MISSISSIPPI RIVER - GULF OUTLET
NEW LOCK AND CONNECTING CHANNELS

SECTION 3 - FOUNDATIONS AND GEOLOGY

PART 1

LOCK AND FLOOD PROTECTION

FOUNDATIONS INVESTIGATION

GENERAL

B.3.1. This report addresses design assumptions and parameters for a new canal lock and associated Mississippi River levee protection. Initial construction will require a bypass channel next to the existing IHNC lock and pipeline relocation at possibly three sites along the channel. After relocation of the pipelines is complete, permanent flood protection can be built. Flood protection will basically follow the present line of levee protection and tie into the first module of the new lock.

B.3.2. The new lock will be pile founded. Excavation of the lock site will be done in the wet down to approximate elev. -61.0 N.G.V.D. Steel pipe piles (48-inch dia.) will be driven for the lock foundation, a grout stabilization slab poured onto which the lock modules will be positioned. Once all the lock modules are in position, sand backfill will be placed against the lock walls. Lock settlement will be discussed in later paragraphs.

FIELD EXPLORATION AND LABORATORY INVESTIGATION

Soil Borings

B.3.3. A total of twenty-four soil borings (18 undisturbed and 6 general type) were used in the foundation analyses for the lock, flood protection and channels. Logs of these borings are presented on plates B-67 thru B-85B. Boring locations are shown on the general plan presented on plates B-13, B-14 and B-15. All borings were made by the rotary drilling method. General samples were taken with a 1-7/8-inch ID core barrel sampler and a 1-3/8-inch ID (2-inch O.D.) split spoon utilizing a 140 pound hammer with a 30-inch drop. Undisturbed samples were taken with a 5-inch diameter steel tube piston-type sampler.

B.3.4. Soils Design Reach I, which encompassed all work riverward of the Claiborne Bridge, was based on 15 undisturbed and 6 general type borings taken between Jun 68 and Sep 83. These borings are located along the Industrial Canal corridor between the Mississippi river and midway between the St. Claude and Claiborne Bridges.

B.3.5. Soils Design Reach II was based on the three undisturbed borings (NC-1U, NC-2U and NC-3U) taken in the channel during May of 1992 through the foot print of the new lock. The first two (NC-1U and NC-2U) were bored to approximately 215 feet of depth while NC-3U was taken to 40 feet of depth.

Laboratory Tests

B.3.6. General. Visual classifications were made on all boring samples. Water content determinations were performed on all cohesive samples. Standard Penetration Resistance blow counts were recorded when sampling in granular strata. Unconfined compression (UC) shear tests and grain size analyses were made on selected samples of cohesive and granular soils, respectively. Unconsolidated-undrained (Q), consolidated-undrained (R) triaxial compression shear tests and consolidation (C) tests were performed on selected undisturbed samples and included Atterberg limits.

Shear Strength Tests

B.3.7. Unconfined Compression Tests. Unconfined compression (UC) tests were performed on representative specimens of cohesive soil from both undisturbed and undisturbed borings. UC test data is presented on the boring logs and shear strength plates.

B.3.8. Triaxial Tests. Unconsolidated-undrained (Q) triaxial tests, consolidated-undrained (R) triaxial test with pore pressure measurements and consolidated-drained (S) triaxial test were performed on selected undisturbed samples. Triaxial test data is presented on the undisturbed boring logs and shear strength plates.

B.3.9. Consolidation Tests. Consolidation (C) tests were obtained on selected samples and used to predict settlement of the new lock. The pressure-void ratio curves for the C-tests are presented in Annex 3. Values for the preconsolidation pressure, pc, were determined graphically as presented in NAVFAC DM-7.1 (MAY 92). It should be noted that before dredging of the channel for the present lock, natural ground was approximately elev. 2.0. In lieu of presenting consolidation test-time curves, values of the coefficient of consolidation, Cv, was determined by the logarithm of time method as presented in "Foundation Analysis and Design" by Bowles. The values for Cv can be found on the e-log p curves presented in Annex 3.

