US Army Corps
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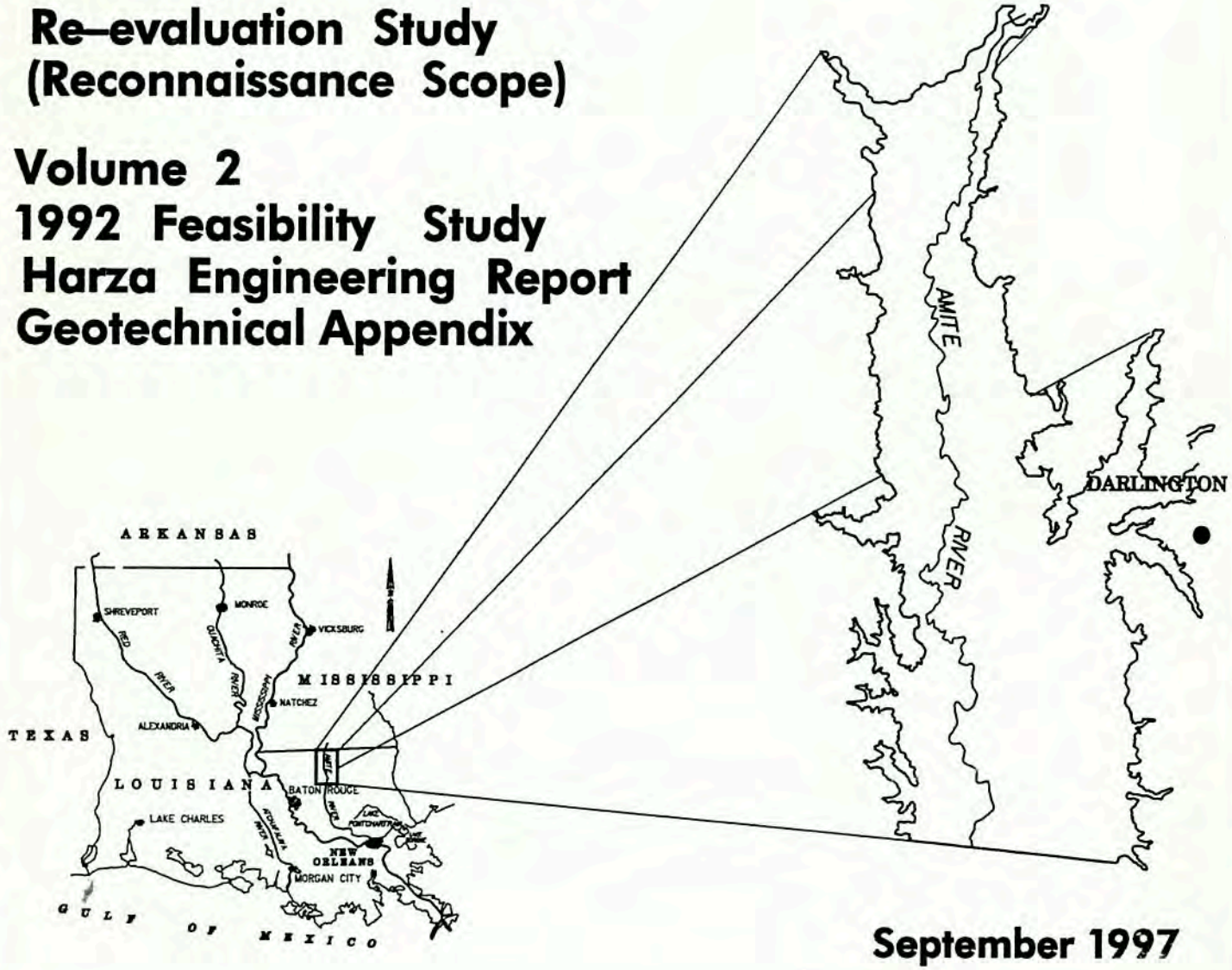
Louisiana Department of
Transportation and Development

Amite River and Tributaries, Louisiana

Darlington Reservoir

Re-evaluation Study (Reconnaissance Scope)

Volume 2
1992 Feasibility Study
Harza Engineering Report
Geotechnical Appendix



September 1997

If you have any questions or require additional information, please contact Ms. Julie Z. LeBlanc, Study Manager, U.S. Army Corps of Engineers, New Orleans District, P. O. Box 60267, New Orleans, LA 70160, telephone number (504) 862-1597.

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MAIN REPORT

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APPENDIX A

AMITE RIVER AND TRIBUTARIES, DARLINGTON RESERVOIR FEASIBILITY STUDY, SEPTEMBER 1992

SYLLABUS

The purpose of the overall Amite River and Tributaries study is to investigate the feasibility of providing flood protection for the residents in the Amite River Basin. This report documents final results of the feasibility phase studies for the mainstem of the Amite River. The goal of the study was to develop solutions to reduce flood damages along the mainstem of the Amite River and lower tributaries streams. The study area encompasses about 2,200 square miles in southeastern Louisiana and southwestern Mississippi. The study area is largely forest and agricultural. These land uses make up 72 percent of the land use. Urban and built-up land comprise 12 percent of the land use. Major urban centers in the study area include the Cities of Baton Rouge, Baker, Zachary, Gonzales, Sorrento, and Denham Springs, Louisiana. The City of Baton Rouge is the State Capitol. In 1991, the study area population was about 544,000. This basin is one of the three most populated basins in the State of Louisiana. Four major floods have occurred in the basin between 1973 and 1983. Flooding within the basin originates from excessive rainfall resulting in headwater and backwater overflow of the Amite River and tributary streams. The maximum flood of record occurred in 1983 and caused approximately $172.0 million in damages.

Numerous structural and nonstructural measures were considered to reduce flood damages along the mainstem of the Amite River and lower tributary streams. In this evaluation it has been determined that construction of a dam and reservoir on the upper Amite River is not a feasible solution to downstream flooding problems. The most cost-effective dam and reservoir plan was determined to have a benefit cost ratio of 0.60. Previous reconnaissance studies have revealed that channelization of the Amite River may be cost-effective but severe environmental mitigation problems and public opposition also make this plan unacceptable. Other reconnaissance studies have indicated that nonstructural measures may be an effective solution to flooding problems in the lower Amite River Basins and that these options should be further evaluated.

The requirements of Section 404(r) of Public Law 92-500, as amended, have been met.

DARLINGTON RESERVOIR FEASIBILITY STUDY

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DARLINGTON RESERVOIR FEASIBILITY STUDY

STUDY AUTHORITY

This study is part of the Amite River and Tributaries (ART) Study being conducted in response to a resolution of the committee on Public Works of the United States Senate. The resolution, sponsored by the late Senator Allen J. Ellender and Senator Russell B. Long of Louisiana, was adopted on April 14, 1967, and reads as follows:

> "RESOLVED BY THE COMMITTEE ON PUBLIC WORKS OF THE UNITED STATES SENATE, That the Board of Engineers for Rivers and Harbors, created under Section 3 of the River and Harbor Act approved June 13, 1902, be, and is hereby requested to review the report of the chief of Engineers on Amite River and Tributaries, Louisiana, published as House Document Numbered 419, Eighty-fourth Congress, and other pertinent reports, with a view to determining whether the existing project should be modified in any way at this time with particular reference to additional improvements for flood control and related purposes on Amite River, Bayou Manchac, and Comite River and their tributaries."

STUDY PURPOSE AND SCOPE

The purpose of the ART study is to investigate the feasibility of providing flood protection for the residents in the Amite River Basin. This study is being conducted in two phases: a reconnaissance phase and a feasibility phase. The reconnaissance phase was initiated in September 1983 and completed in February 1985 with the signing of a feasibility cost-sharing agreement (FCSA). The cost-sharing partner is the Louisiana Department of Transportation and Development, Office of Public Works (D0TD). The feasibility phase was initiated in April 1985. In January 1986, notification was received from the Secretary of the Army Office that cost-sharing on Corps feasibility studies would be implemented on January 15, 1986. Specific terms of the Amite River and Tributaries study cost-sharing agree ment stipulates that cost would be shared 50-50, commencing 60 days after the decision to proceed with cost-sharing. Therefore, all costs incurred

after March 15, 1986 were cost-shared on this study. In February 1990 the ART feasibility cost sharing agreement was modified to include the investigation of the Darlington Dam and Reservoir. This feasibility cost-sharing agreement is contained in Appendix A.

The Amite River Basin is shown on Plate 1. The basin encompasses about 2,200 square miles in southeastern Louisiana and southwestern Mississippi that is drained by the Amite River and tributaries. It includes portions of East Baton Rouge, Ascension, Livingston, East Feliciana, St. Helena, Iberville, St. James, and St. John the Baptist parishes in Louisiana and Wilkinson, Lincoln, Franklin and Amite counties in Mississippi. The 170-mile-long Amite River and its right bank tributary, the Comite River, rise in southwestern Mississippi and flow generally southward to their confluence east of Baton Rouge in the vicinity of Denham Springs. From that point, the Amite River continues in a southerly direction to a juncture with Bayou Manchac at about mile 36 and then southeasterly and easterly to Lake Maurepas. Bayou Manchac, a right bank tributary of the Amite River and a former distributary of the Mississippi River at Mile 215 above the Head of Passes (AHP), extends about 17 miles eastward between the Mississippi River and Amite River at Mile 36. Major urban centers in the basin include Baton Rouge, Baker, Zachary, Gonzales, Sorrento, and Denham Springs, Louisiana.

Due to the complex nature of the flood problem, feasibility phase studies were divided along hydrological and political boundaries to advance the study process. The feasibility study of the Comite River Basin was completed in January 1991. In addition to this investigation of the Darlington Dam and Reservoir other studies are being conducted for the following areas:

East Baton Rouge Parish
Livingston Parish
Ascension Parish

This report is being provided to the local sponsor to document study results. The goal of the study was to develop solutions to reduce flood damages along the Amite River and lower tributaries streams. This report addresses the causes and impacts of flooding and evaluates measures

to alleviate flood damages. This report documents the results of field investigations, hydrologic and hydraulic studies, economic and environmental studies, Federal, state, and local coordination, and public involvement activities accomplished during the studies. Studies were made in the detail necessary for the comparison of alternative plans, the identification of the NED plan, and the development of recommendations for implementation of a recommended plan. The study also addresses the protection of fish and wildlife habitat and the provision of regional recreational opportunities.

PRIOR STUDIES, REPORTS, AND EXISTING WATER PROJECTS

The pertinent studies and reports on water resources development in or near the study area by the U. S. Army Corps of Engineers and other Federal, state, and local agencies are described in the following paragraphs.

A preliminary evaluation report was prepared by the Corps of Engineers in May 1972. The evaluation was conducted under the authorizing resolution for this study. Four reservoir plans, two plans to divert flood waters from the Amite River Basin to the Mississippi River, and four channel improvement plans were investigated. All 10 plans were determined to be economically infeasible. This study was placed on the inactive status in February 1974.

A second reconnaissance study of the Amite River and Tributaries was initiated and subsequently completed by the Corps of Engineers in December 1984. In this initial evaluation report, a number of alternative solutions were developed to mitigate flood damages in the basin. A number of economically justified and environmentally acceptable plans were identified. The findings of this report provided the basis for the authorization of this feasibility study.

Section 206 of the 1960 Flood Control Act (PL 86-646), as amended by the 1960 and 1970 Flood Control acts, the Water Resources Development Act of 1974 and Executive Order 11296, August 10, 1966, authorizes the Corps of Engineers to establish and carry out a Floodplain Management Service Program. The objective of the program is comprehensive

flood damage prevention planning that encourages wise use of the flood plain at all levels of government. Under the program, the Corps prepared six floodplain information reports. They are:

NAME OF STREAM	DATE PREPARED
Bayou Fountain	June 1971
Ward Creek and Tributaries	October 1972
Claycut Bayou, Jones Creek and Tributaries	September 1974
Hurricane Creek, Monte Sano Bayou and Tributaries 1	November 1976
Cypress Bayou and Tributaries	November 1976
New River, Bayou Francois, and Black Bayou	March 1971

The Corps of Engineers prepares flood insurance studies to map eligible communities by risk zones and to determine insurance rates. The studies are made under the provisions of the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973. The program is administered by the Federal Insurance Administration of the Federal Emergency Management Agency. The flood insurance studies prepared for cities and unincorporated areas in the Amite River Basin as of September 1991 are shown in Table 1.

TABLE 1

FLOOD INSURANCE STUDIES - AMITE RIVER BASIN

URBAN PLACE	PARISH	COMPLETION DATE
Unincorporated Areas, Ascension Parish	Ascension	Sep 2, 1981
City of Baker	East Baton Rouge	Oct 19, 1973
Town of Clinton	East Feliciana	Dec 4, 1979
City of Denham Springs	Livingston	Oct 15, 1981
East Baton Rouge Parish	East Baton Rouge	Jul 2, 1979
City of Gonzales	Ascension	Aug 16, 1982
Town of Gramercy	St. James	Jan 24, 1978
Town of Greensburg	St. Helena	April 1, 1980
Unincorporated Areas, Iberville Parish	Iberville	Jun 1, 1978
Town of Jackson	East Feliciana	Jun 4, 1980
Unincorporated Areas, Livingston Parish	Livingston	Sep 30, 1988
Town of Livingston	Livingston	April 15, 1979
Town of Lutcher	St. James	April 24, 1979
Town of Sorrento	Ascension	Jun 1, 1978
St. John the Baptist Parish	St John the Baptist	Jul 1, 1980
Town of Walker	Livingston	Feb 17, 1982
City of Zachary	East Baton Rouge	Sep 15, 1977
Baton Rouge & Vicinity	East Baton Rouge	Nov 15, 1985
Unincorporated Areas, Livingston Parish	Livingston	Sep 1985
Unincorporated Areas, St Helena Parish	St. Helena	Sep 1991
Town of Port Vincent	Livingston	Aug 16, 1988
Town of French Settlement	Livingston	Oct 15, 1985
Village of Killian	Livingston	Aug 1, 1987

The Department of Transportation and Development contracted with Brown and Butler Inc., to investigate the feasibility of a reservoir near Darlington, Louisiana. The proposed reservoir would have a maximum water surface area of about 19,500 acres and a normal water surface area or recreation pool of about 15,000 acres. The study,

completed in March 1984, determined that the reservoir was economically feasible and recommended that the Amite River Basin Drainage and Conservation Commission investigate methods to fund the project.

The Department of Transportation and Development contracted with Brown and Butler, Inc. in May 1985 to investigate the hydraulic and hydrologic parameters in more detail than was in the previous study completed in March 1984. In the study, topographic surveys were taken of the Amite River valley from Interstate Highway 12, to the Louisiana-Mississippi state line, a detailed hydrologic and hydraulic model was developed, and several reservoir designs were analyzed. The study was completed in August 1986. It concluded that the hydrologic and hydraulic analyses conducted as part of this study confirms the related findings of the previous study completed in March.

The Department of Transportation and Development applied to the U.S. Corps of Engineers for a Section 404 permit in April 1985 to construct the Darlington Reservoir. DOTD contracted with Espey Huston and Associates, Inc., in December 1987 to develop the necessary engineering and environmen tal information for the Corps of Engineers to prepare an environmental impact statement. The study was completed in January 1990. In early 1990, however, the State officially withdrew this permit application and requested Federal participation in this project which in turn initiated this (Federal) feasibility study.

The Corps of Engineers completed a feasibility study of flood control measures in the Comite River Basin in September 1990. Numerous structural and nonstructural measures were considered to reduce flood damages along the Comite River and lower tributary streams and to a lesser extent along the Amite River. The recommended plan consists of a 12-mile diversion channel from the Comite River to the Mississippi River. Major features of the plan include a Diversion Structure, a Comite River Stage Control Structure and a levee, a Channel Stage Control Structure, and an 8-mile levee along the southern bank of the diversion channel, and recreation facilities. Detailed design of the project is currently in progress and is expected to be completed in 1995. This project will be cost-shared with the State of Louisiana.

The Louisiana state legislature in their 1982 regular session created the Statewide Flood Control Program by Act 351. The purpose of the program is to provide assistance to the parish and local governments in reducing flood problems. Guidelines and procedures for participating in the program were completed and distributed by the flood control project evaluation committee in March 1983. Through 1988, one drainage project has been funded in the Amite River Basin. East Baton Rouge parish proposed to enlarge Beaver Bayou. Land acquisition for the project has been completed. Construction began in August 1988 and was expected to be completed in 200 working days. However, 1989 was a relatively wet year that slowed construction. Construction was completed in September 1990. The work cost about $700,000. About 70 percent of the cost would be paid by the State-Wide Flood Control Program. East Baton Rouge Parish would provide 30 percent of the cost including lands, easements, and rights-of-way.

In July 1990, Wilbur Smith Associates, Evans-Graves Engineers, and Chenevert-Soderberg Associates prepared a Comprehensive Land Use and Development Plan (known as the Horizon Plan) for the City of Baton Rouge and East Baton Rouge Parish. This study addresses current and future drainage and flood prevention needs of East Baton Rouge Parish and recommends parish-wide and specific watershed solutions to flooding problems. Recommendations include the following:

- implement a parish-wide drainage maintenance program
- implement local drainage improvements and support major drainage projects
- evaluate the potential of flood detention facilities within the Amite River Basin
- develop, maintain, and enforce a Master Drainage Plan and Drainage Criteria
- develop hydrologic and hydraulic modeling capabilities to predict drainage impacts of new development
- implement a public awareness program
- develop a long range plan for implementation and funding of a program to include other local agencies, the State and Federal governments

- utilize subdivision requirements to secure drainage rights-of-way
- implement a parish-wide program to install and maintain survey benchmarks on a single datum.

On October 1, 1990 the Governor's Interagency Task Force on Flood Prevention and Mitigation completed an investigation to control and mitigate floods in the Amite River Basin. Flood control measures examined included floodplain management, stormwater retention, structural elevation and relocation, voluntary privately-owned retention ponds, zoning restrictions, habitat and ground cover preservation and effective drainage improvements. Numerous short and long term recommendations were made including the following:

- implementation of floodplain regulatory standards
- implementation of a state-wide flood disclosure law
- a program for voluntary structure floodproofing, elevation relocation or removal
- institution of a regional outreach awareness program
- improve existing flood forecasting and information
- implementation of a regional channel maintenance program
- assist the Federal Emergency Management Agency with their efforts
- assist local parishes and townships with the implementation of flood control projects
- assist with the implementation of the proposed Comite Diversion Canal
- pursue implementation of a full-size dry reservoir on the upper Amite River
- develop new, logical, cost-effective and environmentally acceptable alternatives.

The Ascension Parish Police Jury completed development of an Ascension Parish Drainage Plan in July 1981. The drainage plan consists of channel enlargements on Black Bayou, Bayou Francois, and New River Canal, levees around Sorrento with a pumping station, two diversion gates on New River to prevent cross flow into New River from Bayou Francois and Bayou Narcisse, and a levee system with floodgates and a pump station. A Section 404 permit was

issued by the U.S. Corps of Engineers in February 1984. Construction of the plan is underway. Work is progressing as funds from a tax referendum are made available. No construction completion date has been established.

Under Section 22 of the Water Resources Development Act of 1974, as amended, the Corps of Engineers is currently conducting an initial evaluation investigation (reconnaissance study) of non-structural flood control measures for the Amite River Basin. Initial findings indicate that non-structural measures may be feasible. Further detailed study at the feasibility level will likely be recommended. The reconnaissance report document will be completed by October 1992.

The existing Federal projects in or near the study area and their authorized features are:

The Amite River and Bayou Manchac navigation project, authorized 1927 and completed in 1928, provides for a 7- by 60-foot channel in the Amite River from its mouth at Lake Maurepas to mile 31 at Port Vincent (about 5 miles downstream of Bayou Manchac) and the clearing and snagging of the Amite River and Bayou Manchac from Port Vincent to the Kansas City Southern Railroad crossing at about mile 8.5 of Bayou Manchac. The project is shown in Plate 2.

