed in this aquifer indicates that this aquifer is hydraulically connected to the Atchafalaya River. During low flow periods the water level in this aquifer appears to be influenced by the Mississippi River rather than the Atchafalaya. A second shallow aquifer is located from approximately –85 to greater than –120 feet NGVD. This aquifer is composed of silty sand and sand and is hydraulically connected to the Mississippi River. The following table lists some selected piezometer readings for 2 piezometers located at Bayou Sorrel Lock. Any engineering design in the study area should consider the hydraulic head in these shallow aquifers.

TABLE C22

SELECTED PIEZOMETER READINGS - BAYOU SORREL LOCK

Date of Reading	Piezometer # 4	Piezometer # 2
	Tip Elevation 30.5	Tip Elevation 97.7
	Elevation of Water Level	Elevation of Water Level
19-Mar-97	8.98	11.50
24-Apr-97	8.81	12.77
9-Sep-97	3.13	5.89
7-Oct-97	2.56	4.69
20-Nov-97	2.66	3.59
8-Jan-98	5.13	4.73
5-Feb-98	7.84	7.90
3-Mar-98	7.63	8.77
14-Apr-98	7.55	10.50
13-May-98	7.81	11.36
10-Jun-98	6.43	10.86
8-Jul-98	5.69	9.65
4-Aug-98	4.38	8.73
16-Sep-98	4.53	6.64
18-Nov-98	3.98	4.60
7-Jan-99	4.69	4.86
3-Feb-99	6.12	6.38
8-Apr-99	9.63	7.08
21-Jun-99	9.17	5.83
30-Aug-99	5.38	2.88
27-Sep-99	3.83	2.56
28-Mar-00	3.42	4.68
14-Aug-00	3.61	2.79
27-Nov-00	1.34	3.94
17-Jan-01	2.19	4.28
22-Mar-01	7.37	8.05
25-Apr-01	7.27	8.95
20-Jun-01	7.71	6.96
10-Sep-01	4.05	4.1
10-Apr-02	5.15	7.66

Another aquifer exits at a depth of 200 to 450 feet below the ground surface and contains fresh but hard water. Deeper aquifers occur at a depth of 1,000 to 2,500 feet. Ground water in the area tends to be hard with a high iron content and an increasing amount of chloride to the south and southwest away from the Mississippi River. In the immediate project area some oilfields have indicated a lack of any fresh water.

C3.2.7 Earthquake Activity. Although some publications indicate faults in southern Louisiana, none are found at the project site. An oil prospect at a salt dome is present approximately 8-9 miles northeast of the project site, and the dome could have some minor faulting associated with it. However, no significant seismic activity is expected in the project area.

C3.2.8 Mineral Resources. The oil prospect to the northeast is not expected to significantly impact the project site. No other mineral resource development is expected near the study area.

C3.2.9 Conclusions. The backswamp environment in this area indicates moist, organic clays of high compressibility. Any dewatering required at the project site would increase the compaction and subsequent subsidence. Precautions will have to be taken to eliminate problems should this occur. No other problems related to the geology of this area are apparent.

C3.2.10 References.

a. Howe, H. V. et. al., 1938, "Reports on the Geology of lberville and Ascension Parishes", Geological Bulletin No. 13, Department of Conservation, Louisiana Geological Survey.

b. Krinitzsky, E. L. et. al., 1950, "Geologic Investigation of Faulting in the Lower Mississippi Valley", Technical Memorandum No. 3-31 1. Waterways Experiment Station.

c. Spicer, B. E. et. al., 1977, "Soil Survey of lberville Parish, Louisiana", United States Department of Agriculture, Soil Conservation Service, in cooperation with the Louisiana Agricultural Experiment Station.