Design Parameters

B.3.10. Shear strength and wet density test results versus elevation for Design Reach I are plotted on plates B-86 and B-87. Reach I has a strength line for both the channel and the existing protection. The shear strength plots reflect the results of all UC tests, Q-tests and R-tests available from boring samples tested.

B.3.11. Reach II design parameters (shown on plate B-88) were derived from two sources of information, the "Lake Pontchartrain, LA & Vicinity, Chalmette Area Plan -DM #3, General Design" and the borings taken in the footprint of the lock. Parameters for the upper strata were derived from borings taken in connection with from the "Lake Pontchartrain, LA & Vicinity, Chalmette Area Plan - DM #3, General Design", while a composite of the footprint borings was employed for data below the channel bottom. Based on the facts that the Chalmette Area Plan starts just north of the Florida Ave. Bridge and that shear strength decreases as the distance from the river increases the use the upper strata parameters from this project is a conservative approach to analyzing the lock. Pleistocene deposits were assumed below elev. -65. The pleistocene parameters are derived from borings NC-1U, NC-2U, NC-3U, and deep boring 4-IU (see plates B-78 thru B-85).

EXISTING FACILITIES

B.3.12. The flood protection levees cross three (3) pipeline locations. The three pipeline corridors are adjacent to existing bridges, St. Claude (MRL sta. 619+22), Claiborne (B/L sta. 29+00) and Florida Ave. (B/L sta. 67+35). The relocated pipelines will be buried 10 feet below the bottom of the channel.

DESIGN REACHES

B.3.13. The job was divided into two design reaches based on boring data. The reaches are as follows:

Reach I - Mississippi River to Lock B/L Sta. 12+55

Reach II - Lock B/L Sta. 12+55 to Florida Ave. Bridge

SOIL CONDITIONS

B.3.14. Soil conditions in the vicinity of this project consist of the following:

a. Natural ground to approx. elev. -32.0 - natural levee deposits underlain by marsh and intradelta deposits. These deposits consist of very soft to medium clays and some silt lenses.

b. Elev. -32.0 to approx. elev. -70.0 - Interdistributary deposits underlain by prodelta deposits consisting of very soft to stiff clays, silt lenses and sand layers near the bottom.

c. Elev. -70.0 to approx. elev. -350.0 - Pleistocene deposits consisting of clays (stiff to very stiff), silts and sands.

STABILITY OF LEVEES

B.3.15. Using cross-sections representative of existing conditions along the proposed alignment, the slopes and berm distances for the proposed levee were designed for the (Q) construction case. The stability analyses are presented as plates B-89 thru B-95. A "Factor of Safety" (F.S.) of 1.3 is required for the levee stability. The pipeline crossing excavation trenches were also designed for a F.S. of 1.3.

B.3.16. A riverside flowline of elev. 17.6 and a low water elev. of -2.0 was used for design and analysis. Lake side flood elevations are 16.3 (Hurricane level) and 14.3 (SWL). All elevations used are N.G.V.D.

STRUCTURE EXCAVATION

B.3.17. The excavation requires deepening and reshaping the existing channel bottom with a hydraulic dredge. Since the excavation is in the existing channel and the structure will be in place prior to relocation of any flood protection, stability analysis addressing failure into the excavation are not presented.

CANTILEVER I-WALL

B.3.18. I-wall stability and required penetration were determined by the "Method of Planes". A "Factor of Safety" was applied to the soil parameters. For the friction angle, the F.S. was applied as follows:

$$\text{Ø}_d = \tan^{-1} \frac{\tan \text{Ø}_a}{\text{factor of safety}}$$

where Ø_a = available friction angle
Ø_d = developed friction angle

The developed friction angle was used in determining lateral earth pressure coefficients.