The Amite River and Tributaries flood control project, authorized in 1955 and completed in 1964, provides for enlargement of the Comite River from Cypress Bayou (mile 10) to the mouth, clearing and snagging the Amite River from the Comite River (mile 54) to Bayou Manchac (mile 35.7), and enlarging and realigning the Amite River from Bayou Manchac to mile 25.3, a riprapped control weir on the south side of the Amite River at mile 25.3 and a diversion channel from the weir to Blind River at mile 4.8, snagging and clearing Blind River from mile 4.8 to Lake Maurepas, and snagging and clearing Bayou Manchac from the Amite River to Ward Creek (mile 8.4). The project is shown on Plate 3. A small navigation channel was provided around the weir between the Amite River and the diversion channel. Snagging and clearing Blind River from mile 4.8 to Lake Maurepas, although authorized, was found to be unnecessary after initial investigations. The Louisiana Department of Public Works enlarged the Comite River to dimensions

considerably in excess of those to be provided under the project and extended the enlargement about 2 miles farther upstream.

Improvements by others in or near the study area and their features are:

The Louisiana Department of Public Works elected to construct the Federal enlargement of the Comite River as "equivalent work" in lieu of a cash contribution toward the Federal improvement of the Amite and Comite Rivers. That agency elected to excavate a much larger channel than provided in the Federal plan. The bottom width was increased from 60 to 90 feet the depth was increased about 4 feet in the lower river and about 10 feet between miles 8 and 10, and the enlargement was extended about 1.1 miles upstream of Cypress Bayou, the head of the Federal project.

The Louisiana Department of Public Works in 1967, under the State-Parish Drainage Plan, enlarged White Bayou for a distance of about 8 miles upstream of Louisiana Highway 64. The lower 4.5 miles was enlarged to a 30-foot bottom width at a depth of about 14 feet. A smaller channel was provided in the upstream area.

The Louisiana Department of Public Works in about 1956, under the State-Parish Drainage Plan, enlarged White Bayou from Louisiana State Highway 64 about 2.4 miles southward and excavated channel (Baker Canal) generally southwestward through the town of Baker to Bayou Baton Rouge, a tributary of the Mississippi River.

PLAN FORMULATION

ASSESSMENT OF WATER AND RELATED LAND RSOURCES PROBLEMS AND OPPORTUNITIES

The planning process for the Darlington Dam and Reservoir was conducted in an organized and systematic manner to ensure that all reasonable alternative plans were considered. The process was conducted in accordance with U. S. Water Resources Council "Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies" Plan Formulation for the

Darlington Dam and Reservoir was an iterative and dynamic process. Plan formulation is directed at achieving the National Economic Development (NED) objective consistent with protecting the nation's environment in accord with national environmental statutes, applicable executive orders, and other Federal planning requirements, and is respon sive to state and local concerns. The NED objective is achieved by increasing the value of the national output of goods and services and reasonably maximizing net economic benefits. Benefits are maximized while giving due consideration to environmental quality, regional development, and social concerns.

During the process, historical trends and existing conditions were used as a base for forecasting future conditions. In an assessment of the nature and extent of changing conditions, problems and needs were identified and specific planning objectives defined. Opportunities in the form of management measures that address the objectives were evaluated. The most feasible measures were incorporated into an array of plans. The plans were then assessed and evaluated in terms of their engineering feasibility and performance as well as their adverse and beneficial effects on the NED objective. The effects on environmental quality were also evaluated. Finally, the plans were compared and a trade-off analysis performed to select the plan that best addresses the NED objective and to provide the rationale for the tentatively selected plan.

EXISTING CONDITIONS

Climate

The climate of the area is humid subtropical, but is subject to significant polar influences during winter, as cold air masses periodically move southward over the area displacing warm moist air. Prevailing southerly winds create a strong maritime character. This movement from the Gulf of Mexico helps to decrease the range between hot and cold temperatures and provides a source of abundant moisture and rainfall.

The average annual normal temperature for the basin as published in the National Climate Center's "Climatological Data," is 67.6 degrees Fahrenheit (°F) for the period 1951-1980. Average monthly normals vary from 51.2°F in January to 82.0°F in July. An overall maximum temperature of 110°F was recorded at Baton Rouge during August 1909 and an extreme low temperature of 1°F is on record at Hammond during February 1899. The mean monthly and annual normal temperatures (1951-1980) for the study area are given in Table 2.

The average annual normal rainfall based on National Climatic Center records at fourteen stations is 60.4 inches covering the period 1951-1980. The wettest month is July with a monthly normal average of 6.46 inches, while October is the driest with 2.78 inches. Several stations have recorded months with rainfall greater than 20 inches. There have also been several months with no measurable rain. The greatest 24-hour rainfall was 14.43 inches on 2 August 1983, recorded at the Baton Rouge Sherwood gage. Snow is a rarity in the study area with the last measurable snowfall occurring February 1988. Table 3 gives the monthly and annual normals for the study area.

Wind records (1973-1987) taken at Baton Rouge's Ryan Airport indicate the prevailing wind flow is from a southerly direction during most of the year and averages 7.2 miles per hour (mph). Average monthly wind speeds vary from 5.2 mph in August to 8.9 mph in March. A maximum wind speed of 58 mph was observed in September 1965 during Hurricane Betsy. Table 4 shows the monthly and annual wind speeds with the resultant directions in Table 5, over the period 1973-1987.

TABLE 2

MEAN MONTHLY AND ANNUAL TEMPERATURE (F)

30-Year Normals (1951-1980)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
BATON ROUGE	50.8	53.6	60.5	68.4	74.8	80.3	82.1	81.4	77.9	68.2	58.7	53.1	67.5
CARVILLE	51.7	54.2	60.8	68.2	74.8	80.3	82.1	81.7	78.4	69.1	60.0	54.0	67.9
HAMMOND	51.0	53.6	60.2	67.6	73.6	79.3	81.4	80.9	77.4	67.3	58.3	52.7	66.9
RESERVE	51.1	53.7	60.3	68.3	74.9	80.4	82.2	81.8	78.4	68.9	59.7	53.5	67.8
AVERAGE	51.2	53.8	60.5	68.1	74.5	80.1	82.0	81.5	78.0	68.4	59.2	53.3	67.6

EXTREME MINIMUM 1°F, IN FEBRUARY 1899.

EXTREME MAXIMUM 110°F, IN AUGUST 1909.

SOURCE: National Climatic Center

TABLE 3

MONTHLY AND ANNUAL PRECIPITATION (inches)

30-Year Normals (1951-1980)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
LOUISIANA STATIONS													
AMITE	5.44	5.90	5.66	6.46	5.48	4.51	7.33	4.86	5.18	2.68	4.58	5.82	63.90
BATON ROUGE AP	4.58	4.97	4.59	5.59	4.82	3.11	7.07	5.05	4.42	2.63	3.95	4.99	55.77
CARVILLE	4.81	4.87	4.45	5.00	5.11	4.15	6.95	5.82	5.10	2.65	4.12	5.60	58.63
HAMMOND	4.81	5.65	5.67	6.15	5.68	3.82	7.53	4.82	5.74	2.94	4.44	5.99	63.24
KENTWOOD	5.39	5.18	5.78	6.17	6.14	4.49	7.15	5.03	4.74	2.79	4.78	5.91	63.55
OAKNOLIA	5.33	5.39	5.33	5.95	5.64	3.80	6.20	4.77	4.24	3.02	4.27	5.44	59.38
PINE GROVE	5.88	5.86	6.08	6.52	6.11	4.26	7.78	5.57	4.94	3.20	4.83	6.46	67.49
RESERVE	5.13	5.58	5.15	4.51	5.29	4.31	6.50	5.70	5.95	3.01	4.07	5.62	60.82
MISSISSIPPI STATIONS													
BOOKHAVAN	5.16	5.02	6.42	5.79	5.06	4.12	5.30	4.36	3.28	2.73	4.46	6.24	57.94
LIBERTY	5.04	4.81	5.93	5.38	5.31	4.53	5.85	4.43	4.29	2.48	4.38	6.04	58.47
MONTICELLO	5.39	5.04	6.42	5.76	5.23	4.02	4.71	3.86	4.06	2.68	4.13	5.94	57.24
TYLERTOWN	5.00	5.23	5.48	5.53	5.56	4.17	5.78	4.45	3.60	2.67	4.16	5.93	57.56
WOODVILLE	5.41	5.55	6.48	5.80	5.71	4.62	5.83	4.15	4.19	2.67	4.81	6.56	61.78
AVERAGE	5.18	5.31	5.65	5.74	5.47	4.15	6.46	4.84	4.59	2.78	4.38	5.89	60.44

SOURCE: National Climatic Center

TABLE 4

AVERAGE MONTHLY AND ANNUAL WIND SPEEDS

1973-1987 (mph)

BATON ROUGE AT RYAN AIRPORT

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
1973	9.3	9.0	10.2	9.9	9.6	6.1	6.7	5.9	7.5	5.9	8.5	9.8	8.2
1974	8.6	10.0	9.0	10.1	8.8	8.3	6.4	6.3	9.1	7.4	8.8	8.3	8.4
1975	8.4	9.7	11.7	10.2	7.6	6.9	5.3	5.0	6.1	6.5	7.8	7.6	7.7
1976	9.3	9.0	9.7	7.4	7.7	6.2	5.5	5.8	5.7	7.5	7.6	8.2	7.5
1977	9.2	9.3	9.6	7.9	7.2	6.5	4.6	5.8	6.3	6.8	7.9	9.2	7.5
1978	9.9	9.4	8.5	8.4	7.7	6.0	6.1	5.5	5.8	5.0	4.8	7.3	7.0
1979	8.4	8.0	8.8	7.8	6.7	5.9	6.6	3.9	6.4	5.9	6.5	6.0	6.7
1980	7.7	8.4	9.6	7.6	6.1	6.2	4.3	4.4	5.0	5.5	5.3	5.4	6.3
1981	5.6	7.1	7.7	5.8	6.5	5.8	4.7	3.9	5.5	6.9	6.8	7.9	6.2
1982	10.0	8.9	9.1	9.3	6.7	6.8	5.4	5.2	6.4	6.7	7.5	9.2	7.6
1983	7.5	9.2	8.5	9.8	8.2	6.3	5.2	5.3	5.5	5.5	7.6	9.1	7.3
1984	7.4	8.1	8.1	9.4	7.9	6.0	5.3	5.3	7.5	6.5	7.9	8.0	7.3
1985	7.9	8.9	8.7	7.8	6.8	6.5	5.8	6.2	7.0	8.1	7.4	7.1	7.4
1986	7.4	8.7	6.9	7.4	6.9	5.1	5.5	5.3	5.8	5.6	8.1	5.9	6.6
1987	7.6	8.5	7.8	7.3	5.9	6.1	5.4	4.8	5.6	6.9	8.9	8.6	7.0
AVERAGE	8.3	8.8	8.9	8.4	7.4	6.3	5.5	5.2	6.3	6.4	7.4	7.8	7.2

SOURCE: National Climatic Center

TABLE 5

RESULTANT WIND DIRECTION

1973-1987

BATON ROUGE AT RYAN AIRPORT

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
1973	05	01	16	17	19	15	26	07	11	08	15	21	13
1974	11	21	17	14	15	15	25	12	07	09	11	18	13
1975	13	16	14	12	13	15	26	12	03	05	10	05	11
1976	06	20	15	15	23	09	27	01	04	01	01	03	05
1977	01	20	14	15	12	22	23	12	14	05	11	15	13
1978	01	36	30	15	13	09	22	08	07	03	07	11	06
1979	01	06	16	12	14	12	14	18	03	13	02	04	08
1980	05	03	11	04	14	21	24	11	09	02	02	03	07
1981	01	36	03	15	13	17	23	10	05	07	08	05	09
1982	12	02	14	11	14	22	22	20	06	06	09	09	10
1983	02	03	27	17	15	07	07	05	05	05	11	34	05
1984	36	16	16	17	12	15	18	12	03	11	04	10	10
1985	32	03	14	11	22	21	25	08	07	07	11	02	07
1986	01	21	17	16	14	15	22	09	13	11	08	03	14
1987	29	08	03	28	12	11	13	26	01	04	06	11	06

* Wind Direction-Numerals indicate tens of degrees clockwise from true north, 00 indicates calm, 09 east, 18 south, 27 west, 36 north. Resultant direction is the vector sum of wind directions diveded by number of observations.

SOURCE: National Climatic Center

Physiography and Geology

The study area is in the Southern Pine Hills of the Eastern Gulf Coastal Plain. Topography in the northern portion of the basin is dominated by plateaus and ridgetops underlain by the Citronelle Formation. The southern portion is dominated by gently sloping Pleistocene terrace surfaces.

The maximum elevation within the basin is approximately 500 feet MSL. Elevations are between 35 feet and 40 feet MSL near the junction of the Comite River and Amite River near Denham Springs. Minimum elevations are between 0 and 5 feet in the lower part of the basin near Lake Maurepas.

Although older sediments are found at depth in the study area only the Plio-Pleistocene, and Holocene sediments exposed at the surface and found near the surface are discussed. Four distinct geologic units are found within the basin: the Citronelle Formation, the Pleistocene terraces, the loess deposits and Holocene alluvium. The Citronelle Formation which varies in age from late Pliocene to Pleistocene, generally consists of a gradational sequence of fluvial gravels, cross bedded sands, silts and clays with the coarser grained material occurring at the base of this sequence. South of the outcrop of the Citronelle Formation are found the relatively flat Pleistocene terraces of less variable lithology than that of the Citronelle Formation. Generally, these terraces are comprised of sediments consisting of silt and sandy clay which grade downward into a fine to coarse grained sand with some gravel.

The study area is located in a stable area of low seismicity. Earthquake activity is relatively rare and is usually less severe than average. Resulting damage to structures and levees (dikes) in the project area would be expected to be minor.

Economy

The general economic study area for purposes of this report includes Ascension, East Baton Rouge, East

Feliciana, Livingston, and St. Helena Parishes. The economy of the study area is based largely on the processing of oil and gas, port activities, the activities of two state universities, supporting sales and services, and activities of government administration. Although the area has experienced the adverse effects of the decline in oil production during the 1970's and 1980's, one of the largest oil refineries in the United States is located in Baton Rouge and is still important to the local economy. Latest data available (1989) show that the Port of Baton Rouge has the fifth largest volume of tonnage in the United States, and that oil and other energy related products are among the major commodities which move through the port. The City of Baton Rouge is the home of the state capital and attracts government activity and related employment. Latest (1991) estimates reported by Louisiana's Department of Agriculture and Forestry show that timber production in the study area accounted for about 7.9 percent of the total stumpage value severed in the state. The latest (1987) Census of Manufactures reported that the value by manufacture in Ascension and East Baton Rouge Parishes represented 21.4 percent of the state total. It reported that about 17,300, or about 10.7 percent, of the state's manufacturing jobs were in these two parishes. (Data for East Feliciana, Livingston, and St. Helena Parishes were withheld by the U.S. Department of Commerce to avoid disclosure of confidential information of individual companies.)

The structure of a region's economy is often reflected by the level of employment in major economic sectors. Most of the study area's work force are covered under the Louisiana Employment Security Law. As of the 3rd quarter of 1991, employment in this group totaled about 232,100 people. Approximately 9.1 percent were employed in public administration. An estimated 33.6 percent were employed in the service sector, with another 17.1 percent employed in retail trade. Manufacturing and construction accounted for 9.5 percent and 13.2 percent, respectively. The remaining 17.5 percent were spread throughout other sectors of the economy, including agriculture, mining, wholesale trade, finance, and transportation.

Human Resources

The 1990 Census of Population estimated the number of inhabitants living within the Amite River Basin study area at about 538,000, an increase of about 6.7 percent since 1980. Population of the state increased from 4,206,312 in 1980 to 4,219,973 in 1990 or about 0.3 percent. Table 6 compares historic and recent population estimates within the study area. The 1990 population of the City of Baton Rouge was about 40 percent of the population in the study area. The population of the parishes which are part of the Baton Rouge Metropolitan Statistical Area (MSA), including Ascension, East Baton Rouge, and Livingston Parishes, made up almost 95 percent of the study area.

The number of households in the study area in 1990 was 190,688, representing an increase of 14.7 percent over the 166,233 figure reported in the 1980 census. The 1990 estimate of total housing units was 215,096, with 125,170 of them owner-occupied. The median value of specified owner-occupied housing units ranged from a low of $40,000 in St. Helena Parish to a high of $69,200 in East Baton Rouge.

As of this writing, detailed general social and economic characteristics from the 1990 census are unavailable. The 1980 census indicated that about 75,000 of the 503,900 people living in the study area were at, or below, the nationally recognized poverty level. This amount represented 14.9 percent of the total population. At that time (1980) the total work force was estimated to be 223,700 with 12,700 unemployed, indicating an unemployment rate of 5.7 percent. Unpublished reports furnished by Louisiana's Office of State Planning show that by 1990 the average civilian labor force had increased to 269,700. The number of unemployed people in the labor force had increased to 14,600, resulting in an unemployment rate of 5.4 percent. As has been well documented and widely reported in news media, and as suggested by the relatively low rate of population increase between 1980 and 1990, unemployment during the 1980's was unusually high. Average unemployment in the study area in 1986, for example, was about 11.0 percent. In 1986 unemployment nationally averaged about 7.0 percent. As of the 1980 census, the median household income in the study area

ranged from $9,653 in St. Helena Parish to $19,192 in Ascension Parish. The mean household income ranged from a low of $13,596 in St.Helena to a high of $21,757 in East Baton Rouge Parish. The April 1992 report of the "Survey of Current Business" published by the U.S. Department of Commerce showed that per capita personal income in East Baton Rouge Parish was $17,881 in 1990, highest among Louisiana Parishes. Per capita personal income for the state was $14,528. Per capita personal income in St. Helena Parish was $8,797, the lowest in the state. Per capita personal income for the 5-parish study area was $16,100, or about the same as St. Charles Parish which ranked seventh among Louisiana parishes in 1990.

Transportation

The study area is served by a variety of transportation systems, including deep-draft navigation by way of the Mississippi River and the Port of Baton Rouge, several small shallow-draft channels leading to Lake Pontchartrain, Federal, State, and local roads, railroads, and a commercial airport. As noted elsewhere in this report, the Port of Baton Rouge is one of the most active in the United States. While commercial use is limited, shallow draft waterborne commerce has been available on the Amite River, Blind River, and Bayou Manchac. The area's highway system includes Interstate Highways 10 and 12 (I-10 and I-12), which pass through the City of Baton Rouge. Its eastern boundary parallels I-55. Many other Federal, State, and local highways in the study area have developed and been improved as industrial growth and urban expansion has occurred, including U.S. Highways 61 and 190, and Louisiana Highways 10, 16, 38, 42, 22, 30, 68, and 19, 67, 37, 63, and 43, as well as many other local roadways. North-south rail transportation is provided primarily by the Louisiana-Arkansas Railway and the Illinois Gulf Central Railroad. The Missouri Pacific Railroad provides east-west transportation. Numerous local railway spurs serve the port and industrial developments in the metro area. Air transportation is provided at Ryan International Airport in Baton Rouge. Several smaller landing strips are also within the study area, used primarily by agricultural purposes.

TABLE 6

HISTORICAL POPULATION TRENDS WITHIN THE STUDY AREA

PARISH	1940	1950	1960	1970	1980	1990	1991
Ascension	21,215	22,387	27,927	37,086	50,086	58,214	59,009
East Baton Rouge	88,415	158,236	230,058	285,167	366,191	380,105	383,983
East Feliciana	18,039	19,133	20,198	17,657	19,015	19,211	19,362
Livingston	17,790	20,054	26,974	36,511	58,806	70,526	71,478
St. Helena	9,542	9,013	9,162	9,937	9,827	9,874	9,912
TOTAL	155,001	228,823	314,319	386,358	503,925	537,930	543,744

SOURCES:

U.S. Department of Commerce, Bureau of the Census, Census of Population, for periods 1940-1990.