C3.3 Field Exploration. Nine continuous undisturbed 5-inch diameter soil boring were made in the project area for this project. Boring 1-BYSU and 3-BYSU were made to a depth of 120 feet at the centerline of the East Atchafalaya Basin Protection Levee (EABPL). Borings 2-BYSU and 4-BYSU were made to a depth of 120 feet at the protected side toe of the EABPL. Borings 5-BYSU, 6BYSU, 7-BYSU and 8-BYSU were made to a depth of 120 feet along the centerline of the channel. Borings 5-BYSU and 6-BYSU were made at the sector gates. Boring 9-BYSU was taken to a depth of 120 feet at the top of the channel bank. Borings 196-UE, 196-AUE, 197-UE and 197-AUE were taken for a previous EABPL levee-raising project. Borings 196-UE and 196-AUE were taken at the levee centerline to a depth of 120 feet. Borings 197-UE and 197-AUE were taken at the floodside levee toe to a depth of 95 feet. The individual logs of these 13 undisturbed borings are shown on plates F1 through F13. Eight general type borings were made using either a 1-7/8 inch Inner Diameter (ID) core barrel or a 1-3/8 inch split spoon sampler. Borings 1-BYSG and 2-BYSG were made along the centerline of the channel to a depth of 100 feet. Borings 3-BYSG, 4-BYSG, 5-BYSG, 6-BYSG, and 7-BYSG were made to a depth of 100 feet along the centerline of the landside earth chamber levee. Boring 8-BYSG was taken to a depth of 100 feet at the top of the channel bank. The locations of the undisturbed and general type borings are shown on plate G2. Five 1.5- inch diameter piezometers were installed at the site. Piezometer PIEZ- 1 was installed at tip El. -108 at the north sector gate. PIEZ-2 and PIEZ-3 were installed in the same borehole at the south sector gate to El. -98 and El. -71. PIEZ-4 and PIEZ-5 were installed to El. -30.5 and El -35.3 south of the new lock. The locations of the piezometers are shown on Plate G2.

C3.4 Laboratory Tests.

C3.4.1 General. All samples obtained from the borings were visually classified. Water content determinations were made on all cohesive soil samples. Unconfined compression (UC) shear tests and Atterberg tests were made on selected samples of cohesive soils. Grain size analyses were made on selected samples of granular soils. Water content determinations, (UC) test results and the D_{10} determined from grain size analysis are shown adjacent to the logs on the boring profiles presented on Plates F14 through F16. Unconsolidated- Undrained (Q), Consolidated - Undrained (R), and Consolidated - Drained (S) shear tests and Consolidation (C) tests were made on representative soil samples. These tests are summarized on the boring logs shown on plates F1 through F13. The individual shear strength data sheets are shown in Annex 3.

C3.4.2 Design Shear Strengths. Design shear strength parameters are shown on plates F17 and F18. Three design shear strength profiles are used for the site: (1) the existing EABPL (2) the new lock, tie-in levees and channel (3) the new EABPL incorporating the existing Bayou Sorrel Lock earth chamber levee.

C3.5 Design Problems. Design problems considered were:

a. Stability of the existing EABPL into the new lock channel.

b. Stability of the existing EABPL into the sector gate excavation.

c. Stability of the excavation slopes.

d. New lock site located southwest of the failure of the original Bayou Sorrel Lock excavation in June 1946.

e. Dewatering sands at excavation site.

f. Pile capacities for sector gate and chamber monoliths.

g. Lateral earth pressures on lock walls.

h. I-wall stability for tie-in walls.

i. Settlement analyses for closure across existing Bayou Sorrel Lock and tie-in levees.

C3.6 Lateral Earth Pressure. Backfill adjacent to the structure and retaining walls will consist of a sand wedge to minimize lateral earth pressure. At rest coefficients (k_o) for the backfill materials were used to determine the lateral earth pressure against the structure. For sand backfill, a lateral earth pressure coefficient of 0.5 was used for design and for clay backfill, a lateral earth pressure coefficient of 0.8 was used. Total unit weights were used above water, and submerged unit weights below the water. The analyses to develop the lateral earth pressure was based on DIVR 1110-1-400 for sloping surfaces which uses a friction angle equivalent to the above recommended earth pressure coefficients for sand and clay. The lateral earth pressure diagrams for the construction, operating, and dewatering cases are shown in cross sections on plates F19 and F20.

C3.7 Hydrostatic Pressure Relief and Underseepage.