B.3.19. Using the resulting shear strengths, net horizontal water and earth pressure diagrams were determined for movement toward each side of the sheet pile. From the earth pressure diagrams, the summation of horizontal forces were equated to zero and the summation of overturning moments were determined for various tip penetrations. The depth of necessary penetration is the point of zero summation of moments.

B.3.20. The following design cases were analyzed for determining required penetration for the levee/I-walls.

No significant wave load on I-wall:

Q-Case

F.S. = 1.5 with static water at still water level (SWL)
F.S. = 1.0 with static water at (SWL) plus 2 feet
General: If the penetration to head ratio is less than 3:1, then increase it to 3:1.

Results of the cantilevered sheetpile stability run are presented on plate B-96.

SHEETPILE COFFERDAM

B.3.21. The temporary protection cells will be sand filled sheetpile cofferdams with an approximate diameter of 78 feet.

FAILURE MODE	F.S. (required)	F.S. (actual)
BURSTING	2.0	17.0
SHEAR FAILURE ALONG CENTERLINE	1.25 *	1700
HORIZONTAL SHEAR (CUMMING'S METHOD)	1.8	4.9
PULLOUT OF OUTBOARD SHEETING	1.5	850
PENETRATION OF INBOARD SHEETING	1.5 **	1.14
BEARING FAILURE OF THE FOUNDATION	1.5	1.5
SLIDING ON THE BASE	1.25 *	236
INTERLOCK TENSION (HOOP)	2.0	4.7

* for temporary construction
**for permanent construction

Design of the sheetpile cofferdam can be found in Annex 4.

PILE CURVES

B.3.22. The pile capacity curves for 14-inch square concrete piles, 14-inch H-piles and 48-inch steel pipe piles are presented on plates B-97 thru B-99. The curves presented indicate ultimate pile capacity.

SETTLEMENT

B.3.23. An analysis of estimated consolidation of the lock (pile group analysis) will be addressed in a future DM. Consolidation tests were performed on soil samples from borings NC-1U, NC-2U, NC-3U and deep boring 4-IU. For each consolidation test, the compression index, Cc vs. elevation was plotted (see plate B-100) to show the range of values at various depths. Also plotted was the preconsolidation pressure vs. elevation and is shown on plate B-101. Consolidation test results are included in Annex 3.

SEEPAGE CONTROL

B.3.24. A sheetpile cutoff extending into the pleistocene will surround the perimeter of the lock. A cutoff will also be placed across the back of each control house.

Project: Industrial Canal Lock Replacement
Subject: BEARING CAPACITY:

<u>Base Slab Underlain by Clay</u>

Foundation Soil Properties: c = 800 psf, ? = 107 pcf
Backfill Soil Properties: c = 0 psf, ? = 122 pcf

For a 180' x 1460' slab: (assume continuous)

$q_{ult} = 1.3\ cN_c + ?'D$

$= 1.3(800)(5.53) + 60(54)$

$q_{ult} = 5750 + 3250 = 9000$ psf

$q_a = q_{ult}/3 = 3000$ psf

FIGURE B-1

PART 2

GRAVING SITE AND FLOOD PROTECTION

FOUNDATIONS INVESTIGATION

GENERAL

B.3.25. This report addresses design assumptions and parameters for a graving site for a new canal lock and associated Mississippi River Gulf Outlet (MRGO) levee protection. Construction of a precast ship lock will require a graving site to float the precast lock sections into place at the new lock site. This report addresses the feasibility of a graving site Northwest of the Paris Road Bridge located in eastern New Orleans, LA. Flood protection along the MRGO will follow the present line of levee protection and tie into the graving site hurricane protection levees. Excavation of the graving site will be done in a dry condition to approximate elev. -27.5 N.G.V.D. Concrete grade beams spaced on 6-foot intervals supported by 74-foot long 12 x 12-inch concrete piles will provide a working foundation. Between the grade beams, a 1-foot sand base will be placed over the bottom of the excavation and overlain by a 4-inch unreinforced concrete stabilization slab to facilitate the fabrication of the lock modules. Flood side protection of the site will consist of a tie-in dike to elev. 7.0, a 45-foot dia. cofferdam cell and crushed stone closure at elev. 0.0 with sheetpile protection to elev. 8.0 to protect against high tides. The closure shall be removed after a lock module is completed and ready for transfer to another site, and the closure reinstalled to facilitate the construction of other modules.