Louisiana Tech University, College of Administration and Business, Research Division (1991 population estimate).

Existing Land Use

Historical land use for the Amite River Basin in 1954, 1972, 1978 and 1985 are shown in Tables 7, 8, 9, and 10. A geographic information system (GIS) was used to map historical land use changes. Land use in the Amite River Basin is largely forest and agricultural. In 1985, these uses made up 72 percent of the total land use. Forest and agricultural land uses have declined since 1954 from about 83 percent of the total land use to 72 percent in 1985. About 85 percent of the forest and agricultural lands are located in the Mississippi portion of the basin, East Feliciana, Livingston, and East Baton Rouge Parishes. This decline is primarily due to the conversion of forest and/or agricultural lands to urban lands. Some forest lands have been converted to agricultural lands. A recent trend is the conversion of agricultural lands to forest lands for the timber that can be harvested. Urban and built-up lands have increased from 1 percent in 1954 to 12 percent in 1985. Most of the urban growth has occurred in East Baton Rouge, Livingston, and Ascension Parishes. In 1985, these parishes contained 55, 18, 14 percent, respectively, of the urban and built-up land. Urban and built-up land are generally located along major transportation arteries and along the banks of major streams and rivers.

Waters, wetlands, and barren land have been relatively constant making up about 16 percent of the total land use. Land use maps for 1954, 1972, 1978, and 1985 are shown on Plates 4, 5, 6, and 7, respectively. The photo interpretation upon which the land uses in the study area is based identifies cypress-tupelo swamps, shrub swamps, and other similar types as wetlands.

Development trends in the two parishes that contain 72 percent of the urban and built-up lands are shown in Table 11.

TABLE 7

LAND USE IN THE AMITE RIVER BASIN, 1954

(ACRES)

STATE/PARISH	URBAN OR BUILT UP LAND	FOREST LAND	NON-FOREST LAND	TOTAL
Louisiana				
Ascension	0	88,816	56,784	145,600
East Baton Rouge	16,183	114,092	139,445	269,720
East Feliciana	0	110,297	123,879	234,176
Iberville	0	9,564	14,711	24,275
Livingston	750	219,893	29,519	250,163
St. Helena	0	76,025	11,060	87,085
St. James	0	63,421	835	64,256
St. John the Baptist	0	14,896	784	15,680
Mississippi	0	158,567	178,094	336,661
TOTAL	16,934	855,571	555,111	1,427,616
% OF TOTAL	1	60	39	100

SOURCE: U.S. Army Corps of Engineers, New Orleans District

TABLE 8

LAND USE IN THE AMITE RIVER BASIN, 1972

(ACRES)

STATE/PARISH	URBAN OR BUILT-UP LAND	AGRICULTURAL LAND	FOREST LAND	WATER	WETLANDS	BARREN
Louisiana						
Ascension	4,368	62,608	29,120	1,456	46,592	1,456
East Baton Rouge	53,195	126,317	82,702	1,100	5,360	1,046
East Feliciana	4,449	124,113	103,037	234	703	1,639
Iberville	1,214	12,623	2,670	243	6,797	728
Livingston	5,003	37,524	155,101	1,251	50,033	1,251
St. Helena	1,219	17,417	58,260	610	2,438	7,141
St. James	1,478	1,285	0	0	61,493	0
St. John the Baptist	1,098	220	0	1,019	13,344	0
Mississippi	10,100	94,265	225,563	3,367	0	3,367
TOTAL	82,873	476,824	654,667	8,988	186,794	22,325
% OF TOTAL	6	33	46	1	13	2

SOURCE: U.S. Army Corps of Engineers, New Orleans District 1

TABLE 9
LAND USE IN THE AMITE RIVER BASIN, 1978
(ACRES)

STATE/PARISH	URBAN OR BUILT-UP LAND	AGRICULTURAL LAND	FOREST LAND	WATER	WETLANDS	BARREN
Louisiana						
Ascension	17,909	45,427	35,818	1,310	44,408	728
East Baton Rouge	79,298	92,514	83,343	809	7,013	6,743
East Feliciana	4,449	111,702	110,765	234	703	6,323
Iberville	1,918	11,215	9,055	97	1,991	0
Livingston	23,015	28,018	142,593	1,251	52,784	2,502
St. Helena	1,219	10,624	65,052	610	2,438	7,141
St. James	1,478	1,285	0	0	61,493	0
St. John the Baptist	1,098	220	0	1,019	13,344	0
Mississippi	10,100	94,265	225,563	3,367	0	3,367
TOTAL	140,483	395,271	672,189	8,697	184,173	26,803
% OF TOTAL	10	28	47	1	13	2

SOURCE: U.S. Army Corps of Engineers, New Orleans District 1

TABLE 10
LAND USE IN THE AMITE RIVER BASIN, 1985
(ACRES)

STATE/PARISH	URBAN OR BUILT-UP LAND	AGRICULTURAL LAND	FOREST LAND	WATER	WETLANDS	BARREN
Louisiana						
Ascension	23,442	42,952	33,051	1,456	44,117	582
East Baton Rouge	92,784	86,580	76,870	1,079	6,473	5,934
East Feliciana	5,386	109,594	111,468	234	703	6,791
Iberville	2,330	10,997	8,885	73	1,991	0
Livingston	29,769	28,769	135,088	1,501	52,784	2,251
St. Helena	1,568	12,540	58,173	871	2,700	11,234
St. James	3,855	321	0	0	60,079	0
St. John the Baptist	784	251	0	1,019	13,626	0
Mississippi	10,100	94,265	225,563	3,367	0	3,367
TOTAL	170,018	386,269	649,097	9,600	182,472	30,159
% OF TOTAL	12	27	45	1	13	2

SOURCE: U.S. Army Corps of Engineers, New Orleans District 1

TABLE 11

HISTORICAL URBAN DEVELOPMENT TRENDS
WITHIN AND OUTSIDE 100-YEAR FLOOD PLAIN
(ACRES)

	1972	
PARISH	WITHIN 100-YEAR FLOODPLAIN	OUTSIDE 100-YEAR FLOODPLAIN
EAST BATON ROUGE	8,307	44,888
% PERCENT	16	84
LIVINGSTON	2,709	2,297
% PERCENT	54	46
TOTAL	11,016	47,185
% PERCENT	19	81

	1978	
PARISH	WITHIN 100-YEAR FLOODPLAIN	OUTSIDE 100-YEAR FLOODPLAIN
EAST BATON ROUGE	18,239	61,059
% PERCENT	23	77
LIVINGSTON	8,516	14,499
% PERCENT	37	63
TOTAL	26,755	75,558
% PERCENT	26	74

	1985	
PARISH	WITHIN 100-YEAR FLOODPLAIN	OUTSIDE 100-YEAR FLOODPLAIN
EAST BATON ROUGE	22,268	70,516
% PERCENT	24	76
LIVINGSTON	10,717	19,052
% PERCENT	36	64
TOTAL	32,985	89,568
% PERCENT	27	73

SOURCE: U.S. Army Corps of Engineers, New Orleans District

The greatest increase in urban development within the 100-year floodplain occurred between 1972 and 1978. In Livingston Parish, urban development outside the 100-year floodplain grew at a faster rate than development within the 100-year flood plain. Growth rates are shown in Table 12.

TABLE 12

INCREASE IN URBAN DEVELOPMENT WITHIN AND OUTSIDE THE 100-YEAR FLOODPLAIN FOR SELECTED PARISHES

	1972-1978		1978-1985		1972-1985	
PARISH	WITHIN 100-YEAR FLOODPLAIN	OUTSIDE 100-YEAR FLOODPLAIN	WITHIN 100-YEAR FLOODPLAIN	OUTSIDE 100-YEAR FLOODPLAIN	WITHIN 100-YEAR FLOODPLAIN	OUTSIDE 100-YEAR FLOODPLAIN
EAST BATON ROUGE	2.2	1.35	1.22	1.16	2.7	1.57
LIVINGSTON	3.1	6.3	1.26	1.31	4.0	8.3

The decrease in the growth rate within the 100-year floodplain can probably be attributable to the general decrease in overall economic growth in the basin. In addition, major floods occurred in the basin in 1977, 1979, and 1983 that have placed more emphasis on the judicious use of the floodplain between 1978 and 1985. Urban development within and outside the floodplain grew at about the same rate between 1978 and 1985.

In St. Helena Parish, most of the land is forest lands. Agricultural lands in the parish are primarily used for daily farming and pasture, no extensive clearing of land has taken place in the last 15 years. The major land clearing in the parish is by the International Paper Company and Crown Zellerbach. According to the Soil and Water Conservation District, these companies used a select cut method to obtain trees to reduce significant increases in runoff.

The most significant land clearing in the last 15 years has been in East Baton Rouge and Livingston Parishes.

The clearing has been for urban development. The land clearing has slowed in recent years due to the overall down turn in the economy. Between 1954 and 1985, about 106,000 acres were cleared for urban development in the parishes. In Livingston Parish, houses are being built quite some distance apart. However, runoff from these lands will still affect local drainage in Livingston Parish. East Baton Rouge Parish is being more densely developed with subdivisions and shopping centers and runoff from these areas has significantly affected the local drainage basin.

Very little land clearing in the basin in recent years is due to agriculture. The major crops in East Baton Rouge Parish are soybeans and wheat. In Livingston Parish, most of the agricultural land is devoted to pasture with less than 500 acres in soybeans. There are about 10 dairy farms in Livingston Parish.

Some land in East Feliciana Parish has been cleared for urban development. However, the overall land clearing has been small in terms of total wooded acres. Most of the land clearing in the parish has been in the southern portion. Major crops in the parish are pasture, forest products, and soybeans.

Significant urban development has occurred in Ascension Parish at the expense of agricultural land and, to a lesser extent, wetlands. About 23,000 acres has been developed since 1954 in the Amite River basin. No extensive urban development or land clearing has occurred in Iberville, St. James, or St. John the Baptist Parishes or Counties in Mississippi.

Biological Resources

The area that is described as the study area for environmental analysis is that area that is to be economically benefitted by the most effective planned flood control action. That area is approximately 50 percent forested, 25 percent cleared agricultural lands, and 25 percent residential and commercially developed land (see Table 13).

TABLE 13
1985 LAND USE

Land Use Type	Number of Acres
Forested lands	172,523
Agricultural lands	84,629
Developed lands	84,447
Transitional lands	4,876
Total lands	346,475

SOURCE: U.S. Army Corps of Engineers, New Orleans District

Most of the cleared agricultural land is utilized as pasture, with a small amount being in row crops. There are pine plantations in the northern portion, with bottomland hardwood forests and some mixed pines occurring naturally on the remainder of the wooded portions of the area. The hardwood forests are extremely valuable habitat for numerous species of game and non-game wildlife. The wildlife habitat provided by the cleared lands of the study area is not considered to be nearly as significant as that provided by wooded lands. The urban area made up of the Baton Rouge metropolitan area is extensive. Some older residential portions have lots with large trees which provide limited habitat for song birds and gray squirrels. The major aquatic resources of the area consist of the Amite and Comite Rivers. Other aquatic resources include White, Cypress, Lilly, and Cooper Bayous and Bayou Baton Rouge. The Amite and Comite Rivers are meandering streams with steep slopes and highly mobile beds in their upper reaches. The Comite is often relatively clear except during responses to rainfall. Extensive sand and gravel mining occurs on the upper Amite and as a result this river is commonly very turbid. Below where the Comite flows into the Amite near Denham Springs, the Amite widens and the flow is generally slower than in upper reaches. The river bottoms consist mainly of sand and gravel with benthic populations indicative of that type of substrate. The fishery consists mainly of species typical of flowing streams. Studies have shown the Comite to possess a

standing crop of about 100 pounds per acre, while the Amite is less productive, containing only about 40 pounds per acre. Studies report the lower portion of the Amite has the largest numbers of benthic fauna. The banks of both rivers are heavily lined with willows and river birches. This riparian vegetation is extremely valuable since it provides shade and leaf litter. The litter is highly significant since it is the major component of the detritus base of the aquatic food web. The overhanging trees also increase the esthetic appeal of the streams.

Surface Water

The major rivers in the study area are the Amite River and the Comite River. The Amite River is used for recreation, propagation of fish and wildlife, and to a lesser extent, for water supply, navigation, and waste disposal. The Amite River has a drainage area of about 2,200 square miles and an average flow of about 2,000 cfs at Denham Springs. The Comite River has a drainage area of 334 square miles and an average flow of 457 cubic feet per second (cfs) near Comite, Louisiana. Significant tributaries of the Amite River are Beaver, Darling and Bluff Creeks, Wind Bayou, the Comite River, Jones Creek, Clay Cut Bayou, Bayou Manchac, Henderson Bayou, and Colvell Bayou.

Groundwater

Fresh groundwater in the study area is obtained from the Pliocene, Miocene, and quaternary age deposits as well as undifferentiated aquifers that occur in alluvial coastal and upland deposits. Deposits of Pliocene age consist of medium to very fine grained sand beds alternating with silt and clay beds. These sediments thicken and dip steeply toward the Gulf of Mexico, reaching a thickness of about 2,200 feet near the southern limit of freshwater availability. Miocene age deposits consist mostly of lenticular deposits of fine- to medium-grained sand and beds of silt and clay. In some areas, very coarse sand and gravel are present. Individual sands may be as thick as 150 feet. These deposits are wedge-shaped and thicken greatly as they extend toward the Gulf of Mexico.

Quaternary deposits cover Miocene and Pliocene aquifers in nearly all of the study area. The Quaternary deposits range in thickness from less than 50 feet in the north to more than 3,500 feet near the coast. The maximum depth at which these deposits contain fresh water is about 1,000 feet.

Water Supply

Historical and existing water use in the Amite River Basin is shown in Table 14. In 1980, about 90 percent of the water use is from surface water sources. The major source of surface water is the Mississippi River which is located outside the Amite River Basin. The base of fresh groundwater aquifers occur at various depths including depths of more than 3,000 feet below NGVD. Below the base of fresh groundwater, the water gradually grades into brackish water and then saline water. In general, moving southward through the Amite River Basin, the fresh water rises and brackish and saline waters occurs progressively closer to the surface. As a result, some difficulties may be experienced in obtaining sufficient quantities of fresh water for public supply or industrial requirements. Some layers or lenses of fresh groundwater are present; however, pumping must be carefully controlled to prevent saltwater encroachment. Surface water is a major source of water for industrial use in the Amite River Basin. Surface water is a major source of drinking water in Ascension, St. James, and St. John the Baptist Parishes.

Table 14

HISTORICAL AND EXISTING WATER USE IN THE LOUISIANA PORTION OF AMITE RIVER BASIN

(million gallons per day)

		PUBLIC SUPPLY		INDUSTRIAL		POWER GENERATION		RURAL	LIVE STOCK		RICE		OTHER IRRIG.		AQUACULTURE		TOTAL		
	YEAR	GROUND	SURFACE	GROUND	SURFACE	GROUND	SURFACE	GROUND	GROUND	SURFACE	GROUND	SURFACE	GROUND	SURFACE	GROUND	SURFACE	GROUND	SURFACE	TOTAL
ASCENSION	1960	0.30	1.26	1.12	2.00	0.00	0.00	0.54	0.24	0.01	0.00	0.00	0.04	0.06	0.00	0.00	2.24	3.33	5.57
	1965	0.47	0.69	2.79	12.23	0.00	0.00	0.66	0.16	0.01	0.00	2.48	0.64	0.33	0.00	0.00	4.72	15.74	20.46
	1970	0.50	0.73	3.35	108.53	0.00	0.00	0.66	0.13	0.01	0.00	0.00	0.38	0.00	0.00	0.00	5.02	109.27	114.29
	1975	0.67	1.14	2.50	115.00	0.00	0.00	0.66	0.14	0.02	0.00	0.00	1.23	0.00	0.00	0.00	5.20	116.16	121.36
	1980	0.72	1.40	4.20	202.00	0.00	0.00	2.17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.36	7.09	203.76	210.85
E.BATON ROUGE																			
	1960	19.20	0.00	67.73	366.62	7.52	7.56	1.45	0.02	0.48	0.00	0.00	0.00	0.00	0.00	0.00	95.92	374.66	470.58
	1965	28.93	0.00	59.99	339.96	7.32	5.76	0.94	0.13	0.29	0.00	0.00	0.00	0.00	0.00	0.00	95.31	346.01	441.32
	1970	32.40	0.00	99.59	361.53	7.48	7.63	0.35	0.28	0.38	0.00	0.00	0.35	0.05	0.00	0.00	140.45	369.59	510.04
	1975	39.90	0.00	84.70	114.00	7.14	5.29	0.31	0.14	0.14	0.00	0.00	0.19	0.11	0.00	0.00	132.38	119.54	251.92
	1980	53.90	0.00	86.50	69.00	7.07	4.03	1.73	0.14	0.02	0.00	0.00	0.14	0.00	0.08	0.00	149.54	73.05	222.59
E. FELICIANA																			
	1960	0.20	0.00	0.84	2.60	0.00	0.00	0.25	0.38	0.02	0.00	0.00	0.00	0.00	0.00	0.00	1.67	2.62	4.29
	1965	0.34	0.00	1.31	0.00	0.00	0.00	0.32	0.02	0.35	0.00	0.00	0.00	0.00	0.00	0.00	1.99	0.35	2.34
	1970	0.35	0.00	1.55	0.00	0.00	0.00	0.49	0.02	0.37	0.00	0.00	0.00	0.00	0.00	0.00	2.41	0.37	2.78
	1975	0.88	0.00	0.03	0.00	0.00	0.00	0.19	0.02	0.35	0.00	0.00	0.05	0.00	0.00	0.00	1.17	0.35	1.52
	1980	1.05	0.00	0.03	0.00	0.00	0.00	0.42	0.12	0.12	0.00	0.00	0.03	0.00	0.03	0.00	1.68	0.12	1.80
IBERVILLE																			
	1960	1.00	0.00	5.41	155.00	0.41	136.00	0.31	0.21	0.05	0.00	0.00	0.00	0.00	0.00	0.00	7.34	291.05	298.39
	1965	1.84	0.00	7.59	278.42	0.86	230.40	0.49	0.22	0.05	0.00	2.43	0.00	0.00	0.00	0.00	10.80	511.30	522.10
	1970	0.59	0.00	11.49	525.11	1.30	572.00	0.28	0.07	0.13	0.00	0.00	0.00	0.00	0.00	0.00	13.73	1097.24	1110.97
	1975	1.35	0.00	30.10	643.00	1.32	632.00	0.01	0.26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	33.04	1275.00	1308.04
	1980	1.99	0.00	36.90	655.00	2.20	730.00	0.16	0.15	0.00	0.00	0.00	0.00	0.00	0.00	3.36	41.40	1388.36	1429.78
LIVINGSTON																			
	1960	0.82	0.00	0.28	3.00	0.00	0.00	0.55	0.02	0.00	0.00	0.00	0.76	1.10	0.00	0.00	2.43	4.10	6.53
	1965	0.89	0.00	0.05	0.00	0.00	0.00	1.54	0.30	0.06	0.00	0.00	0.63	0.22	0.00	0.00	3.41	0.28	3.69
	1970	1.28	0.00	1.21	0.00	0.00	0.00	3.84	0.24	0.08	0.00	0.00	0.94	0.00	0.00	0.00	7.51	0.08	7.59
	1975	0.97	0.00	1.16	0.00	0.00	0.00	0.65	0.16	0.05	0.00	0.00	0.19	0.00	0.00	0.00	3.13	0.05	3.18
	1980	2.92	0.00	0.03	0.00	0.00	0.00	2.82	0.12	0.00	0.00	0.00	0.04	0.00	0.00	0.00	5.93	0.00	5.93
ST. HELENA																			
	1960	0.07	0.00	0.05	0.50	0.00	0.00	0.25	0.02	0.48	0.00	0.00	0.00	0.00	0.00	0.00	0.39	0.98	1.37
	1965	0.08	0.00	0.00	0.00	0.00	0.00	0.27	0.03	0.50	0.00	0.00	0.00	0.03	0.00	0.00	0.38	0.53	0.91
	1970	0.09	0.00	0.00	0.00	0.00	0.00	0.28	0.02	0.29	0.00	0.00	0.00	0.45	0.00	0.00	0.39	0.74	1.13
	1975	0.18	0.00	0.00	0.00	0.00	0.00	0.21	0.11	0.22	0.00	0.00	0.00	0.00	0.00	0.00	0.48	0.22	0.70
	1980	0.21	0.00	0.00	0.00	0.00	0.00	0.52	0.49	0.06	0.00	0.00	0.00	0.00	0.00	0.00	1.22	0.06	1.28
ST. JAMES																			
	1960	0.00	0.11	5.41	42.40	0.00	0.00	0.06	0.07	0.00	0.00	0.00	0.01	0.01	0.00	0.00	5.55	42.52	48.07
	1965	0.00	2.36	7.83	8.08	0.00	0.17	0.07	0.00	0.00	2.14	0.27	0.18	0.00	0.00	0.00	10.22	10.88	21.10
	1970	0.00	0.99	5.36	220.00	0.00	0.00	0.14	0.06	0.01	0.00	0.00	0.02	0.45	0.00	0.00	5.58	221.45	227.03
	1975	0.00	1.66	5.15	275.00	0.00	0.00	0.04	0.02	0.00	0.00	0.00	0.11	7.91	0.00	0.00	5.32	284.57	289.89
	1980	0.00	2.39	7.47	264.00	0.00	0.00	0.03	0.00	0.00	0.00	0.00	0.01	0.00	0.00	22.03	7.51	288.42	295.93
ST. JOHN																			
	1960	0.10	1.55	0.18	15.70	0.00	0.00	0.06	0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.36	17.27	17.63
	1965	0.05	1.77	3.83	32.60	0.00	0.00	0.13	0.02	0.00	0.00	1.08	0.00	0.00	0.00	0.00	4.03	35.45	39.48
	1970	0.07	1.35	5.25	53.73	0.00	0.00	0.09	0.01	0.01	0.00	0.00	0.07	0.00	0.00	0.00	5.49	55.09	60.58
	1975	1.03	1.37	3.88	87.70	0.00	0.00	0.05	0.01	0.01	0.00	0.00	0.00	0.35	0.00	0.00	4.97	89.43	94.40
	1980	1.52	3.25	4.03	93.70	0.00	0.00	0.33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.16	5.88	97.11	102.99
TOTAL	1960	21.89	2.92	81.02	587.82	7.93	143.56	3.47	0.98	1.06	0.00	0.00	0.81	1.17	0.00	0.00	115.90	736.53	852.43
	1965	30.40	4.82	83.39	671.29	8.18	236.33	4.42	0.86	1.26	2.14	6.26	1.45	0.58	0.00	0.00	130.86	920.54	1061.40
	1970	35.28	3.07	127.80	1268.90	8.78	579.63	6.13	0.93	1.28	0.00	0.00	1.76	0.95	0.00	0.00	180.58	1853.83	2034.41
	1975	44.96	4.17	127.52	1234.70	8.46	637.29	2.12	0.86	0.79	0.00	0.00	1.77	8.37	0.00	0.00	186.69	1885.32	2071.01
	1980	62.31	7.04	139.16	1283.70	9.27	734.03	8.18	1.02	0.20	0.00	0.00	0.22	0.00	0.08	25.93	220.26	2050.90	2271.15