C3.7.1 Hydrostatic Pressure Relief. To build the structure in the dry and insure stability of the structure excavation during construction, hydrostatic pressure relief will be provided in the pervious layers in the structure excavation area. Temporary piezometers will be installed in the pervious layers to monitor the pressure during dewatering. The method of lowering the groundwater is to be left to the construction contractor with performance specifications being prepared on an "end-result" basis. The specifications will allow the use of wells, sumps, pumps, etc., as well as well points. The groundwater at the site will be tested both for mineral and biological sources to determine the potential for clogging the dewatering systems. The deep sands are connected to the Mississippi River. The excavation for the gate bay and chamber in the dry will require dewatering. Piezometric readings are shown in Annex 2. Gage readings for the Mississippi River at Baton Rouge and gage readings at Bayou Sorrel Lock are also shown in Annex 2. The Baton Rouge gage is the nearest gage on the Mississippi River with data during the time piezometer readings were taken. The shallow sands will also require dewatering. A piezometric headline of El 27.0 was used for the shallow sands. A piezometric headline of El. 16.0 was used for the deep sands. The dewatering system presented on plate F21 is for cost estimating purposes and for use in evaluating the adequacy of the Contractor's proposed pressure relief system. A soil boring must be made during the Design Documentation Report to determine the depth of the deep sand. The depth of the deep wells on plate F21 will be adjusted based on the depth of the deep sand aquifer from the soil boring. The depth of the deep sand will be included in the plans and specifications. Both the upper sands and the lower sands must have water quality testing done and this information will be included in the plans and specifications.

C3.7.2 Underseepage at Sector Gate Structure. A sheet pile cutoff will be placed below the sector gate bay. Harr's Method was used to determine the sheet pile tip penetration, and analyses are shown on Plate F43.

C3.8 Pile Foundations.

C3.8.1 Ultimate compression and tension pile capacities versus tip elevations developed for 14-inch square prestressed concrete piles for the sector gates and the chamber monoliths are shown on plates F22 and F23. Values of soil to frictional resistance, lateral earth pressure coefficients for compression and tension, and bearing capacity factors used to compute pile capacities are shown in Table C22. The tip elevations for cost estimating purposes are based on applying the factors-of-safety shown in Table C23.

C3.8.2 Subgrade moduli curves for estimating lateral resistance of the soil beneath the sector gate structure and pile supported T-walls are shown on plates F22 and F23.

C3.8.3 Settlement. The existing Bayou Sorrel Lock gate bays have settled less than 1.5 inches in 39 years (1991 surveys). The existing Bayou Sorrel Lock is supported by timber piles tipped in the sand stratum at El -83.5 MLG (El -84.28 NGVD). The new Bayou Sorrel Lock will be supported by 14-inch concrete piles tipped in the sand stratum below El -82.0 NGVD. The estimated maximum settlement of the new north gate bay structure will be less than 1-inch at the T-wall tie-ins to the EABL. The estimated maximum settlement of the new south gate bay structure will be less than 0.5-inches since there is no levee tie-in. The estimated maximum settlement of the chamber monolith will be less than 0.5-inches.

TABLE C23
PILE CAPACITIES FOR Q AND S CASES

	Q-CASE						S-CASE					
	ϕ	K_c	K_t	N_c	Nq	δ	ϕ	K_c	K_t	Nc	Nq	δ
Clay	0°	1	0.7	9	1.0	0 °	23°	1	0.70	0	10	23°
Silt	15°	1	0.5	12.9	4.4	15°	30°	1	0.70	0	10	23°
Sand	30°	1	0.7	0	22	30°	30°	1	0.70	0	10	23°

TABLE C24
RECOMMENDED FACTORS OF SAFETY
FOR PILE CAPACITY CURVES

WITH PILE LOAD TEST	WITHOUT PILE LOAD TEST
Q-CASE 2.0	Q-CASE 3.0
S-CASE 2.0	S-CASE 3.0

C3.9 Shear Stability.

C3.9.1 Levees. Stability was determined by the LMVD Method of Planes analysis for a minimum factor of safety of either 1.3 or 1.4 with respect to the design shear strength. A 1.4 factor of safety was used for the EABPL stability analyses into the excavation or the new channel. The borings used to develop a design shear strength profile for the navigation lock are shown on plates F5, F6 and F7. The borings used to develop a design shear strength profile for the existing EABPL are shown on plates F10, F11, F12 and F13. The stratification used to develop a design shear strength profile for the levee closure across the existing lock channel are shown on plates F3 and F4. Plates F24 and F25 show stability analyses of the existing EABPL into the gate bay excavation and into the earth chamber excavation. The existing EABPL levee will be degraded to El 28.7 at the gate bay excavation only. A temporary sheet pile wall will be driven to maintain the net grade of El 30.7. Plate F26 shows the stability analyses of the existing EABPL into the channel excavation for the placement of stone. Plates F27 and F28 show the stability analyses of the temporary landside earth cofferdam into the gate bay. Plate F29 shows the stability analyses of the temporary landside earth cofferdam into the chamber excavation. Plate F30 shows the stability analysis of the temporary landside cofferdam into the existing channel. Plates F31 and F32 show the stability analyses of the first lift levee closure across the existing Bayou Sorrel Lock earth chamber for the high water and low water elevations. The first lift levee will be at El. 29.0 with a sheet pile wall providing protection during the four years in between lifts. Plates F33 and F34 show the stability analyses of the second lift levee closure to El 33.0 for the high water and low water elevations. Plate F35 shows a mass stability analysis for the tie-in walls into the new channel. Plate F36 shows the stability analyses of the new EABPL at high water into the new channel with the bottom of the channel at El. -19.0 for a four-foot placement of rock in the wet. Plate F37 shows the stability analyses of the bank into the new channel at low water with the bottom of the channel at El. -19.0 for a four-foot placement of rock in the wet. Plate F38 shows the stability analyses of the bank into the new channel at low water with the bottom of the channel at El. -15.0. The existing Bayou Sorrel Lock chamber levee was enlarged to form the EABPL. Plate F39 shows the stability analyses of the enlarged levee into the channel. Plate F40 shows a plan view of the excavation.