FIELD EXPLORATION AND LABORATORY INVESTIGATION

Soil Borings

B.3.26. At the proposed new graving site, three undisturbed borings were made in March and April of 1983. These borings, taken for the MRGO hurricane protection levee (Citrus Back Levee), were Bor. 9-CBU, 10-CBU and 12-CBU. The borings were approximately 75 feet deep. A geologic profile of the borings are shown on Plate b-103C.

Design Parameters

B.3.27. Shear strength and wet densities for various soil stratification layers were based on a 1984 Soils Report of Lake Pontchartrain, LA and Vicinity, Citrus Back Levee, Third Lift, Sta. 0+00 to Sta. 398+00 C/L (specifically, Reach IIB). Boring locations are shown on the geologic profile Plate B-103C.

Soil Conditions

B.3.28. Soil conditions in the vicinity of this project consist of the following:

a. Natural ground is at approx. elev. 0.0 and natural levee deposits are underlain by swamp and interdistributary deposits. These deposits consist of soft to medium clays and silts.

b. Elev. -32.0 to approx. elev. -48.0 - Prodelta deposits consisting of medium clays.

c. Elev. -48.0 to approx. elev. -53.0 - Nearshore gulf deposits consisting of silty sands.

d. Elev. -53.0 and below - pleistocene deposits consisting primarily of clays (stiff) and lenses of silts down past elev. -70.0 (maximum boring depth).

STABILITY OF LEVEES

B.3.29. Using existing conditions along the proposed alignment, the slopes and berm distances for the proposed levee were designed for the (Q) construction case. A "Factor of Safety" (F.S.) of 1.3 is required for levee stability. A channelside flowline of elev. 6.0 and a low water elev. of 0.0 was used for design and analysis. All elevations used are N.G.V.D. Stability analyses are presented as Plates B-102A and B-103A. Plate B-103A presents a stability analysis of the hurricane protection levee relevant to the dewatered excavation. Plate B-102A presents an analysis of the sheetpile/closure protection for a dewatered excavation condition.

EXCAVATION

B.3.30. The excavation is designed for construction in a dry condition. Material excavated will be used to construct the hurricane levee protection surrounding the graving site and the tie-in dikes between the channel closure and the main levees. Approximately 177,000 cubic yards of material is to be excavated from the graving site initially. The initial closure excavation will yield an additional 48,000 cy of soil. Approximately 90,000 cy of the excavated material will be used for construction of the new hurricane levee surrounding the site. An additional 50,000 cy of crushed stone will be required to rebuild the closure after the first module is floated out. As additional modules are floated off site, an additional 10,000 cy of crushed stone will be required to rebuild the closure.

DEWATERING

B.3.31. The excavation will require a well and/or wellpoint dewatering system to keep the slopes and base dry. Piezometric levels should be lowered a minimum of 5 feet below the slopes and the bottom of the excavation. Piezometers will be required for monitoring water levels below the slopes and the excavation base.

CANTILEVER I-WALL

B.3.32. I-wall stability for the closure and tie-in levees and required penetration were determined by the "Method of Planes". A "Factor of Safety" was applied to the soil parameters. For the friction angle, the F.S. was applied as follows:

$$Ø_d = \tan^{-1} \frac{\tan Ø_a}{\text{factor of safety}}$$

where $Ø_a$ = available friction angle
$Ø_d$ = developed friction angle

The developed friction angle was used in determining lateral earth pressure coefficients.

B.3.33. Using the resulting shear strengths, net horizontal water and earth pressure diagrams were determined for movement toward each side of the sheet pile. From the earth pressure diagrams, the summation of horizontal forces were equated to zero and the summation of overturning moments were determined for various tip penetrations. The depth of necessary penetration is the point of zero summation of moments.