Source: Louisiana Department of Transportation and Development

Water Quality

No water quality samples were specifically collected and analyzed to provide data for this study. Water quality data used to characterize the general water quality of water bodies in the study area were obtained from the water quality files of the Environmental Protection Agency (EPA) computerized storage and retrieval (STORET) system database. Water quality sampling stations used in the analyses are shown in Plates 8 and 9. Several of the stations frequency of sampling were both sporadic and infrequent.

Comite River

The Comite River, from Louisiana Highway 10 to White Bayou, has been designated a Louisiana Natural and Scenic stream by the Louisiana Department of Environmental Quality (LDEQ). The Comite River is categorized as an effluent limited stream; which is, by definition, any stream segment in which the best practicable treatment levels for point source discharges are required to maintain the stream's standards.

Of the parameters analyzed for the Comite River only pH values violated the state standards. The state standards indicate that pH should generally fall within the range of 6.0 to 8.5. Low pH values were observed in the Comite River near Olive Branch, Louisiana, and Comite, Louisiana. Near Olive Branch, Louisiana, only one pH value (6% of the total pH values) was below the minimum 6.0 (standard units) SU state standard. Two pH values or about 5% of the total pH observations near Comite, Louisiana, were below the state standard.

Though no DO concentrations are available at these three sampling locations on the Comite River, the Louisiana Department of Wildlife and Fisheries (LDWF) collected water quality samples on the lower Comite River in October 1973 and again in May 1980. The LDWF collected a total of six samples and reported that the DO levels were consistently between 7 and 9 mg/l. These values are well above the minimum 5.0 mg/l state standard. It should be noted that the above samples were collected at times of low flow conditions. Generally, lower DO values along a stream

segment are found during low flow and warm weather conditions. Samples taken on Comite River tributaries on the same dates yield mean DO levels between 1.7 and 9.3 mg/l with 0.0 mg/l reported for one measurement on North Branch of Hurricane Creek. The EPA standard of 100 mg/l was violated for both of the two observation of total phosphorus along the Comite River.

Though no fecal coliform data was collected at the three stations on the Comite River, the LDWF collected water quality samples on the lower Comite River in October 1973 and May 1980. The LDWF collected a total of 6 samples and reported averages of 330 fecal coliform colonies/100 ml. This is in excess of the maximum 200/100 ml state standard. These violations are the result of the numerous package treatment plants that treat municipal waste from subdivisions along the Comite River and tributary streams. Samples taken on the same dates on Comite River tributaries yielded average fecal coliform counts of 7,000; 6,000; 22,000; 170,000; 500; and 8,000 colonies/100 ml.

The water conditions described in the above paragraphs are based on low flow conditions. The characterization is based upon limited data. At low flow conditions, water quality is likely low in dissolved oxygen (DO) and high on coliform bacteria. Recent water quality data collected for input into a water quality model is summarized below:

Date	Stream	Dissolved Oxygen (mg/l)	Coliform (Colonies/100ml)	Total Phosphorus (mg/l)	pH
9/10/90	Comite River	6.1	680	0.02	6.6
9/10/90	White Bayou	2.5	320	0.1	6.7
10/9/90	Comite River	6.2	106	0.16	6.1
10/9/90	White Bayou	3.7	96	0.12	6.5
4/1/91	Comite River	9.4	54	0.11	5.9
4/1/91	White Bayou	7.2	118	0.26	6.1
5/8/91	Comite River	5.2	2960	0.45	5.5
5/8/91	White Bayou	4.2	3160	0.34	5.5

In general, the water quality of the Comite River & Tributaries streams in the area during average flow conditions can be characterize as generally good.

Amite River

The Amite River, from the Louisiana-Mississippi state line to LA Hwy 37 is designated a Louisiana Natural and Scenic stream. The Amite River is also an effluent limited stream segment. Standards for pH and total dissolved solids (TDS) were exceeded at all five sampling locations.

The station near Darlington, Louisiana, at Highway 10 had the greatest percentage of violations with respect to pH values, with 19% of the values below the pH standard of 6.0 SU. The minimum pH value recorded at this site was 5.2 SU. The station located near Magnolia had the lowest pH value which was 4.9 SU. However, only 6% of the total pH values measured at the Magnolia location were in violation of the state standard. Low pH values are of concern since many pollutants are known to become more toxic as pH becomes lower. It is interesting that, like the Comite River, the pH levels in the Amite River increase at the downstream locations. Since the northern portions of the river basins are mostly forested and agricultural lands, perhaps these lower pH values are the result of agricultural and silviocultural practices.

Although the state standard for total dissolved solids (TDS) was violated at all five locations, the maximum percent exceedance was 7% at the farthest downstream location near the Highway 42 Bridge. Except for this location, the mean TDS concentrations for the other sampling locations is about 54 mg/l; well below the maximum 150 mg/l state standard.

For the three station locations on the upper Amite River, there were no DO violations. The mean DO concentrations at these stations were about 8.3 mg/l. The station at the 4H Camp near Denham Springs had one DO violation in 131 observations. However, about 22% of the DO concentration observed at the Highway 42 bridge violated the minimum 5.0 mg/l state standard. The mean violation was about 4.1 mg/l with the violations equally distributed throughout the months of May through October. Severe

oxygen depletion has been reported in the Amite River below the Amite River Diversion Canal.

The mean chloride and sulfate concentrations are well within the state standards. The percent exceedance values are generally rare and much less than 5%. The furthest downstream location is the exception with a percent exceedance value of 6%. The exceedances at this downstream location are probably due to the influence of brackish water from Lakes Pontchartrain and Maurepas.

The LDEQ has set guidelines for maximum turbidity levels in the Amite River at 50 nephelometric turbidity unites (NTU). For the reach of the Amite River designated as scenic, the guideline is 25 NTU. Although, the mean turbidity levels measured at all of the stations are within these guidelines, there is about a 20 - 25% exceedance value at each of the stations. These high turbidity levels are the result of early storm runoff and sand and gravel mining operations in the Amite River. In the Amite River, 73% of the total phosphorus values exceeded the 50 mg/l EPA standard.

Generally the quality of water, with regard to fecal coliforms, decreases as one progresses downstream. The log means range from 153 colonies/100 ml at the Grangeville Bridge location to 884 colonies/100ml at the Highway 42 Bridge location. The 90th percentile values for all five locations are well above the 400 colonies/100 ml state standard. These fecal coliform violations can be attributed to stormwater runoff and domestic wastewater discharges from Baton Rouge that enter the Amite River directly or via other tributaries.

There are consistent exceedances of the acute criteria for cadmium, copper, and lead. The acute criteria for mercury is exceeded only at the downstream location at the Highway 42 bridge. Mercury is of concern because of bioaccumulated effects. Zinc and nickel data were collected only at the 4H Camp location near Denham Springs, Louisiana, which has an exceedance ration of 69 percent for zinc and no exceedances for nickel.

As expected, the chronic exceedances at the five locations equalled or exceeded the acute criteria

exceedances. Of particular significance are the much higher exceedance ratios for the trace metal mercury.

Lake Maurepas and Pass Manchac

Lake Maurepas and Pass Manchac have been designated as effluent limited segments. Due to the close proximity of the three sampling stations data, many of the general water quality parameters are quite similar. However, significant differences in the computed means of some constituents likely result from the relative influence of periodic tidal inflow from Lake Pontchartrain to Lake Maurepas via Pass Manchac.

The mean temperatures at the three locations were close and ranged from 21.9°C at the U.S. Army Corps of Engineers, New Orleans District station to 21.3°C at the U.S. Geological Survey station. Temperatures which exceeded the maximum 32°C state standard were rare, but did occur at each of the three stations with the highest recorded temperature of 34.0°C being reported for the Pass Manchac station.

There were no DO violations at the USGS station location in the middle of Lake Maurepas. However, the USACE, NOD station in the center of Lake Maurepas had a percent exceedance value of 4. The Pass Manchac station had a 3% exceedance value. The mean DO levels, however, were well above the minimum 5.0 mg/l state standard with values of 7.6, 8.2, and 8.0 mg/l for the USACE, NOD, USGS, and Pass Manchac locations, respectively.

There are no violations at the USGS station for pH levels. The USACE, NOD station had one pH value below the state standard of 6.0 SU and 18 pH values above the maximum state standard of 8.5 SU. The Pass Manchac station had 3 pH values which fell below the minimum 6.5 SU state standard.

There was no available TDS data for the USGS station in the middle of Lake Maurepas. Although the mean TDS concentration at the Pass Manchac location was well within the maximum state standard of 3,000 mg/l, the station had an 8% exceedance ratio for TDS. There were no contraventions for chlorides and sulfates at the USGS

Station, whereas the Pass Manchac station had exceedance ratios of 14 and 12 for chlorides and sulfates, respectively. The exceedances at Pass Manchac most likely reflect the influence of brackish inflows from Lake Pontchartrain.

The logarithmic means of the bacteria data range from about 5 colonies/100 ml for the Lake Maurepas sampling location to about 41 colonies/100 ml for the Pass Manchac location. These values are well below the maximum 200 colones/100 ml state standard. The 90th percentile values are also well below the 400 colonies/100 ml state standard. Therefore, the data for these two locations imply no violations of the state standards.

There are relatively high acute criteria exceedance ratios for cadmium and copper. These cadmium exceedance ratios range from 29 percent in the middle of Lake Maurepas to 6 percent at the Pass Manchac station. The copper exceedance ratios range from 12 percent in the middle of Lake Maurepas to 54 percent at Pass Manchac. No exceedances were found for mercury and zinc at either location. The chronic freshwater criteria exceedances for cadmium, copper, lead, and mercury were relatively high. The Pass Manchac station exceedance ratio for mercury was extremely high at 94 percent.

Cultural Resources

There are 20 significant cultural resources within the economically benefited area, which includes portions of East Baton Rouge, Ascension, Livingston, St. Helena, and East Feleciana Parishes. Sixteen of the total are historic structures which are listed on the National Register of Historic Places. The remaining four are archeological sites. Site 16EBR51 is listed as eligible to the National Register; 16EBR6 is located within the boundaries of-a listed property; and sites 16AN39 and 16EBR35 were reported to the State Historic Preservation Officer as significant sites, potentially eligible to the National Register. In addition to these significant resources are 44 recorded archeological sites, fourteen of which have been tested to determine their significance. These range in age and function from Paleo-Indian Period lithic scatters and Archaic Period mound sites to nineteenth century

plantations, farmsteads, shipwrecks, landings, and sawmills. There are many potential sites in the study area (i.e., historic houses, early roads, and Civil War campsites) which may eventually receive formal site designation. Ten shipwrecks are recorded in the study area. Six vessels went down in the vicinity of Profit Island, one burned in Bayou Manchac, and three were lost in the Amite River. The Amite River was a major commercial waterway, from its mouth at Lake Maurepas to 100 miles above its Head of Navigation, from the eighteenth through early twentieth centuries. Bayou Manchac was seasonally navigable by smaller vessels throughout the eighteenth century until its closure to the Mississippi River in 1826. The navigability of the Comite River during the historic period has not been established. The State of Louisiana completed magnetometer survey of approximately 20.3 miles of the Amite River within the study area. Over 56 anomalies were recorded, with recommendations for future study. Some 8.3 miles of Bayou Manchac were subjected to exploratory magnetics, locating 19 anomalies. The Comite has not been magnetically surveyed. Data patterns identified by these studies predict an additional 20 sensitive areas on the Amite River between miles 25.75 and 54.0, and 17 sensitive areas on the upper Amite adjacent to acreage identified for possible mitigation use. Although both the Comite and Amite Rivers have been modified (either cleared and snagged or dredged), these magnetic surveys demonstrate that there is a substantial number of underwater resources requiring additional investigation in these formerly commercial waterways, and that prior channel modification, improvement or maintenance may damage and scatter underwater resources but can not be assumed to destroy them entirely.

Recreation Resources

This heavily forested area of southeastern Louisiana supports a large year-round resource base that offers a variety of popular recreational pursuits. The recreation study area is relatively rural with the exception of the Baton Rouge area. The existing recreational lands and facilities in the study area can be categorized into private, public, and commercial use types. The recreational market area include the parishes of Ascension, East Baton Rouge, East Feliciana, Iberville, Livingston,

Pointe Coupee, St. Helena, West Baton Rouge, and West Feliciana. Currently, within the nine parish recreational market area are one state park, one state wildlife management area, six state commemorative areas, and numerous tent and recreational vehicle campgrounds, boat launching facilities, picnic areas, hiking trails, hunting lands, scenic streams, and scenic roads. Recreational activities taking place in this rural environment have added emphasis on hunting and fishing. Other activities include boating, boat and bank fishing, water skiing, picnicking, jogging, horseback riding, nature trail walking, sight-seeing, and observing wildlife. Five scenic streams are present in the study area. Within the recreational market area, there are five scenic streams. Depending on the stream or segment of the stream, one may observe numerous canoes on a river in any summer weekend day. Some smaller streams are traveled less intensively, however, they are used to search the upper stretches for creek fishing and nature study.

The recreation inventory for this study was compiled from information obtained from the Louisiana Department of Culture, Recreation, and Tourism, Division of Outdoor Recreation, Office of State Parks, Louisiana State Comprehensive Outdoor Recreation Plan (SCORP), and field investigations. The inventory was compiled for all major recreational facilities and activities within the recreation market area. A summary of the facilities for the study area include 71 boat launching ramps, numerous fishing piers, 1495 picnic tables, 972 recreational vehicle camping spaces, 220 tent camping spaces, and 10 hiking trails.

Of the many heavily pursued recreational activities within the area, the most significant are hunting and fishing. Recreational fishing is by far the most popular and heavily pursued activity in the study area. In the recreation market area, 65,400 resident sport fishing licenses were issued in the 1986-1987 season. Most of the fishing that occurs is by boat. Boat use is reflected in the number of motorboat registrations (40,000) issued in 1986-1987 within the nine parish area. Results of the 1983 (SCORP) Demand Survey indicates boat fishing as the second most preferred activity of recreationists statewide. Hunting activities within the study area are as varied as

fishing activities. Hunting for small game is the most prevalent activity and a wide range of species and habitat types are available. Big game hunting for whitetail deer is relegated to the more productive habitats such as bottomland hardwood area. For the 1986-1987 hunting season, 49,000 resident hunting licenses were issued in the nine parish market area.

Overall, the primary users of the recreation resources in the study area are residents of southern Louisiana, and their primary recreational activity is hunting and fishing. The 1980 recreation survey conducted by the Louisiana Department of Culture, Recreation, and Tourism indicates that 81.7 percent of boat fishing activity occasions and 86.6 percent of the small game hunting activity occasions occur within 45 miles of the participant's residence.

Description of Flood Problem

Major Floods

Past storms that have caused heavy rainfall in the Amite River Basin have all shown similar characteristics. The typical storm produces heavy rainfall over a three to five day period. It is usually the result of a near-stationary cold front that moves across southeastern Louisiana and southern Mississippi with warm and humid air from the Gulf of Mexico moving inland ahead of the front. This condition produces very heavy thunderstorms and intense rainfall rates of one or more inches per hour. Coupled with higher than normal rainfall for several preceding months, the antecedent condition produces high stormwater runoff and the potential for extreme flooding.

The low-lying areas along the Amite and Comite River Systems have commonly experienced flooding during periods of heavy rainfall. Flood problems within the basin are caused by the excessive rainfall that results in headwater and backwater overflow in the lower reaches of the Amite River and the tributary streams in the vicinity of their confluence with the Amite River.

Historically, most of the flooding was confined to swampland and to rural, sparsely populated, largely wooded areas with only scattered agricultural usage. The Baton

Rouge metropolitan area has expanded further into the floodplain to accommodate population growth. The consequence of flooding in the floodplain is extremely large economic losses due to property damage.

Significant floods have occurred in the basin in 1921, 1928, 1942, 1947, 1953, 1957, 1962, 1964, 1973, 1977, 1979, and 1983. Rainfall amounts associated with the last four major floods are shown in Table 15. A summary of flow data for the last four major floods are given in Table 16.