C3.9.2 The original Bayou Sorrel Lock site was approximately 1000 feet upstream of the present day Bayou Sorrel Lock. The 1946 excavation site is located approximately within the existing channel. The west side of the excavation failed into the excavation. The excavation was complete and a 1- to 2-foot layer of very high organic content was being removed. A sand boil broke out in the north gate bay and about 500 cubic yards of fine sand and silt was carried to the surface. Approximately two days later a sudden failure occurred when the west levee dropped 7 feet. Quoting from conclusion of a paper[2] presented to the 2nd International Conference on Soil Mechanics & Foundation Engineering, "The indications are that the hydrostatic uplift in the sand was probably a major contributing factor in the failure of the excavation slope." The east side slope of the north gate bay excavation will be at the west side of the 1946 failure. Inclinometers will be placed along the temporary landside cofferdam at the north gate bay to monitor any movements of the excavation. If excessive movements occur, the gate bay foundation piles may be driven with a follower, the earth cofferdam could be replaced with a sheet pile cofferdam, or the earth cofferdam could be moved away from the excavation.

C3.10 I-Walls. The required penetration of the steel sheet piling below ground surface was determined by the Method of Planes using "Q" shear case design strengths based on data shown on Plate F17. The factors-of-safety were applied to the design shear strengths as follows: ϕ developed = arctan ϕ (tan ϕ available/factor-of-safety) and cohesion/factor-of-safety. Using the resulting shear strengths, net lateral soil and water pressure diagrams were developed for movement toward each side of the sheet pile. With these pressure distributions, the summation of horizontal forces was equated to zero for various tip penetrations and the overturning moments about the tip of the sheet pile were determined. The required depth of penetration to satisfy the stability criteria was determined where the summation of moments was equal to zero. Following is sheet pile wall design criteria used for the tie-in walls and the temporary excavation sheet pile wall for the gate bay excavations:

TABLE C25
TIP PENETRATIONS

Q-CASE
F.S. = 1.5 with water to flowline
F.S. = 1.25 with water to flowline plus freeboard

S-CASE
F.S. = 1.2 with water to flowline
F.S. = 1.0 with water to flowline plus freeboard

If the penetration to head ratio is less than 3 to 1, it is increased to 3 to 1. The flowline is used to calculate head, for penetration ratio to head ratio. Plate F39 shows I-wall stability analyses for the tie-in floodwalls. Plate F40 shows the I-wall stability analyses for the temporary sheet pile wall in the EABPL near the gate bay excavations.

C3.11 T-Walls. A deep-seated analysis utilizing a 1.3 factor of safety incorporated into the soil properties was performed for various potential failure surfaces for the T-walls that tie into the gate bays. The analyses are shown on plate F41. The summation of horizontal driving and resisting forces results in a value that is positive at the base and negative as the elevation of the failure surface is lowered. Since the net driving forces is less than the net at rest force, the structure is assumed to be stable and all loads (vertical and horizontal) must be developed in pile capacity below the slip plane. The estimated maximum settlement for the T-walls is 1 inch.