B.3.34. The following design cases were analyzed for determining required penetration for the levee/I-walls.

No significant wave load on I-wall:

Q-Case

F.S. = 1.5 with static water at still water level (SWL)
F.S. = 1.0 with static water at (SWL) plus 2 feet
General: If the penetration to head ratio is less than 3:1, then increase it to 3:1.

B.3.35. An I-wall analyses for the crushed stone channel closure is presented on Plate B-102B. Final penetration was ultimately based on seepage cutoff considerations. Soil pressures are based on water to elev. 7 and a Factor-of-Safety of 1.5. I-wall sheetpile (PSA-28) will be utilized for the closure and tie-in levees.

COFFERDAM CELLS

B.3.36. At each end of the sand or stone closure a 45-foot dia. sheetpile cofferdam cell (PSA-28) will tie together the dike and closure. Cell dimensions are shown on Plate B-102C. The cell was analyzed for the following modes of failure.

FAILURE MODE	F.S. (required)	F.S. (actual)
BURSTING	2.0	17.0
SHEAR FAILURE ALONG CENTERLINE	1.25 *	2.7
HORIZONTAL SHEAR (CUMMING'S METHOD)	1.8	4.9
PULLOUT OF OUTBOARD SHEETING	1.5	7.4
PENETRATION OF INBOARD SHEETING	1.5	12.0
BEARING FAILURE OF THE FOUNDATION	1.5	2.0
SLIDING ON THE BASE	1.25 *	1.6

* for temporary construction

PILE CURVES

B.3.37. The pile capacity for a 12-inch square concrete pile is presented on Plate B-99A. The pile will have a tip elevation of -100 N.G.V.D. and will support the grade beams that the lock modules will be constructed upon.

GRADE BEAMS AND STABILIZATION PAD

B.3.38. Concrete grade beams (3' deep x 2' wide x 220' long) will facilitate fabrication of the lock modules. The grade beams will be set on 12-inch piles and run the length of the excavation base. Between the grade beams, a 4-inch stabilization slab underlain by 1-foot of sand will be poured to provide a good working base.

SETTLEMENT

B.3.39. The new hurricane levee surrounding the graving site will experience some consolidation during and after construction and it will therefore be necessary to retain an elevation close to 15.0 as settlement occurs. The graving site grade beams will be pile founded and, depending on the weight of the modules constructed, should not experience significant settlement.

SEEPAGE CONTROL

B.3.40. A sheetpile seepage cutoff will extend into the pleistocene along the length of the channel closure. The tip of the sheetpile will be driven to approximate elevation -55 to substantially slow down seepage from the channel.

PART 3

GEOLOGY

PHYSIOGRAPHY

B.3.41. The project area is located within the Central Gulf Coastal Plain. Specifically, the project is on the eastern flank of the Mississippi River Deltaic Plain. Dominant physiographic features include natural levees, inland swamp, and marshes; however, urban development has masked these features in the immediate project area. This is an area of low relief ranging from a maximum of +6 feet NGVD on the landside slopes of natural levee near the river to a minimum of -2 feet NGVD in the drained areas and marshes.

GENERAL GEOLOGY

B.3.42. 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. Approximately 4,500 years ago the first Recent deltaic sediments were carried into the project area when the Mississippi River was depositing the St. Bernard Delta sequence. 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 begin building the

Lafourche Delta sequence. The project area was not subjected to a heavy influx of sediments again until approximately 1000 years ago when the Mississippi River shifted eastward to the present Plaquemines Delta sequence. Construction of levees and other development has eliminated any further deposition of fluvial/deltaic sediments in the project area.

SUBSIDENCE

B.3.43. Regional subsidence and geosynclinal downwarping have been occurring since the end of the Pleistocene epoch. The long-term rate of subsidence in the project area is approximately 0.48 feet per century. In addition, man induced subsidence of the ground surface has occurred in reclaimed marsh and swampland due to the shrinking of highly organic soils after drainage.