TABLE 15

CUMULATIVE DAILY RAINFALL AT SELECTED STATIONS IN THE AMITE RIVER BASIN

STATION NAME	TOTAL RAINFALL (INCHES) YEAR			
	1973 MARCH 23-25	1977 APRIL 8-23	1979 APRIL 21-25	1983 APRIL 5-7
Baton Rouge	7.28	6.42	8.8	10.1
WSO AP Clinton	6.25	9.28	11.12	11.7
Clinton 5 SE	--	8.10	9.11	13.3
Denham Springs	--	--	11.23	9.0
Gonzales	--	12.84	12.54	7.1
Greenwell Springs	7.56	8.8	10.89	10.5
LSU Ben. Hur. Exp. Sta.	--	8.73	11.25	4.6
Zachary	--	8.88	9.36	10.6

Source: Climatogical data for Louisiana and Mississippi, National Climatological Center, 1977-1983

TABLE 16

PERTINENT FLOOD DATA ON THE MAJOR FLOODS IN AMITE RIVER BASIN
1973-1983

	YEAR			
	1973	1977	1979	1983
			(cfs)	
DARLINGTON GAGE				
Peak Discharge, cfs	62,100	76,400	43,400	63,300
Total Discharge, Acre-Feet	172,000	238,000	170,000	283,000
Recurrence Interval, Years*	10.0	18.0	5.0	8.0
Crest Elevation, Feet NGVD	166.0	167.57	165.06	166.10
DENHAM SPRINGS GAGE				
Peak Discharge, cfs	60,200	110,000	68,600	112,000
Total Discharge, Acre-Feet	363,000	528,000	495,000	469,000
Recurrence Interval, Years*	7.0	42.0	10.0	45.0
Crest Elevation, Feet NGVD	36.50	41.08	36.70	41.50

*Based on Stage-frequency relationship as of July 1989

SOURCE: Modified from Conceptual Stage Engineering and Economics Feasibility Report prepared by Brown and Bulter, Inc. for the Louisiana Department of Transportation and Development, January 1984, and U.S. Geological Survey

Other historical storms of interest are the storms of August 2, 1983, and Hurricane Juan from October 26-31, 1985. The August 2, 1983, storm caused severe localized precipitation at various locations as shown below:

GAGE LOCATION	PRECIPITATION (INCHES)
Baton Rouge, Woodlawn	14.43
Denham Springs	13.80
Baton Rouge, Central	11.42
Watson, 3 ENE (3 miles East Northeast of Watson)	11.06
Baton Rouge Airport	1.11 +5.11 on 8/1/83
Clinton	4.80
Clinton, 5 SE	6.91
Oaknolia	6.31
Zachary, 2 ESE	4.72

This rain gage data indicates a very localized summer thunderstorm east of Baton Rouge. This localized storm produced a peak discharge of 36,800 cfs at the Amite River near Denham Springs gage. The high water mark elevation reached 30.67 feet NGVD. This is only 1.67 feet above the flood stage elevation of 29.0 feet NGVD at the Denham Springs gage set by the National Weather Service.

Hurricane Juan caused much damage throughout the state of Louisiana creating havoc for the agricultural industry in south Louisiana and flooding homes and businesses. A large area was flooded on the Amite River below Interstate 12 at Bayou Manchac, Port Vincent,and French Settlement. Rain gage amounts for the period October 26-31 1985, are shown below:

GAGE	PRECIPITATION (INCHES)
Baton Rouge Airport	5.4
Denham Springs	4.9
Greenwell Springs	8.6
Pine Grove Fire Tower	10.1
Clinton	10.3
Clinton, 5 SE	11.2
Liberty, 1 WNW	6.6

Heavy winds from tropical storm Juan produced tides in Lake Maurepas six to eight feet higher than normal. The Amite River near Denham Springs gage had only a peak discharge of 43,900 cfs on October 31, 1985, with a high water mark elevation of 32.19 feet NGVD only 3.19 feet above the flood stage of 29.0 feet NGVD. The Darlington gage had a peak discharge of 17,500 cfs on October 30, 1985, during Hurricane Juan with a stage of 161.73 feet NGVD.

The rainfall and rainfall intensity was less than that of the August 1983 event, yet caused somewhat more flooding along the Amite River from Denham Springs to Lake Maurepas. This was probably due to the wind and tidal effects of the hurricane passing in the vicinity of Lake Maurepas.

Flood Damages

Flood damages from the 1973 flood are estimated at $2.1 million. About 105,000 acres were flooded. During the 1977 flood, approximately 164,000 acres were flooded. Over 4,000 structures were flooded and the associated damages totaled nearly $78.0 million. This figure does not include business losses experienced by commercial establishments due to closings, the loss of wages from lost work time, agricultural losses, or damages to roads, bridges, and utilities. The 1979 flood was not as widespread as the 1977 flood. Flood damages associated with the flood were estimated at over $9.0 million. The 1983 flood was the flood of record in most of the basin. Flood stages reached the highest level at 8 recorded locations along the Amite River and its tributaries. Over 357,000 acres were inundated in East Baton Rouge, Livingston, Iberville, Ascension, St. James, and St. John the Baptist Parishes. About 5,300 homes and 200 businesses were flooded. Flood damages were estimated to be about $172 million. Of the $172 million, $113 million or 66 percent were attributable to flood damages in urban areas. Flood damages in the Comite River subbasin were estimated at $48,000,000. Flood problems and damages are described by parish, in the following paragraphs.

St. Helena and East Feliciana Parishes and Mississippi Counties. These parishes and counties are located in the upper part of the Amite River Basin. They are mostly upland forest and agricultural lands and are sparsely populated. Flooded areas adjacent to the Amite River and tributaries are mostly wooded lands and, to a lesser extent, agricultural lands. Flood damages from past floods have been determined to be insignificant and have not been estimated.

East Baton Rouge. Flood problems in northern and northeastern portion of the parish are due to headwater overflows from the Comite River, Cypress Bayou, White Bayou, Sandy Creek, Beaver Bayou, South Canal, Baker Canal, Monte Sano Bayou, and tributaries of these streams. Overflow from backwater flooding creates problems along Hurricane Creek, lower Comite River, and lower reaches of its tributaries streams. Flood problems in the southern and southeastern portion of the parish are caused by headwater overflow from Ward Creek and Tributaries, the Amite River, Clay Cut Bayou, Jones Creek and tributaries, and Bayou Fountain and tributaries. Backwater flooding occurs along lower Ward Creek from Bayou Manchac and the Amite River. The area along lower Clay Cut Bayou, Honey Cut Bayou from the Amite River, and Jones Creek is subject to backwater flooding from the Amite River. Backwater flooding occurs in the lower reaches of Bayou Fountain from Bayou Manchac and the Amite River.

During the April 1977 flood, about 25,000 acres of land were inundated in the Baton Rouge area. A total of 1,500 urban residences and some of the business establishments were flooded. Inundated structures were flooded in depth over the floor from a few inches to about eight feet. Inundated structures were flooded from a few hours to several days. Damages to structures and contents were estimated at about $20.7 million. Total damages in the parish were $24.0 million. Limited flooding occurred in April 1979, causing an estimated $1.4 million in damages to the Baton Rouge area.

East Baton Rouge Parish was severely flooded in 1983 along the Amite and Comite Rivers, Clay Cut Bayou, Cypress Bayou, Beaver Bayou, Sandy Creek and White Bayou. About 55,000 acres of land was flooded, and a total of 1558 urban

residences, 20 rural residences, and 37 urban business establishments were damaged. Flood damages were estimated at $65.2 million. About 75 percent of the damages occurred along the Comite River and tributary streams. Flooding up to eight feet above the first floor was reported with inundation of structures lasting from a few hours to several days. Some streets and yards were reported flooded for a longer period of time. Agricultural flooding occurred; however, much of the land was fallow at the time of the flood. About 10,000 acres of improved pasture flooded. The pasture was damaged, but the water did not stay long enough to kill the grass. Approximately 30 tons of hay were reported lost.

Livingston Parish. Flooding in Livingston Parish is caused by headwater overflow of the Amite River, Cotton Creek, and Long Slash Branch. Backwater overflows occur along the lower reaches of Cotton Creek, Grays Creek, and Long Slash Branch, Beaver Creek, Bayou Barbary, and Blind River. Widespread flooding occurred in Livingston Parish in 1977. About 71,000 acres of land was flooded. The residents of more than 700 houses and 70 business establishments incurred an estimated $15.0 million in damages. Inundated structures were flooded over the floor from a few inches in depth to about three feet. Limited flooding also occurred in April 1979, causing $350,000 in damages in the Denham Springs area. In 1983, about 137,00 acres were flooded in Livingston Parish. Total damages were estimated at $83.8 million. Approximately 1,400 urban homes, 1,475 rural residences, and 117 business establishments were reported flooded. Up to eight feet of water inundated some homes. About 9,000 acres of improved pasture was flooded. Most of the pasture was damaged, but not lost. Approximately 133 tons of hay, 125 cows, and 28 hogs were lost.

Ascension Parish. Flooding in Ascension Parish is caused by headwater overflow of the Amite River, New River, Black Bayou, and Bayous Narcisse, Francois and Conway. Backwater flooding occurs along the lower reaches of the streams and along Bayou Manchac. During the April 1977 flood, about 68,000 acres were flooded with damages estimated at $12.1 million. About 350 structures were inundated in the Gonzales area. Inundated structures were flooded in depth over floor from a few inches to about four feet. The flood damages in Gonzales were estimated to exceed $2.0 million.

Developed areas in low-lying portions of Ascension Parish, Brittany, Duplessis, St. Amant, Little Prairie, Hope Villa, and Sorrento, had flooded in depth over floor from a few inches to eight feet for a duration of one to four days. The 1979 flood inundated 275 structures in the Gonzales area causing nearly $1 million in flood damages. Depths of flooding varied from a few inches to about three feet.

Most of the flooding in Ascension Parish during the 1983 flood occurred in the northeastern and extreme eastern portions of the parish and was caused by backwater from Bayou Manchac and its tributarires. In the eastern part of the parish, severe flooding occurred along New River and Bayou Francois as a result of backwater flooding. About 78,000 acres were flooded with flood damages estimated at $20.4 million. A total of 107 urban residences, 686 rural residences, and 18 business establishments were reported flooded. Approximately 5,000 acres of pastures were flooded. On about 2,000 acres, the pasture was totally lost, with 3,000 acres sustaining 5-percent losses. An estimated 150 tons of hay was lost. A reported 150 acres of vegetable crops, primarily tomatoes and cabbage, were lost.

Iberville Parish. Flooding in this parish occurs as a result of backwater overflow from Bayou Manchac, Bayou Paul, and Alligator Bayou. Most of the areas flooded are natural wetlands as well as some upland wooded areas. No substantial flood damages were reported. About 13,000 acres were flooded during the 1983 flood. Damages were estimated at about $26,000 to structures, roads, crops, etc.

St. James Parish. Flooding in St. James Parish occurs primarily in the wetlands. During the 1983 flood, 38,700 acres were flooded, but no substantial flood damages were reported. Flood damages for the 1983 flood are estimated at $23,100 to structures, roads, crops, etc.

St. John the Baptist Parish. In 1983 flooding in St. John the Baptist Parish occurs primarily in wetland areas south and west of Lake Maurepas. Flood damages were estimated at $180,000, about 45 percent was attributable to damages in urban areas. Due to backwater flooding from the Reserve Relief Canal, 10 homes were flooded. No business establishments were reported flooded. Sixty-five acres of

vegetable crops were lost. Crops lost included cucumbers, squash, cabbages, and collard and mustard greens.

Flood Damage Potential

A portion of the Amite River Basin was subdivided into 60 hydrologic subbasins. These subbasins are shown on Plate 10. The hydrologic and hydraulic analysis, land use, and the economic analyses were conducted on a subbasin level. This allowed damage centers to be more clearly identified as well as the causes of flooding. The flood damage potential was evaluated for each subbasin. This potential is a indication of the level of flood protection that can be economically justified. Table 17 shows the number of structures located in the various floodplains by subbasin. The existing average annual damages by subbasin is also shown.

FUTURE CONDITIONS (IF NO FEDERAL ACTION IS TAKEN)

The most probable future conditions if no Federal action is taken are determined by projection conditions that will prevail in the area over the planning period 1997 to 2047. The conditions described are based on available information. This scenario serves as the base condition to which all alternative plans were composed to assess the effects of each plan. For resources not described in this section, future conditions are not expected to be significantly different from existing conditions.

Economy and Human Resources

Based on historical trends, population growth in the area would continue. The exact locations of this growth would be influenced by many factors, including the availability of land throughout the area, construction costs, flood protection, environmental concerns, differences in lifestyles, and the proximity of housing to the work place and commercial centers. The economic potential of the area appears favorable inspite of recent declines in petrochemical industries. The area's mild climate, natural resources, port activities and state government operations are major factors that would encourage growth. The population is expected to increase by about 246,000 people or 45 percent by the year 2047.

The growth rate between 1986 and 2047 is expected to average 0.6 percent annually. Table 18 shows the projected population for the study area. Tables 19 and 20 display pertinent data on anticipated population, earnings, and employment for the Baton Rouge Standard Metropolitan.

Statistical Area (SMSA). The Baton Rouge SMSA includes the parishes of East Baton Rouge, Ascension, Livingston, and West Baton Rouge.

Future Land Use

The projection of future land use was based upon three principles: knowledge of planned activities in the study area, awareness of constraints upon development, and the extension of historical trends. The methodology used to project future land use is contained in Appendix D. The land use projections were made for the U.S. Army Corps of Engineers by the Louisiana State Planning Office. Future land use was not projected for East Feliciana, and St. Helena Parishes, Amite, Franklin, and Wilkinson counties. These parishes and counties have historically been rural and experienced very little growth. This same trend is expected to occur in the future. Future land use was projected for that portion of the Amite River Basin where substantial urban growth is envisioned to occur. Substantial urban growth is expected to occur in East Baton Rouge, Livingston, Ascension, and Iberville Parishes.

The future land use projections indicate a strong growth trend for the Baton Rouge urbanized area toward the east southeast. The transportation facility provided by Interstate 10 and Airline Highway is undoubtedly a major factor in directionality of this growth. A second area of growth toward the east along Interstate 12 is also highly significant. Growth to the north and northeast is weak, though the Zachary-Baker area appears to have the strongest history of development in the sector. Future growth patterns described above are supported by the East Baton Rouge City Parish Planning Commission.

East Baton Rouge Parish has plenty of available land for expansion in all directions of the city of Baton Rouge except the west. Factors such as highway improvements,

TABLE 17
NUMBER OF STRUCTURES IN THE VARIOUS
FLOODPLAINS IN THE AMITE RIVER BASIN
— WITHOUT PROJECT

SUB-BASIN NUMBER	SUB-BASIN NAME	CATAGORY OF STRUCTURE	0-10 YEAR	10-25 YEAR	25-50 YEAR	50-100 YEAR	100-500 YEAR	ABOVE 500 YEAR
2	UPPER CYPRESS BAYOU	1-STORY	13	1	17	14	24	325
		2-STORY	3	0	0	0	1	30
		MOBILE HOME	1	0	1	0	2	35
		COMMERCIAL	1	0	3	1	4	57
		TOTAL	18	1	21	15	31	447
3	UPPER WHITE BAYOU	1-STORY	152	30	14	11	30	21
		2-STORY	11	0	2	0	1	0
		MOBILE HOME	61	2	4	0	12	13
		COMMERCIAL	29	10	3	2	8	0
		TOTAL	253	42	23	13	51	34
7	UPPER CYPRESS & WHITE BAYOU	1-STORY	11	19	3	7	9	28
		2-STORY	2	0	0	0	0	0
		MOBILE HOME	7	2	2	15	20	41
		COMMERCIAL	4	14	2	1	11	16
		TOTAL	24	35	7	23	40	85
8	COMITE RIVER	1-STORY	237	316	222	96	23	36
		2-STORY	19	15	8	2	6	1
		MOBILE HOME	35	112	101	63	15	
		COMMERCIAL	12	15	18	7	4	7
		TOTAL	303	458	349	168	48	44
10	LOWER CYPRESS BAYOU	1-STORY	84	71	77	328	259	1045
		2-STORY	0	0	4	11	4	24
		MOBILE HOME	0	35	6	20	39	141
		COMMERCIAL	3	12	6	9	23	120
		TOTAL	87	118	93	368	325	1330
12	LOWER WHITE BAYOU	1-STORY	395	154	72	44	24	197
		2-STORY	8	7	4	1	0	9
		MOBILE HOME	7	0	1	13	4	18
		COMMERCIAL	27	6	19	2	1	27
		TOTAL	437	167	96	60	29	251

TABLE 17 (CONTINUED)

NUMBER OF STRUCTURES IN THE VARIOUS FLOODPLAINS IN THE AMITE RIVER BASIN — WITHOUT PROJECT

SUB-BASIN NUMBER	SUB-BASIN NAME	CATAGORY OF STRUCTURE	0-10 YEAR	10-25 YEAR	25-50 YEAR	50-100 YEAR	100-500 YEAR	ABOVE 500 YEAR
13	BLACKWATER BAYOU	1-STORY	282	140	306	125	49	53
		2-STORY	28	5	6	7	2	2
		MOBILE HOME	12	24	23	15	54	33
		COMMERCIAL	9	6	17	10	5	9
		TOTAL	331	175	352	157	110	97
14	BEAVER BAYOU	1-STORY	509	88	84	27	94	445
		2-STORY	21	11	1	1	4	15
		MOBILE HOME	46	13	33	8	19	133
		COMMERCIAL	95	6	3	4	6	133
		TOTAL	671	118	121	40	123	726
16	HURRICANE CREEK	1-STORY	123	246	248	301	518	1379
		2-STORY	15	27	4	3	4	8
		MOBILE HOME	0	38	56	0	0	26
		COMMERCIAL	82	72	51	61	4	27
		TOTAL	220	383	359	365	526	1440
17	COMITE RIVER	1-STORY	2	11	9	4	41	32
		2-STORY	0	0	1	0	0	0
		MOBILE HOME	0	1	3	0	1	13
		COMMERCIAL	1	7	2	0	2	1
		TOTAL	3	19	15	4	44	46
13	COMITE RIVER & SHOE CREEK	1-STORY	109	184	239	244	126	122
		2-STORY	14	9	8	8	1	10
		MOBILE HOME	0	8	6	15	2	42
		COMMERCIAL	1	13	25	4	6	4
		TOTAL	124	214	278	271	135	178
19	COMITE RIVER & DRAUGHAN CREEK	1-STORY	23	87	95	248	174	218
		2-STORY	1	2	3	13	21	44
		MOBILE HOME	0	1	0	0	11	14
		COMMERCIAL	2	4	1	14	8	34
		TOTAL	26	94	99	275	214	310

TABLE 17 (CONTINUED)

NUMBER OF STRUCTURES IN THE VARIOUS FLOODPLAINS IN THE AMITE RIVER BASIN — WITHOUT PROJECT

SUB-BASIN NUMBER	SUB-BASIN NAME	CATAGORY OF STRUCTURE	0-10 YEAR	10-25 YEAR	25-50 YEAR	50-100 YEAR	100-500 YEAR	ABOVE 500 YEAR
22	JONES CREEK	1-STORY	90	150	72	62	319	810
		2-STORY	14	29	7	17	70	166
		MOBILE HOME	2	1	1	0	1	4
		COMMERCIAL	43	36	29	23	65	184
		TOTAL	149	216	109	102	455	1164
29	BAYOU FOUNTAIN	1-STORY	32	99	2	200	22	139
		2-STORY	9	57	3	128	24	111
		MOBILE HOME	0	0	0	1	0	5
		COMMERCIAL	23	24	0	28	41	99
		TOTAL	64	180	5	357	87	354
31	CLAYCUT BAYOU	1-STORY	121	377	86	251	97	108
		2-STORY	19	21	13	16	17	21
		MOBILE HOME	4	15	47	5	4	40
		COMMERCIAL	31	5	2	19	20	28
		TOTAL	175	418	148	291	138	197
32	LOWER WARD'S CREEK	1-STORY	90	10	64	19	31	123
		2-STORY	4	4	2	1	1	15
		MOBILE HOME	4	0	0	0	1	71
		COMMERCIAL	24	21	15	0	9	10
		TOTAL	122	35	81	20	41	219
42	AMITE RIVER	1-STORY	2	12	12	7	46	101
		2-STORY	0	4	7	1	1	6
		MOBILE HOME	0	0	0	0	9	113
		COMMERCIAL	8	0	2	2	2	14
		TOTAL	10	16	21	10	57	234
43	MUDDY CREEK	1-STORY	5	26	36	62	43	11
		2-STORY	3	6	0	0	3	3
		MOBILE HOME	8	6	5	25	17	44
		COMMERCIAL	0	0	1	2	9	1
		TOTAL	16	38	42	89	72	59