C3.12 Levee Settlements. The existing EABPL Reach E54, Sta. 29+0255 to Sta. 29+1755, crown elevations from the 1994 survey are between El. 29.8 to 32.5. The levee was raised to El 32.8 in the early 1980's. The predicted settlement from the E-54 Soils Report dated 1980 was 1.6 feet. The settlement ranged between 0.3 feet to 3 feet. At the lowest point the levee is 0.9 feet below the design grade of El. 30.7 (1986 design elevation with two feet freeboard). Based on empirical data the levee sections that are below grade should be raised to El 31.5. According to the E-54 Soils Report, the ultimate settlement from the 1983 levee enlargement will be complete by the year 2000. Any future construction will be to raise deficiencies in the design grade and for the areas where the levee will be lowered to El 28.7 during excavation of the gate bays. The settlement of the levee closure across the existing Bayou Sorrel Lock earth chamber is greater than 8 feet. Since the full closure is across only 156 feet (channel width of 56 feet, bottom elevation -15.0 and slopes of 50 feet with top of bank elevation of 10.0) the crown elevation will be grossed to El 29. The levee will settle to a net elevation of 23.0. After 4 years a second lift to El. 33.0 will be placed. A temporary sheet pile floodwall of 180 feet will span the existing channel cross section to provide temporary protection until the second lift is placed. The two-foot layer of riprap at the bottom of the existing channel will be removed within the existing theoretical levee section to cutoff seepage through the riprap. The tie-in levee embankment to elevation 20.0 for the 40 feet. T-wall monolith will have consolidation settlement of 0.5 feet at the gatebay and will increase to 1 feet at the opposite end of the monolith from the gatebay. The tie-in levee embankment to elevation 23.0 for the I-wall will have consolidation settlement of 1-foot. The tie-in levees should be constructed to 1-foot above design grade and allowed to settle six months before construction of the T-wall and I-wall to eliminate shrinkage of the fill, which will range from 1 to 2 feet. The coefficient for consolidation used in the settlement analyses was 0.1 foot^2/day that is much greater than laboratory calculated values but was backfigured from the test section at E-84 (New Embankment Re-Evaluation Analysis of EABPL Item E-84, Vol. 1). The settlement analyses for the levee closure and tie-in levees were a 3-D analysis (V-Stress). A three-month construction period was assumed for the levee construction. The fill for the T-wall levee embankment should be fully compacted to minimize shrinkage and negative skin friction at the gate bay. The fill for the I-wall levee embankment was assumed to be placed at 90% control compaction. The settlement analyses were based on empirical data and theoretical analyses.

C3.13 Settlement of the T-wall for the Earthen Chamber Lock Alternative. The earthen chamber alternative included a T-wall constructed on new levee fill. A T-wall on a new levee was chosen since a full levee section would extend into the existing channel. The levee fill will cause significant settlement of the T-wall and additional loading on the piles from negative skin friction. Lift construction is not possible since the settlement will take years and must be eliminated before the T-wall is constructed. Two procedures were investigated to minimize the settlement of the new levee fill. Preloading for two years with no other construction activity or preloading with wick drains for one year while excavation and construction of the gate bays proceed. Either procedure will significantly reduce settlements and eliminate negative skin friction. The advantage of the wick drains is that settlement will occur in a maximum of one year reducing the time of construction. Circular sheet pile cells would be constructed tying in the gate bays with the earth chamber T-wall for the wick drain alternative. The circular sheet pile cells would provide cofferdam support during excavation for the gate bays and would remain in place as permanent protection. The circular sheet pile cells would also eliminate the need to preload the area next to the gate bays. The preload with wick drains during excavation of the gate bays would also require some geotextile reinforcement to maintain an acceptable factor of safety for stability into the excavation. For either preloading alternative a portion of the floodside levee toe of the earth chamber levee with T-wall will encroach on the existing channel at the northern side of the new lock. Rock will be placed in the existing channel and clay fill levee will be constructed above the water surface on the rock base. Constructing the gate bays concurrent with preloading will be more expensive because of the wick drains, circular sheet pile cells, geotextile, additional rock and complexity of construction.

C3.14. Float-In Sector Gate Structure. Pile capacity curves were furnished for the float-in cost estimate. At the site where the gate will be lowered in place the excavation below elevation –15.0 will be supported by cantilever sheet pile. The same sheet pile will be used as a seepage cutoff wall after the float-in gate is lowered in place. The same analyses used for the tie-in floodwalls for the U-frame lock were used for the float-in gate structure.

C3.15 <u>References</u>.

a. "Investigation of Foundation Bayou Sorrel Lock", U.S. Waterways Experiment Station, Vicksburg, Mississippi, 25 September 1946.

b. "Failure of an Excavation Slope", Paper No. 8.2, Second International Conference on Soil Mechanics and Foundation Engineering, by S.J. Johnson, Waterways Experiment Station