INVESTIGATIONS PERFORMED

B.3.44. Previous geologic profiles and reports on the project, as well as other reports, maps and publications, were used for the interpretation of physiography and the surface and subsurface conditions of the area. A geologic profile of borings in the lock vicinity are shown on Plate B-103B.

SUBSURFACE CONDITIONS

B.3.45. The project area contains natural levee, inland swamp, and marsh deposits. The natural levee is located at the southern end of the project area near the river. These deposits consist of interbedded soft to stiff clays and average 10 to 15 feet thick. Natural levee deposits are bordered by inland swamp and marsh deposits which may underlie part of the natural levee. Swamp and marsh deposits consist of interbedded very soft clays with organic matter and occasional lenses and layers of peat and average 8 to 10 feet thick. Underlying the natural levee, swamp, and marsh are interdistributary deposits which consist of soft to very soft clays with occasional lenses and layers of silt and sand, and are approximately 25 feet thick. A layer of sand with shells and shell fragments occurs below the interdistributary deposits. This deposit averages 10 feet thick and is probably a relict beach sand. Below the sand layer a thin layer of prodelta clays may be present or the beach sand may lie directly upon Pleistocene deposits. The prodelta deposits consist of interbedded medium to stiff clays and average 6 feet thick. Pleistocene underlies the prodelta and beach sand, extends to an unknown depth, and consists of interbedded stiff to very stiff oxidized clays, silt, silty sand, and sand. The contact with the Pleistocene surface averages 65 feet in depth.

SOILS

B.3.346. Soils in the area have been classified as being level, poorly drained and somewhat poorly drained having a clayey or loamy surface layer and a clayey subsoil or are loamy throughout. (Trahan, Larry J. et al, 1989)

GROUND WATER

B.3.47. There are four principle aquifers in the project area. The shallow Holocene aquifers consist of point bars, distributary channel sands, and beach sands and are formed by river migrations and delta building. These aquifers are generally of a discontinuous limited extent. The 200 foot sand is a zone of discontinuous layers and lenses and generally contains salt water in the project area. The 700 foot sand is continuous over most of Orleans Parish and contains fresh and salt water. The 1200 foot sand is continuous but is slightly saline to brine with less than 10000 ppm dissolved solids in the project area.

EARTHQUAKE ACTIVITY

B.3.48. Although some faults have been mapped in this region, no significant seismic activity is expected in this area. (Krinitzsky, 1950)

MINERAL RESOURCES

B.3.49. Oil and gas production is not found in the immediate vicinity of the project. However, future exploration and production of these natural resources will not be adversely affected by the project.

CONCLUSIONS

B.3.50. Because of the low shear strength and compressibility of some of the sediments in the study area, stability and settlement are major concerns. Also, because of urban development and drainage in much of the study area, settlement in this area is higher than the natural rate of subsidence. The existence of a large sand deposit in the subsurface is conducive to seepage and uplift problems.

REFERENCES

B.3.51.

Fisk, H. N., 1944, "Geological Investigation of the Alluvial Valley of the Lower Mississippi River", conducted for Mississippi River Commission.

Krinitzsky, E. L. et al, 1950, "Geologic Investigation of Faulting in the Lower Mississippi Valley", Waterways Experiment Station, Technical Memorandum No. 3-331.

Rollo J. R., 1966, "Ground-Water Resources of the Greater New Orleans Area, Louisiana", Louisiana Department of Conservation, Geological Survey and Louisiana Department of Public Works in cooperation with the United States Geological Survey, Water Resources Bulletin No. 9.

Trahan, Larry J. et al, 1989, "Soil Survey of Orleans Parish, Louisiana", United States Department of Agriculture, Soil Conservation Service in cooperation with Louisiana Agricultural Experiment Station and Louisiana Soil and Water Conservation Committee.