TABLE 17 (CONTINUED)
NUMBER OF STRUCTURES IN THE VARIOUS
FLOODPLAINS IN THE AMITE RIVER BASIN
-- WITHOUT PROJECT

SUB-BASIN NUMBER	SUB-BASIN NAME	CATAGORY OF STRUCTURE	0-10 YEAR	10-25 YEAR	25-50 YEAR	50-100 YEAR	100-500 YEAR	ABOVE 500 YEAR
44	AMITE RIVER	1-STORY	261	295	102	107	71	103
		2-STORY	34	16	12	3	4	12
		MOBILE HOME	55	141	77	104	105	107
		COMMERCIAL	13	11	12	13	19	11
		TOTAL	363	463	203	227	199	233
45	COLYELL CREEK	1-STORY	128	10	13	3	7	196
		2-STORY	9	2	1	0	0	22
		MOBILE HOME	36	4	6	9	0	178
		COMMERCIAL	6	2	0	0	0	28
		TOTAL	179	18	20	12	7	424
46	GRAY'S CREEK	1-STORY	131	163	115	87	65	84
		2-STORY	15	10	7	5	4	6
		MOBILE HOME	40	49	79	81	54	82
		COMMERCIAL	8	7	23	13	5	8
		TOTAL	194	229	224	186	128	182
47	AMITE RIVER	1-STORY	85	84	208	124	134	151
		2-STORY	18	23	27	5	8	4
		MOBILE HOME	13	1	12	15	42	19
		COMMERCIAL	2	2	34	7	27	73
		TOTAL	118	110	281	151	211	247
48	HONEY CUT	1-STORY	11	67	232	102	193	16
		2-STORY	0	0	1	14	24	5
		MOBILE HOME	0	0	1	3	3	2
		COMMERCIAL	0	1	54	18	48	4
		TOTAL	11	68	288	137	268	27
49	MILLER CANAL	1-STORY	253	2	40	19	44	154
		2-STORY	24	0	5	4	2	3
		MOBILE HOME	141	1	38	68	96	3
		COMMERCIAL	38	1	41	29	25	17
		TOTAL	456	4	124	120	167	177

TABLE 17 (CONTINUED)

NUMBER OF STRUCTURES IN THE VARIOUS
FLOODPLAINS IN THE AMITE RIVER BASIN
— WITHOUT PROJECT

SUB-BASIN NUMBER	SUB-BASIN NAME	CATAGORY OF STRUCTURE	0-10 YEAR	10-25 YEAR	25-50 YEAR	50-100 YEAR	100-500 YEAR	ABOVE 500 YEAR
50	UPPER GRAY'S CREEK	1-STORY	55	21	0	118	104	302
		2-STORY	3	0	0	3	4	2
		MOBILE HOME	22	6	0	38	53	16
		COMMERCIAL	4	1	0	3	57	100
		TOTAL	84	28	0	162	218	420
51	UPPER AMITE RIVER	1-STORY	36	121	179	251	365	85
		2-STORY	1	4	12	10	20	4
		MOBILE HOME	11	22	90	76	81	7
		COMMERCIAL	10	16	30	31	68	23
		TOTAL	58	163	311	368	534	119
64	BAYOU MANCHAC	1-STORY	38	40	42	10	30	107
		2-STORY	11	3	4	4	5	15
		MOBILE HOME	23	14	49	11	26	31
		COMMERCIAL	2	7	0	1	4	3
		TOTAL	74	64	95	26	65	156
TOTALS FOR ALL BASINS		1-STORY	3,278	2,824	2,589	2,871	2,942	6,391
		2-STORY	286	255	142	257	231	540
		MOBILE HOME	528	496	641	585	670	1,231
		COMMERCIAL	478	299	393	304	480	1,038
		TOTAL	4,570	3,874	3,765	4,017	4,323	9,200

TABLE 18

PROJECTED POPULATION IN THE STUDY AREA

PARISH	1970	1980	1990	2000	2005	2015	2035	2040	2047
Ascension	37,086	50,068	58,214	67,000	70,000	75,000	82,000	84,000	87,000
East Baton Rouge	285,167	366,191	380,105	442,000	453,600	489,700	530,000	541,000	557,000
East Feliciana	17,657	19,015	19,211	22,600	23,000	24,400	27,000	27,700	28,700
Livingston	36,511	58,806	70,526	90,000	96,000	104,000	112,000	114,000	117,000
St. Helena	9,937	9,827	9,874	10,700	11,100	12,000	13,000	13,200	13,600
TOTALS	386,358	503,925	537,930	632,300	653,700	705,100	764,000	779,900	803,300

SOURCE: U.S. Army Corps of Engineers, New Orleans District

TABLE 19

POPULATION, PERSONAL INCOME, AND EARNINGS,

1969-1983, AND PROJECTED, 1990-2035

FOR BATON ROUGE, LA (MSA)

	1969	1973	1978	1983	1990	1995	2000	2005	2015	2035
Population as of July 1 (thousands)	371.0	407.4	466.8	531.1	572.3	599.5	622.0	643.1	693.2	751.0
				Millions of 1972 dollars						
Total personal income (place of residence)	1,350.8	1,601.4	2,337.4	2,798.1	3,573.4	4,065.0	4,532.4	4,996.3	5,849.0	7,549.9
By place of work										
Total earnings	1,106.2	1,282.1	1,859.9	2,005.9	2,662.9	3,049.7	3,430.0	3,795.0	4,389.9	5,444.7
Farm	11.7	17.5	8.7	9.5	12.0	11.3	11.0	10.9	11.1	12.2
Nonfarm	1,094.5	1,264.6	1,851.3	1,996.3	2,650.9	3,038.3	3,419.0	3,784.2	4,378.9	5,432.5
Private	885.1	995.5	1,517.0	1,599.7	2,161.8	2,489.7	2,807.5	3,107.2	3,597.3	4,471.3
Agricultural services, forestry, fisheries, and other	(D)	(D)	(D)	3.9	5.7	6.7	7.5	8.5	9.8	12.1
Mining	(D)	(D)	(D)	11.1	13.7	14.8	15.7	16.7	17.8	20.0
Construction	(D)	159.1	(D)	238.8	288.6	296.5	309.6	324.2	344.8	390.1
Manufacturing	262.0	282.8	378.1	377.0	509.9	581.3	646.3	705.9	816.3	1,016.6
Nondurable goods	227.5	244.1	325.8	333.1	434.7	487.0	534.7	578.2	661.9	815.1
Durable goods	34.5	38.6	52.3	43.9	75.2	94.2	111.6	127.7	154.3	201.5
Transportation and public utilities	64.1	82.0	116.9	133.5	182.0	210.6	241.4	272.1	322.4	409.1
Wholesale trade	(D)	(D)	(D)	122.4	169.4	195.3	221.4	248.5	290.3	359.1
Retail trade	117.5	138.5	194.4	213.5	267.0	305.5	344.4	378.9	430.7	538.0
Finance, insurance, and real estate	52.2	68.3	97.6	118.2	169.4	200.3	229.1	255.4	295.5	363.6
Service	(D)	(D)	(D)	381.3	556.1	678.7	792.1	896.9	1,069.6	1,362.7
Government and government enterprises	209.3	269.1	334.3	396.7	489.1	548.7	611.5	677.0	781.6	961.2
Federal, civilian	15.8	19.3	20.9	23.7	27.4	30.0	32.9	36.2	41.5	51.6
Federal, military	2.8	3.9	4.2	6.0	6.6	7.0	7.3	7.7	8.5	10.4
State and local	190.7	245.9	309.2	367.0	455.1	511.7	571.3	633.1	731.6	899.2

SOURCE: 1985 OBERS BEA REGIONAL PROJECTIONS, Volume 2, U.S. Department of Commerce, Bureau of Economic Analysis

TABLE 20

EMPLOYMENT

1969-1983, AND PROJECTED, 1990-2035

FOR BATON ROUGE, LA (MSA)

	1969	1973	1978	1983	1990	1995	2000	2005	2015	2035
Total employment	145.1	161.0	211.1	229.5	274.4	300.2	322.6	340.7	358.5	367.2
Farm	3.8	3.5	2.8	2.5	2.5	2.5	2.5	2.5	2.4	2.3
Nonfarm	141.3	157.6	208.4	227.0	271.9	297.6	320.1	338.3	356.1	364.9
Private	107.8	117.9	160.8	173.7	213.8	236.8	256.9	272.9	289.6	299.4
Agricultural services, forestry, fisheries, and other	(D)	(D)	(D)	.8	1.1	1.3	1.4	1.5	1.5	1.5
Mining	(D)	(D)	(D)	1.0	.9	.9	.9	.9	.8	.7
Construction	(D)	16.5	(D)	22.4	25.2	25.3	25.7	26.2	26.2	25.1
Manufacturing	21.7	21.5	24.7	22.4	26.0	27.6	28.8	29.7	30.6	30.9
Nondurable goods	17.8	17.6	19.8	18.5	20.4	21.2	21.9	22.3	22.8	22.8
Durable goods	3.9	4.0	4.9	4.0	5.6	6.3	6.9	7.3	7.7	8.0
Transportation and public utilities	7.3	8.0	10.0	10.6	12.3	13.2	14.2	15.0	15.9	16.2
Wholesale trade	(D)	(D)	(D)	11.4	14.4	16.0	17.3	18.6	19.9	20.8
Retail trade	21.9	25.1	35.0	40.9	50.6	56.9	62.6	67.2	72.0	75.5
Finance, insurance, and real estate	6.1	8.4	11.4	13.6	17.3	19.4	21.2	22.6	23.9	24.4
Service	(D)	(D)	(D)	50.1	66.0	76.3	84.8	91.3	96.8	104.3
Government and government enterprises	33.5	39.6	47.6	53.7	58.1	60.8	63.2	65.3	66.5	65.5
Federal, civilian	1.5	1.6	1.6	1.9	2.0	2.0	2.1	2.2	2.2	2.3
Federal, military	1.9	2.0	2.1	2.4	2.4	2.4	2.4	2.4	2.4	2.4
State and local	30.0	36.0	43.9	49.5	53.8	56.4	58.7	60.8	61.9	60.9

SOURCE: 1985 OBERS BEA REGIONAL PROJECTIONS, Volume 2, U.S. Department of Commerce, Bureau of Economic Analysis

changes in attitudes toward certain areas, and the location of major employers could influence variances from the patterns predicted by the historical trends used for the projections in the study area.

For the purpose of projecting future land use, the area was grouped into eight regions. The region followed subbasin boundaries and are shown in Plate 11. Table 21 summarizes future land use by regions. The northwest region is an area of generally slow growth. In the time period preceding the period of record for this study, the area experienced a more rapid expansion due to the "urban retreat" of many of the blue collar workers from the industrial facilities of north Baton Rouge. These facilities have decreased employment in recent years and the growth of the area has correspondingly declined. Within the study period most of the growth recorded occurred in the area around Zachary, Louisiana. This is probably due, in part, to the influx of workers for the construction of the River Bend Nuclear Generator several miles to the north. Subbasin 1, located to the west of Zachary is the most rapidly growing area. In summary, the northwest region is not projected to be a rapid growth area in the future. About 7.8% of the total growth in the study area is slated for this area which makes up over 13% of the total acreage in the study area.

The northeast is an area of very slow growth. Transportation routes to the area are not well developed and the area is quite a distance from major employers. Much of the land in subbasins 55, 56, 57, and 58 is within the actual valley of the Amite River and is ill-suited for development. The remaining subbasin, 53, is projected for slow growth which might increase upon the improvement of transportation to the area. This area is not likely to contribute greatly to the problems associated with development within the foreseeable future. The entire area is slated for only about 1.5% of the growth for the study area while it covers about 12% of the total area.

This central region, located to the northeast of urbanized Baton Rouge, is likely to experience moderate growth in the future. Though transportation routes are inadequate, the area is near enough to the urbanized area

Table 21

FUTURE LAND USE BY GROWTH REGIONS, 1978-2047
(Acres)

NORTHWEST REGION

YEAR	RESIDENTIAL	COMMERCIAL	INDUSTRIAL	AGRICULTURE	FOREST	TRANSITION	WATER, WET LANDS, QUARRY	TOTAL
1978	14671	1095	771	56357	28603	969	365	102831
1985	16158	1356	882	54718	28896	251	572	102833
1990	16653	1404	962	54286	28872	86	572	102835
1995	17251	1459	1052	53714	28743	43	572	102832
1996	17370	1469	1069	53599	28717	34	572	102831
1997	17490	1480	1087	53485	28691	26	572	102831
2000	17848	1513	1141	53141	28614	0	572	102829
2007	18602	1587	1266	52353	28452	0	572	102832
2010	18925	1619	1320	52015	28382	0	572	102833
2017	19637	1686	1445	51271	28222	0	572	102834
2020	19942	1715	1499	50952	28154	0	572	102834
2027	20527	1771	1624	50326	28013	0	572	102833
2030	20777	1795	1678	50058	27952	0	572	102832
2037	21387	1852	1803	49415	27803	0	572	102833
2040	21648	1877	1857	49140	27739	0	572	102833
2047	22258	1934	1982	48497	27590	0	572	102834

NORTHEAST REGION

YEAR	RESIDENTIAL	COMMERCIAL	INDUSTRIAL	AGRICULTURE	FOREST	TRANSITION	WATER, WET LANDS, QUARRY	TOTAL
1978	1995	258	196	28666	53599	110	8551	93375
1985	2325	284	196	28104	53157	47	9261	93374
1990	2421	287	196	28055	53106	47	9261	93373
1995	2538	291	196	27996	53046	47	9261	93374
1996	2561	292	196	27984	53033	47	9261	93374
1997	2584	293	196	27972	53021	47	9261	93374
2000	2654	295	196	27937	52985	47	9261	93375
2007	2800	299	196	27864	52907	47	9261	93374
2010	2862	301	196	27832	52874	47	9261	93373
2017	3001	306	196	27761	52801	47	9261	93373
2020	3060	308	196	27731	52770	47	9261	93373
2027	3173	312	196	27674	52711	47	9261	93373
2030	3222	313	196	27649	52685	47	9261	93373
2037	3340	317	196	27589	52623	47	9261	93373
2040	3391	319	196	27563	52596	47	9261	93373
2047	3509	323	196	27503	52534	47	9261	93373

Table 21 (Continued)

FUTURE LAND USE BY GROWTH REGIONS, 1978-2047

(Acres)

ASCENSION REGION

YEAR	RESIDENTIAL	COMMERCIAL	INDUSTRIAL	AGRICULTURE	FOREST	TRANSITION	WATER, WET LANDS, QUARRY	TOTAL
1978	8092	1249	4970	42889	34668	287	7774	99929
1985	10334	1939	5320	42154	32206	287	7686	99926
1990	10999	2101	5570	41550	32022	0	7686	99928
1995	11781	2289	5821	40667	31684	0	7686	99926
1996	11937	2326	5871	40490	31616	0	7686	99926
1997	12093	2364	5921	40314	31548	0	7686	99925
2000	12562	2476	6071	39784	31345	0	7686	99924
2007	13557	2715	6422	38658	30886	0	7686	99924
2010	13983	2818	6572	38176	30689	0	7686	99924
2017	14924	3043	6923	37116	30231	0	7686	99923
2020	15327	3139	7073	36662	30035	0	7686	99922
2027	16098	3325	7424	35764	29623	0	7686	99921
2030	16429	3405	7574	35379	29447	0	7686	99920
2037	17233	3600	7925	34464	29016	0	7686	99923
2040	17577	3683	8075	34072	28831	0	7686	99924
2047	18381	3878	8426	33157	28400	0	7686	99927

LOWER BASIN REGION

YEAR	RESIDENTIAL	COMMERCIAL	INDUSTRIAL	AGRICULTURE	FOREST	TRANSITION	WATER, WET LANDS, QUARRY	TOTAL
1978	5136	93	907	15339	32689	235	111088	165487
1985	6845	137	915	15257	31595	145	110596	165490
1990	7351	149	921	15072	31383	17	110596	165489
1995	7948	164	927	14796	31052	9	110596	165490
1996	8067	166	928	14740	30985	7	110596	165490
1997	8186	169	929	14685	30919	5	110596	165490
2000	8544	178	933	14519	30720	0	110596	165490
2007	9304	196	941	14156	30297	0	110596	165490
2010	9629	204	945	14001	30115	0	110596	165490
2017	10344	222	953	13664	29711	0	110596	165490
2020	10651	229	957	13519	29538	0	110596	165490
2027	11240	243	965	13244	29202	0	110596	165491
2030	11493	249	969	13126	29058	0	110596	165491
2037	12105	264	977	12843	28706	0	110596	165491
2040	12367	270	981	12722	28555	0	110596	165491
2047	12979	285	989	12439	28203	0	110596	165491

Table 21 (Continued)

FUTURE LAND USE BY GROWTH REGIONS, 1978-2047
(Acres)

SOUTHERN REGION

YEAR	RESIDENTIAL	COMMERCIAL	INDUSTRIAL	AGRICULTURE	FOREST	TRANSITION	WATER, WET LANDS, QUARRY	TOTAL
1978	8212	1214	1217	23998	14105	1171	4299	54216
1985	14804	1479	1738	19821	11011	1151	4212	54216
1990	16752	2035	2110	18553	10553	0	4212	54215
1995	19042	2688	2482	16063	9729	0	4212	54216
1996	19500	2818	2556	15565	9564	0	4212	54216
1997	19958	2949	2631	15067	9399	0	4212	54216
2000	21332	3340	2854	13573	8905	0	4212	54216
2007	24244	4172	3375	10666	7549	0	4212	54217
2010	25492	4528	3598	9420	6968	0	4212	54218
2017	27689	4954	4119	7302	5942	0	4212	54217
2020	28631	5136	4342	6394	5502	0	4212	54217
2027	29926	5390	4863	5037	4789	0	4212	54217
2030	30481	5499	5086	4456	4483	0	4212	54217
2037	31699	5745	5607	3711	3243	0	4212	54216
2040	32221	5850	5830	3392	2711	0	4212	54216
2047	33439	6096	6351	2647	1471	0	4212	54215

LIVINGSTON REGION

YEAR	RESIDENTIAL	COMMERCIAL	INDUSTRIAL	AGRICULTURE	FOREST	TRANSITION	WATER, WET LANDS, QUARRY	TOTAL
1978	17852	1060	1310	20264	118187	1489	4715	164877
1985	23038	1562	1570	20791	112208	943	4765	164877
1990	24552	1672	1756	20397	111737	0	4765	164879
1995	26375	1804	1942	20019	109976	0	4765	164879
1996	26739	1830	1979	19943	109623	0	4765	164878
1997	27104	1856	2016	19867	109271	0	4765	164878
2000	28197	1935	2127	19640	108214	0	4765	164878
2007	30498	2100	2387	19050	106077	0	4765	164877
2010	31484	2171	2498	18797	105161	0	4765	164876
2017	33658	2329	2758	18243	103126	0	4765	164878
2020	34590	2396	2869	18005	102254	0	4765	164879
2027	36375	2525	3129	17545	100539	0	4765	164878
2030	37140	2580	3240	17348	99804	0	4765	164877
2037	39000	2715	3500	16876	98023	0	4765	164879
2040	39797	2773	3611	16674	97260	0	4765	164880
2047	41657	2908	3871	16202	95479	0	4765	164882

Table 21 (Continued)

FUTURE LAND USE BY GROWTH REGIONS, 1978-2047

(Acres)

CENTRAL REGION

YEAR	RESIDENTIAL	COMMERCIAL	INDUSTRIAL	AGRICULTURE	FOREST	TRANSITION	WATER, WET LANDS, QUARRY	TOTAL
1978	8217	277	209	8003	11703	861	649	29919
1985	9067	318	209	7808	11550	223	743	29918
1990	9297	334	209	7672	11571	92	743	29918
1995	9573	354	209	7529	11425	86	743	29918
1996	9628	358	209	7500	11395	85	743	29918
1997	9683	362	209	7471	11366	84	743	29918
2000	9849	374	209	7385	11278	80	743	29918
2007	10199	399	209	7154	11136	79	743	29919
2010	10349	410	209	7055	11075	78	743	29919
2017	10678	434	209	6839	10939	77	743	29918
2020	10819	444	209	6746	10881	76	743	29918
2027	11091	463	209	6570	10768	75	743	29918
2030	11207	471	209	6495	10719	74	743	29918
2037	11490	491	209	6313	10599	73	743	29919
2040	11611	500	209	6235	10548	73	743	29919
2047	11894	520	209	6053	10428	72	743	29920

BATON ROUGE URBAN REGION

YEAR	RESIDENTIAL	COMMERCIAL	INDUSTRIAL	AGRICULTURE	FOREST	TRANSITION	WATER, WET LANDS, QUARRY	TOTAL
1978	26809	9941	7451	4563	10316	961	1173	61214
1985	30431	10783	7576	3258	7608	297	1261	61214
1990	31369	11459	7665	2531	6880	46	1261	61211
1995	32073	11930	7755	2034	6137	23	1261	61212
1996	32213	12024	7772	1935	5988	18	1261	61212
1997	32354	12118	7790	1835	5839	14	1261	61212
2000	32776	12401	7844	1537	5393	0	1261	61212
2007	33211	12684	7969	1193	4896	0	1261	61214
2010	33397	12805	8023	1046	4683	0	1261	61215
2017	33769	13047	8148	831	4158	0	1261	61214
2020	33929	13150	8202	739	3933	0	1261	61214
2027	34108	13266	8327	656	3596	0	1261	61215
2030	34185	13316	8381	621	3451	0	1261	61215
2037	34356	13428	8506	487	3178	0	1261	61216
2040	34429	13476	8560	430	3061	0	1261	61217
2047	34600	13588	8685	296	2788	0	1261	61218

Table 21 (Continued)

FUTURE LAND USE BY GROWTH REGIONS, 1978-2047
(Acres)

YEAR	RESIDENTIAL	COMMERCIAL	INDUSTRIAL	TOTALS AGRICULTURE	FOREST	TRANSITION	WATER,WET LANDS, QUARRY	TOTAL
1978	90984	15187	17031	200079	303870	6083	138614	771848
1985	113002	17858	18406	191911	288231	3344	139096	771848
1990	119394	19441	19389	188116	286124	288	139096	771848
1995	126578	20977	20382	182816	281789	208	139096	771845
1996	128015	21284	20581	181756	280922	191	139096	771844
1997	129452	21591	20779	180696	280055	175	139096	771844
2000	133762	22512	21375	177516	277454	127	139096	771842
2007	142413	24153	22765	171094	272199	126	139096	771846
2010	146121	24856	23361	168342	269947	125	139096	771848
2017	153701	26019	24751	163026	265131	124	139096	771847
2020	156949	26517	25347	160748	263067	123	139096	771847
2027	162539	27295	26737	156817	259239	122	139096	771844
2030	164934	27628	27333	155132	257599	121	139096	771843
2037	170609	28412	28723	151699	253190	120	139096	771850
2040	173041	28748	29319	150228	251301	120	139096	771853
2047	178716	29532	30709	146795	246892	119	139096	771860

SOURCE : LOUISIANA STATE UNIVERSITY, LOUISIANA STATE PLANNING OFFICE, AND U.S. ARMY CORPS OF ENGINEERS, NEW ORLEANS District

to be highly likely to receive continued development. Portions of the region are within the valleys of the Amite and Comite and not suitable for development, which fact has and will continue to limit the growth in the area. Improvements to the transportation facilities in the area would likely increase the growth potential. The central region, while away from the growth focus for the study area, is likely to experience development at a nearly average rate for the study area. This area, representing roughly 4% of the study area, is projected to receive 3.5% of the area growth.

The Baton Rouge urban region is already heavily urbanized. Several of the subbasins are virtually completely developed at the present time (11, 15, 20, 23, 26, 27). Areas in the southern and eastern parts of this area are projected to become fully developed early in the projection period. The only subbasins not projected to be fully developed by 2040 are 16 (to the north), 24, and 48. While subbasin 16 is not likely to grow rapidly, subbasins 24 and 48 will likely become fully urbanized in the near future. The model apparently mispredicated for these two subbasins in part due to their small size. Subbasin 21 is not projected for full urbanization until 2030. This subbasin contains two large parcels of land, the Burden tract and the Whitter tract, which may not develop in the foreseeable future. The Burden tract is administered by the Louisiana State University College of Agriculture as a park area and a research farm. The Whitter tract is owned by an individual who wishes the area remain in farmland and forest. Most of the remainder of the subbasin is fully developed at the present time. This region is projected to experience 14.9% of all growth in the study area despite its present high degree of urbanization and small areal extent of less than 8% of the total study area.

This rapidly developing southern region located to the south and southeast of the urbanized area contains the major traffic arteries, I-10 and Airline Highway. Major industrial sites are located along the Mississippi River portion of this region. The area serves as the place of residence for workers in both Baton Rouge and the river industries. Commercial growth is strong in the northern portion of the area, as well. Subbasins 43 and 60 will probably never become densely developed because much of the

land is divided into parcels of one to five acres with single family residences located upon them. Subbasin 29 has a very great potential for growth as it is located quite near the center city of Baton Rouge and to the Louisiana State University major employer. It also borders on the Mississippi River which provides opportunities for industrial expansion. The five subbasins in this region, comprising only 7% of the study area, are projected to receive 29.7% of all growth in the study area. This region will show the greatest transformation of land uses by far.

Interstate 12 passes through the center of the Livingston Parish region and is likely to provide impetus for continued growth. The region contains the small urban center of Denham Springs and to the west it borders upon the urbanized area of Baton Rouge. The area contains abundant land well-suited to development. Subbasins 46, 47, 51, and 52 lie partly within the Amite River flood plain and this will somewhat limit their potential for growth. However all of these do contain land at higher elevations. This area is projected to receive 23.4% of the study area growth, only slightly above average for the study area as it contains over 21% of the land area.

The Ascension region is composed of parts of both Iberville and Ascension parishes. It contains the small urban center of Gonzales, I-10, Airline Highway, and a major industrial area located along the Mississippi. The potential for growth is obviously great. Employment is focused upon the river industries, which, while experiencing continued growth, have not expanded in recent years at the rapid rate of the early 1970's. Gonzales, though small, is located within five of the subbasins. This may result in some distortion of the projections for individual subbasins. This region, containing 13% of the study area, is projected to receive 10.2% of the growth.

The lower region contains a large amount of low-lying swamp. The only areas suitable for development are in the western portions of subbasins 40, 42, and 44, and , to a limited extent, along the natural levees of the Amite and its distributaries. The area is heavily used for recreational purposes and many of the structures in the area are camps, not primary residences. The area does provide residence for river industry employees, and, to a

lesser extent, for Baton Rouge workers.

Despite its large size (21.5% of the study area), this area is projected to receive 7.7% of the study area growth. This will be concentrated in the northern and extreme western portions of the area.

Biological Resources

Biological resources in the study area are closely related to land use regardless of past, present, or future times. Land use studies and projec tions indicate a gradual reduction in the forested land and agricultural land category (see Table 22). Forested land provides habitat for a large number of wildlife species, and winter flooded woodlands provide signifi cant habitat for wintering waterfowl. Agricultural lands provide habitat for species that require open lands; however, regionally and nationally, habitat for these species is abundant. The rapid trend of conversion of forested and agricultural lands to residential and commercial development that was experienced from the mid-1960's to the early 1980's has slowed and is projected to remain stable (see Appendix B). Wildlife population reduc tions would be slowed accordingly, but would still occur. Timber harvests would be made to some of the forested lands of the area as stands mature. The development of, or encroachment upon, the riparian zone in scattered locations would further reduce the escape and travel cover provided to ground-dwelling mammals as well as some birds. These actions would reduce the total amount of input of organic material to the Comite and Amite Rivers. The input of this material is the primary basis for the productiv ity of these rivers and the loss of this material would cause a decline in species diversity and numbers associated with that encroachment. If the projected improvement of the water quality conditions below Baton Rouge occurs, that improvement should result in a more downstream extension of desirable habitat for aquatic species requiring waters of higher quality, and especially higher dissolved oxygen content, in both the Comite and Amite Rivers.

TABLE 22
EXISTING AND FUTURE LAND USE ACREAGE
WITHIN THE STUDY AREA

Land Use Type	1985	2040	2047
Forest lands	172,523	145,199	144,426
Agricultural lands	84,629	55,196	54,119
Developed lands	84,447	143,630	147,308
Transitional lands	4,876	2,450	622
Total lands	346,475	346,475	346,475

SOURCE: U.S. Army Corps of Engineers, New Orleans District

Water Resources

During periods of excessive rainfall, runoff will exceed capacity of the Amite and tributary streams and flooding in the Amite River Basin will continue to cause millions of dollars in damages. These flood damages would increase in the future as urbanization increases in the floodplain. During the period 1972-1978, urban development occurred about 51 percent within the floodplain and 49 percent outside the floodplain. Between 1978-1985, slightly more of the development (about 53 percent) was outside the 100-year floodplain. The distribution of future growth within and outside the floodplain is expected to shift in the direction where more growth occurs outside the 100-year floodplain. This is due to the recent implementation of floodplain management programs in East Baton Rouge, Livingston, and Ascension Parishes. Since the four major floods between 1973 and 1983, it appears that there has been a more judicious use of the floodplain. In addition, implementation of a Federal project would require that the local sponsor adopt regulations as necessary to prevent unwise future development in the floodplain. The local sponsor is required to ensure that future development is compatible with the protection level provided by a proposed Federal project.

There is no indication that water quality in the Comite River, Amite River, or Lake Maurepas would worsen in the future. In fact, it seems that the water quality of the aforementioned water bodies would improve as a result of the implementation of the waste management practices set forth in the Louisiana water quality management plan. The East Baton Rouge Parish's plan is to divert a large portion of the municipal waste that is currently being discharged to tributaries of the Amite River to the Mississippi River. This should improve water quality in the future.

Future water use for the Louisiana portion of the basin is shown on Table 23. Water use is expected to almost triple between 1980 and 2040. Over 90 percent of the water use is expected to be from surface water. Water supply sources have been determined to be capable of meeting the projected requirements.

Cultural Resources

Six trends affect preservation of cultural resources in the study area. The first is urbanization encroaching on the central basin from its western and southern edges. The region's annual flooding pattern has limited twentieth century settlement choices. As a consequence, construction has been roughly contained within corridors along major highways, inadvertently protecting riverine oriented prehistoric sites and early homesteads from rezoning and large scale clearing usually associated with construction of tract housing or light industry. This trend is slowly changing as developmental corridors widen. Clearing removes sites which are close to the surface, then exposes remaining deposits to lateral erosion from increased

Table 23

FUTURE WATER USE IN THE LOUISIANA PORTION OF AMITE RIVER BASIN

(Million Gallons Per Day)

		PUBLIC SUPPLY		INDUS TRIAL		POWER GENER ATION		RURAL DOM.	LIVE STOCK		RICE		OTHER IRRIG.		AQUACU LTURE		TOTAL		
	YEAR	GROUND	SURFACE	GROUND	SURFACE	GROUND	SURFACE	GROUND	GROUND	SURFACE	GROUND	SURFACE	GROUND	SURFACE	GROUND	SURFACE	GROUND	SURFACE	TOTAL
ASCENSION	1990	3.65	1.47	9.12	261.84	0.00	0.00	3.05	0.19	0.02	0.00	0.00	0.59	0.00	0.00	0.42	16.60	263.75	280.35
	2000	7.05	1.54	13.38	305.70	0.00	0.00	3.77	0.23	0.01	0.00	0.00	0.59	0.00	0.00	0.45	25.02	307.70	332.72
	2010	10.45	1.61	17.63	349.58	0.00	0.00	4.48	0.27	0.01	0.00	0.00	0.59	0.00	0.00	0.47	33.42	351.65	385.07
	2020	13.85	1.68	21.88	393.42	0.00	0.00	5.20	0.31	0.01	0.00	0.00	0.58	0.00	0.00	0.49	41.82	395.60	437.42
E.BATON ROUGE																			
	1990	58.80	0.00	93.54	128.84	6.80	3.87	2.43	0.19	0.03	0.00	0.00	0.15	0.00	0.15	0.00	162.06	132.74	294.80
	2000	67.38	0.00	107.03	172.70	6.73	3.76	3.00	0.23	0.03	0.00	0.00	0.15	0.00	1.05	0.00	185.57	176.49	362.06
	2010	85.21	0.00	120.52	216.56	6.67	3.65	3.57	0.27	0.03	0.00	0.00	0.14	0.00	1.10	0.00	217.48	220.24	437.72
	2020	102.50	0.00	134.01	260.42	6.60	3.54	4.15	0.31	0.03	0.00	0.00	0.14	0.00	1.16	0.00	248.87	263.99	512.86
E. FELICIANA																			
	1990	1.07	0.00	0.43	3.22	0.00	0.00	0.59	0.16	0.18	0.00	0.00	0.03	0.00	0.04	0.00	2.32	3.40	5.72
	2000	1.10	0.00	0.74	5.56	0.00	0.00	0.73	0.20	0.17	0.00	0.00	0.03	0.00	0.04	0.00	2.84	5.73	8.57
	2010	1.13	0.00	1.07	7.88	0.00	0.00	0.87	0.24	0.16	0.00	0.00	0.03	0.00	0.05	0.00	3.39	8.04	11.43
	2020	1.16	0.00	1.40	10.22	0.00	0.00	1.01	0.28	0.15	0.00	0.00	0.03	0.00	0.05	0.00	3.93	10.37	14.30
IBERVILLE																			
	1990	3.89	0.00	37.14	714.84	2.30	730.00	0.22	0.20	0.00	0.00	0.00	0.00	0.00	0.00	6.30	43.75	1451.14	1494.89
	2000	4.05	0.00	37.34	758.70	2.50	730.00	0.28	0.24	0.00	0.00	0.00	0.00	0.00	0.00	10.95	44.41	1499.65	1544.06
	2010	4.21	0.00	37.54	802.56	2.70	730.00	0.33	0.20	0.00	0.00	0.00	0.00	0.00	0.00	11.50	44.98	1544.06	1589.04
	2020	4.38	0.00	37.74	846.42	2.90	730.00	0.38	0.33	0.00	0.00	0.00	0.00	0.00	0.00	12.08	45.73	1588.50	1634.23
LIVINGSTON																			
	1990	6.08	0.00	1.34	0.00	0.00	0.00	3.96	0.16	0.08	0.00	0.00	0.04	0.00	0.00	0.00	11.58	0.08	11.66
	2000	9.75	0.00	1.93	0.00	0.00	0.00	4.89	0.20	0.07	0.00	0.00	0.04	0.00	0.00	0.00	16.81	0.07	16.88
	2010	13.42	0.00	2.52	0.00	0.00	0.00	5.83	0.24	0.07	0.00	0.00	0.04	0.00	0.00	0.00	22.05	0.07	22.12
	2020	17.09	0.00	3.08	0.00	0.00	0.00	6.76	0.28	0.06	0.00	0.00	0.04	0.00	0.00	0.00	27.25	0.06	27.31
ST. HELENA																			
	1990	0.23	0.00	0.00	0.00	0.00	0.00	0.73	0.67	0.09	0.00	0.00	0.00	0.00	0.00	0.00	1.63	0.09	1.72
	2000	0.26	0.00	0.00	0.00	0.00	0.00	0.90	0.81	0.08	0.00	0.00	0.00	0.00	0.00	0.00	1.97	0.08	2.05
	2010	0.32	0.00	0.00	0.00	0.00	0.00	1.07	0.95	0.08	0.00	0.00	0.00	0.00	0.00	0.00	2.34	0.08	2.42
	2020	0.34	0.00	0.00	0.00	0.00	0.00	1.25	1.09	0.06	0.00	0.00	0.00	0.00	0.00	0.00	2.68	0.06	2.74
ST. JAMES																			
	1990	0.00	2.45	4.87	458.00	0.00	1000.00	0.06	0.03	0.00	0.00	0.00	0.12	0.00	0.00	28.20	5.08	1488.65	1493.73
	2000	0.00	2.53	3.75	590.30	0.00	2000.00	0.07	0.04	0.00	0.00	0.00	0.12	0.00	0.00	34.50	3.98	2627.33	2631.31
	2010	0.00	2.61	2.51	732.60	0.00	2000.00	0.09	0.05	0.00	0.00	0.00	0.14	0.00	0.00	37.90	2.79	2773.11	2775.90
	2020	0.00	2.69	0.86	874.90	0.00	2000.00	0.12	0.06	0.00	0.00	0.00	0.18	0.00	0.00	41.70	1.22	2919.29	2920.51
ST. JOHN																			
	1990	1.90	4.00	2.63	219.90	0.00	0.00	0.61	0.01	0.02	0.00	0.00	0.00	0.00	0.00	0.60	5.15	224.52	229.67
	2000	2.38	4.91	2.02	317.50	0.00	0.00	0.79	0.01	0.02	0.00	0.00	0.00	0.00	0.00	0.70	5.20	323.13	328.33
	2010	2.85	5.82	1.36	415.10	0.00	0.00	0.97	0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.80	5.20	421.74	426.94
	2020	3.33	6.73	0.47	512.70	0.00	0.00	1.37	0.03	0.03	0.00	0.00	0.00	0.00	0.00	0.90	5.20	520.36	525.56
TOTAL	1990	76.62	7.92	149.07	1786.64	9.10	1733.87	11.65	1.61	0.42	0.00	0.00	0.93	0.00	0.19	35.62	248.17	3564.37	3812.54
	2000	91.97	8.98	166.19	2150.46	9.23	2733.76	14.43	1.96	0.38	0.00	0.00	0.93	0.00	1.09	46.60	285.80	4940.18	5225.98
	2010	117.69	10.04	183.16	2524.26	9.37	2733.65	17.21	2.24	0.37	0.00	0.00	0.94	0.00	1.16	50.67	331.86	5318.99	5650.84
	2020	142.66	11.10	199.44	2898.08	9.50	2733.54	20.24	2.69	0.34	0.00	0.00	0.97	0.00	1.21	55.17	376.70	5698.23	6074.93

Source: Louisiana Department of Transportation and Development

localized run-off. Riverine sites are directly impacted by development of recreational camps along the Amite River. Camp building, a second trend, localizes construction impact without areal clearing. Campsite selection echoes prehistoric and eighteenth century site selection, disturbing and adding a modern component to sites on the natural levee. Vandalism, a third trend, has been identified by the State Archeologist as prevalent near urban areas where obvious sites, such as mounds or those exposed by construction, are at jeopardy because of their accessibility. A fourth trend, also associated with development and land use change, is insensitive alteration or modification of historic structures which otherwise might be eligible to the National Register of Historic Places. Fifth is agricultural cropping north and east of Baton Rouge which disturbs subsurface deposits during clearing, plowing, and deep tilling of the soil. The final trend is prolonged flooding, followed either by alluviation or scouring of sites adjacent to channels. Scouring, which destroys site integrity, is a factor of elevational slope and natural channel migration. By contrast, alluvial and colluvial deposition buffers sites from shallow surface disturbance and may be interpreted as having some positive preservation benefit. All six of these trends are active in the study area, and can be expected to continue at present rates.

Recreation Resources

In the future, the study area is expected to continue to experience a slow increase in rural population, while urban areas such as Baton Rouge will experience a more rapid population growth. Population growth translates into increased per capita recreation demand. Future expanded populations will require additional recreational facility development as well as expansion of existing facilities. Outdoor recreation pursuits existing in the predominantly rural setting will experience continued use in the future. At times, during periods of peak use in the summer season, maximum use will occur.

PROBLEMS

Floods

The storms that have caused heavy rainfall in the Amite River Basin have been similar. The rainfall occurs over a 3-to 5-day period. A near stationary cold front moves across southeast Louisiana and southern Mississippi and warm and humid air from the Gulf of Mexico moves inland ahead of the front. This situation produces very intense thunderstorms and heavy rainfall rates one inch or more an hour.

If there have already been several months of heavier-than-normal rainfall when this intense storm occurs--as in the early spring--the stage is set for a disastrous flood. The high rate of rainfall runoff exceeds the capacity of the Amite River and its tributaries. The result is headwater and backwater flooding. Headwater overflow occurs in the upper reaches of the Amite River and its tributary streams. Backwater flooding occurs in the lower reaches of the river and on the streams in the vicinity of the confluence with the Amite.

The brunt of these storms in the Amite Basin is primarily felt by agricultural lands, wooded areas, and swampland. But extensive flooding also occurs in and around the cities of Baton Rouge, Denham Springs, Gonzales, Baker, and Zachary, and the towns of French Settlement and Port Vincent.

Flood damages from the 1973 flood were more than $2 million and about 105,000 acres were flooded. In the 1977 flood, about 164,000 acres were flooded. More than 4,000 structures were flooded and the damages came to $78 million. The 1979 flood was not as widespread as the one in 1977. Flood damages in 1979 were estimated at over $9 million.

The 1983 flood was the flood of record in most of the basin. Over 357,000 acres were flooded in East Baton Rouge, Livingston, Iberville, Ascension, St. James, and St. John the Baptist Parishes. About 5,300 homes and 200 businesses were flooded. Flood damages were estimated at a record $172 million.

The 1983 flood damages by parish is summarized in Table 24.

TABLE 24

1983 FLOOD DAMAGES BY PARISH

PARISH	HOMES DAMAGED	BUSINESSES DAMAGED	ACRES FLOODED	FLOOD DAMAGES
ASCENSION	793	17	78,400	$ 20,358,500
EAST BATON ROUGE	1,578	37	55,000	65,263,000
IBERVILLE	---	--	13,000	26,300
LIVINGSTON	2,875	150	136,750	83,814,100
ST. JAMES	1	--	38,700	23,100
ST. JOHN THE BAPTIST	10	--	35,950	100,000
Relief Cost*	--	--	--	2,056,100
TOTAL	5,257	204	357,850	$171,721,100

*Relief cost not collected by parishes

SOURCE: U.S. Army Corps of Engineers, New Orleans District

Biological

Problems to biological resources consist primarily of the loss of wildlife habitat and habitat quality caused by residential and commercial development. All upstream development contributes to aquatic habitat problems because of the resulting higher flood peaks and runoff from urban areas deteriorates water quality. Development within the riparian zone is a problem of major concern.

Water Quality

Water quality in the basin has deteriorated in the lower basin due to municipal and industrial discharges, urban stormwater runoff, and to a lesser extent, agricultural runoff. The implementation of the state water quality management plan and East Baton Rouge Parish's plan

to discharge most of municipal waste to Mississippi River should improve water quality. However, during low flow conditions, the Amite River between the Amite River Diversion Canal and Lake Maurepas has experienced undesirable environmental conditions. These conditions include increased siltation in the river, reduced game fish population and low DO concentrations.

Recreation

The potential for sport fishing and hunting is constrained by a lack of access. There are not enough camping, picknicking, swimming, and sightseeing facilities.

NEEDS AND OPPORTUNITIES

Flood Control

There is a need to reduce or alleviate flood problems in the Amite River Basin. Partial or full flood protection would reduce the financial risk involved to home owners, businesses, and agriculture enterprises. A range of opportunities are available to address the problem. These opportunities could be realized by constructing reservoirs, channel modifications, diversions, levees, floodgates, pumping stations, floodplain management, and nonstructural measures.

Biological

There is a need to slow the trend of habitat and habitat quality reduction for both terrestrial and aquatic species. Mitigation opportunities for both terrestrial and aquatic species should be considered an essential part of any Federal action plan developed.

Water Supply

As stated above, existing water supply sources are adequate to meet future area needs. The creation of new supplies is therefore not a primary consideration in the development of flood control alternatives for this project.

Water Quality

There is a need to improve water quality in the mid to lower Amite River and opportunities exist to accomplish this.

Recreation

There is a need to improve recreation in the study area that includes additional boating, camping, picknicking, swimming, and sightseeing facilities. In addition, there is a need to improve the freshwater fishery. These needs can be partially satisfied by constructing additional recreation facilities in the area. Resources can be improved in some areas by the possible creation of a permanent lake and by regulating river flow to produce more water habitat conducive to fish production and recreational activities.

Energy Supply

Area electrical power supplies are adequate to meet future needs. The creation of new supplies (hydro-electric power) is not a primary consideration in the development of flood control alternatives for this projects.

PLANNING CONSTRAINTS

Legislative and executive authorities specify constraints and criteria that must be applied when evaluating alternative plans and the range of impacts to be assessed. In developing plans, tangible and intangible benefits and costs are considered as well as effects on the ecological, social, and economic well-being of the region. Federal participation in development requires that any plan be complete within itself, efficient and safe, economically feasible in terms of current prices, environmentally acceptable, and consistent with local, regional, and state plans.

The plan formulation goal for this study is to develop alternative plans to reduce flood damages along the Amite River and its lower tributaries. It is realized by all agencies involved that one flood control measure would

not solve the flood problem in the Amite River Basin. The goal is to develop a comprehensive flood control plan for the Amite River Basin. Recreation and environmental measures were considered as part of a proposed flood control project or mitigation lands.

PLANNING OBJECTIVES

Planning objectives stem from national, state, and local water and related land resources management needs specific to the study area. These objectives have been developed through problem analysis and an intense public improvement program. They have provided the basis for plan formulation. The planning objectives are as follows:

a) Reduce flood damages along the lower Amite River and its lower tributaries.

b) Minimize adverse environmental impacts associated with the implementation of flood control measures.

c) Minimize to the extent possible the destruction of archaeological and historical resources.

d) Minimize particuliarly the loss of bottomland hardwood forests or if not possible, mitigate those losses "in kind" to the extent practicable.

e) Incorporate to the extent possible recreation facilities in flood control plans to increase recreation opportunities.

f) Mitigate for all unavoidable impacts to significant fish and wildlife resources.

DEVELOPMENT OF ALTERNATIVE PLANS

MANAGEMENT MEASURES

Structural measures considered for providing flood control along the main stem of the Amite River included the following:

Reservoir(s)

Channel modification

Levees

Channel Diversion

Pump Station(s)

These measures would also address other planning objectives. Nonstructural measures considered included:

Floodplain Management

Floodproofing Structures

Raise Structures in Place

Build Small Earthen Levees or Floodwalls

Ring Levees around Selected Subdivisions

Flood Forecasting and Warning

Removal of Structures from Floodplain

Public Acquisition of Floodplain Land

PLAN FORMULATION RATIONALE

The Water Resources Council Principles and Guidelines require various alternative plans be formulated in a systematic manner to ensure that all reasonable

alternatives are evaluated. Each alternative is to be formulated in consideration of four criteria: completeness, effectiveness, efficiency, and acceptability. Completeness is the extent to which a given alternative plan provides for all necessary investments or other actions to ensure the realization of the planned effects. Effectiveness is the extent to which an alternative plan alleviates the specified problems and achieves the specified opportunities. Efficiency is the extent to which an alternative plan is the most cost-effective means of alleviating the specified problems and realizing the specified opportunities, consistent with protecting the Nation's environment. Acceptability is the workability and viability of the alternative plan with respect for acceptance by state and local entities and the public, and compatibility with existing laws, regulations, and public policies. In addition, mitigation of adverse effects is to be an integral part of each plan. In general, when formulating alternative plans, an effort is made to include only increments that increase the net NED benefits on a first- and last-added basis.

INITIAL PLAN FORMULATION AND SCREENING

Plan formulation was initially performed in the 1984 Amite River and Tributaries Initial Evaluation Report on Flood Control (Federal Reconnaissance Report). This study identified two structural alternatives, a single reservoir and channel modifications, as potentially feasible structural plans. The latter alternative entails clearing and snagging, and widening of the Amite River. This plan was determined to be more costly than a single reservoir and to have significant environmental and implementation problems. "Channelization" of the Amite River would significantly impact the environmental quality of this waterbody. Also, the disposal of massive quantities of excavated material was also determined to be quite problematic with this alternative. Significant public opposition to this plan was expressed making its implementation questionable. This study also determined that non-structural alternatives are feasible but will not solve all basin-wide flooding problems. Use of non-structural measures in combination with a structural alternative was determined to be feasible.

An extensive plan formulation, evaluation and screening analysis was performed by Espey, Huston & Associates, Inc. in their 1989 Amite River Flood Control Study. See Appendix B. This analysis entailed initial, secondary and final screening of 33 potential alternatives plus a No Action plan. Alternatives evaluated were single reservoirs of various storage capacities, both wet and dry (with and without recreational lakes of various sizes), multiple reservoirs, channelization of the Amite River, non-structural measures, river bank levees, and several combinations of the above with and without the influence of the proposed Comite River Diversion Canal project. This screening analysis resulted in the recommendation of four alternatives for final detailed evaluation. They are:

- a single reservoir with maximum flood storage capacity with a permanent full size recreation lake (full size wet reservoir)

- a single reservoir with maximum flood storage capacity without a permanent recreation lake (full size dry reservoir)

- a single reservoir with reduced flood storage capacity without a permanent recreation lake (reduced size dry reservoir) in combination with selected non-structural measures

- no action plan

The Espey, Huston & Associates plan formulation and screening analysis was reviewed and determined to be sound and comprehensive. It was therefore determined that this work should not be repeated.

Additional plan formulation was developed based on the results of the above report. Initial evaluation of reservoir costs and related flood reduction and recreation benefits indicated that a broader range of final alternatives should be evaluated.

It was determined that significant flood control

benefits exist with a 25 year level flood control project design, thus confirming the potential feasibility of a small reservoir. This factor resulted in the formulation of various reservoir alternatives with flood storage capacities ranging from a 25 year to a 100 year storm event.

It was also determined that significant recreation benefits exist with incorporating a permanent lake as part of the reservoir scheme. Both large and small recreation lake plans were integrated with the above listed range of flood control options. They were respectively designated as "Wet" and "Reduced Wet" alternatives.

Since "wet" reservoirs can operate with or without a service spillway, this option was also incorporated into the final plans (see Hydrologic and Hydraulic Design for further discussion on service spillway details).

Summarizing the above, three types of reservoirs were selected for final detailed evaluation. They are:

- Wet (reservoir with a full size 12,800 acre recreation lake

- Reduced Wet (reservoir with a small 4,400 acre recreation lake

- Dry (reservoir with no recreation lake)

Incorporating three levels of flood control protection 25, 50, and 100 year, and considering each wet reservoir both with and without a service spillway yielded a final array of fifteen final alternatives as listed in Table 25.

TABLE 25

FINAL ARRAY OF FLOOD CONTROL ALTERNATIVES

1. Wet Reservoir with Service Spillway; 100 year protection
2. Wet Reservoir with Service Spillway; 50 year protection
3. Wet Reservoir with Service Spillway; 25 year protection
4. Wet Reservoir without Service Spillway; 100 year protection
5. Wet Reservoir without Service Spillway; 50 year protection
6. Wet Reservoir without Service Spillway; 25 year protection
7. Reduced Wet Reservoir with Service Spillway; 100 year protection
8. Reduced Wet Reservoir with Service Spillway; 50 year protection
9. Reduced Wet Reservoir with Service Spillway; 25 year protection
10. Reduced Wet Reservoir without Service Spillway; 100 year protection
11. Reduced Wet Reservoir without Service Spillway; 50 year protection
12. Reduced Wet Reservoir without Service Spillway; 25 year protection
13. Dry Reservoir; 100 year protection
14. Dry Reservoir; 50 year protection
15. Dry Reservoir; 25 year protection

EVALUATION OF FINAL ALTERNATIVES

As detailed below, evaluations of the final alternatives were performed with analyses of the following:

- Hydrologic and Hydraulic Design
- Geology and Dam Location
- Geotechnical and Structural Design
- Roadway and Utility Relocations
- Environmental Impact and Mitigation
- Real Estate
- Recreation Resources
- National Economic Development Benefits

Analyses were done to the extent necessary to adequately determine the feasibility of each alternative. In some cases, only a range of several plans were specifically evaluated. In some categories it was only necessary to analyze two plans to obtain adequate data needed to determine the feasibility of all alternatives.

HYDROLOGIC AND HYDRAULIC DESIGN

HYDROLOGIC MODELING

Comprehensive flood hydrographs were required for estimating peak flows for specified return periods. This was accomplished by use of the rainfall/runoff model HEC-1 that was developed by the U.S. Army Corps of Engineers. A complete description of this model, including data requirements, limitations, and examples of applications are given in the HEC-1 Users Manual (U.S. Army Corps of Engineers, 1990).

For the Amite River Basin, a HEC-1 model was developed. Hypothetical precipitation values for the 1, 2, 5, 10, 25, 50, 100, 200, and 500 year return period storms were obtained from Hydro 35, Technical Paper 40, and Technical

Paper 49. The storms were centered over the entire watershed and had a 7 day duration. Precipitation values were also developed for the Standard Project Storm (SPS) and the Probable Maximum Storm (PMS) using Engineering Manual 1110-2-1411 and Hydrometeorological Report No. 51 and 52. Discharge frequency curves were developed at each gage location with recorded discharges. The curves were plotted using HEC-WRC which utilizes the Log-Pearson Type III distribution for its computations. The HEC-1 model was calibrated to the hypothetical storms and the calibration checked against the 1977, 1979, and 1983 historical events. The results of this calibration are described in detail in the Comite River Basin Feasibility Study.

The existing conditions HEC-1 model for the Comite River Basin Feasibility Study was provided to LaDOTD and Espey Huston and Associates, Inc. (EH&A) for use in their studies of the Amite River Basin. EH&A revised the upper Amite River portion of the model upstream of Denham Springs to concentrate on that segment of the watershed. Their analyses concentrated on the 10, 50, and 100 year events. The discharge frequency target values remained the same for the gaged locations except for the Darlington Gage where EH&A developed a volume analysis to determine frequency flow.

For the reservoir feasibility study, the resulting EH&A HEC-1 model described above was utilized for the USACE analyses. The 1, 2, 5, 25, 200, 500, SPS, and PMS events were again included and the discharge frequency relationships at each of the gages were reviewed and the model's calibration was checked.

HYDRAULIC MODELING

The computer model HEC-2 was used to simulate the hydraulic response of the watershed. The HEC-2 program was developed by the U.S. Army Corps of Engineers. A complete description of this model, including data requirements, limitations, options, and examples of application are given in the HEC-2 Users Manual (U.S. Army Corps of Engineers, 1990).

As with the HEC-1 model, the New Orleans District developed the HEC-2 model for the Amite River Basin for the

Comite River Basin Feasibility Study. The model was developed in detail for the Comite River and the lower Amite River below Denham Springs and to a lesser degree for the upper Amite River. The model was calibrated to three historical events; the 1977, 1979, and 1983 flood events. The results of this calibration are discussed in detail in the Comite River Basin Feasibility Study Report.

LaDOTD and EH&A used the COE supplied HEC-2 model to analyze the Darlington Reservoir in their study. They refined the upper Amite River portion of the model to better simulate existing and improved conditions.

For this study, New Orleans District used the refined EH&A HEC-2 model for the analyses. The model calibration was checked and verified against historical flood events and also against stage-frequency curves developed at each of the gage locations.

ALTERNATIVES

The EH&A report identified three feasible reservoir alternatives; a 50-year wet reservoir, a 50-year dry reservoir, and a reduced sized flood control dry reservoir. The wet reservoir studied had a conservation pool elevation at 171.0 ft NGVD, a flood control pool elevation of 186.6 ft NGVD, and a top of dam (including 6.0 ft of freeboard) at 206.0 ft NGVD. The structure consisted of an earthen embankment (with an impervious core) with 1V on 3H side slopes and a 20 foot top width, a concrete service spillway with a crest elevation of 147.5 ft NGVD with three 40'x40' tainter gates, a 1000 foot-long concrete emergency spillway at elevation 186.6 ft NGVD, and a low-flow outlet consisting of three 8'x10' gated concrete box culverts.

The EH&A 50-year dry reservoir would not have a conservation pool, but would have a flood control pool elevation of 174.5 ft NGVD and a top of dam elevation of 202.2 ft NGVD (including 6.0 ft of freeboard). The structure would consist of an earthen embankment (with an impervious core) with 1V on 3H side slopes and a 20 foot top width, a 1000 foot-long concrete emergency spillway at elevation 174.5 ft NGVD and a low-flow outlet consisting of three 8'x10' non-gated concrete box culverts. No service spillway would be required for a dry reservoir.

The EH&A reduced dry reservoir would provide a lesser level of protection than the 50-year dry reservoir. The dimensions and features of this reservoir design are available from the EH&A report.

For this feasibility study, several alternatives were investigated. The Interdisciplinary Planning Team (IPT) decided early on in the study to analyze in detail the full size 50-year wet and dry reservoirs proposed in the EH&A report. In addition, the IPT decided to analyze three levels of protection for each type of reservoir. By level of protection, it is meant the frequency event for which the flood control pool is set and thereby determining the emergency spillway crest elevation. For example, the 50-year wet reservoir with a service spillway has a flood control pool elevation of 186.9 ft NGVD (under the COE analysis). This reservoir will contain (except for reservoir low-flow discharges) the runoff from the watershed above the reservoir site for events up to the 50-year event. The IPT also decided to analyze a reduced wet reservoir with a lower conservation pool elevation of 150.0 ft NGVD. Finally, the IPT decided to analyze both the wet and reduced wet reservoir with and without a service spillway. This resulted in a total of 15 reservoir alternatives. The results of the hydrologic and hydraulic analyses of these 15 alternatives are provided in Table 26. Areas inundated by the permanent lake and PMS are shown for the 50 year Wet, Reduced Wet, and Dry Alternatives on Plates 12-14, respectively.

Stage Lowerings Stage lowerings for the 15 reservoir alternatives were determined for various points (reaches) along the Amite and Comite Rivers as well as their primary tributaries. These stages were determined by routing the reservoir outflow hydrographs downstream through the Amite River, using the HEC-1 and HEC-2 models. Tables 27 through 32 provide existing and with-reservoir conditions for the 15 reservoir alternatives at principal locations along the Amite and Comite Rivers.

Flood Control Pool The HEC-1 models were used to determine the Flood Control Pool elevation for each reservoir. The design storm for each dam was routed on top of the conservation pool for the large and reduced wet reservoirs. The invert elevation of the Amite River

established initial conditions for the dry alternative. For the wet reservoirs, the dam was assumed to pass a maximum of 10,000 cfs between conservation pool and flood control pool through the low level outlets and the service spillway. The dry reservoir was designed to pass the maximum 10,000 cfs through the uncontrolled low level outlets. This maximum release of 10,000 cfs is the bank full capacity below the reservoir. Flood control pool elevation established the crest elevation of the emergency spillway.

Height of Dams ER-1110-8-2 (FR) **Inflow Design Floods for Dams and Reservoirs** was used to determine criteria for calculating the height of each dam alternative. The height of each dam was computed as the maximum surcharge pool elevation plus freeboard.

The Maximum Surcharge Pool Elevation was calculated by routing the runoff from the Probable Maximum Precipitation (PMP) on top of the Flood Control Pool Elevation. The PMP rainfall was determined using the HMR52 Probable Maximum Storm computer program. The number of subareas upstream of the reservoir were reduced from 8 to 3 to account for the impact of the reservoir pool on the inflow hydrograph at the reservoir structure. An induced surcharge pool of 3 feet above the Flood Control Pool Elevation was used for the service spillway design.

Freeboard requirements for wave run-up and wind setup were calculated using ETL-1110-2-305 **Determining Sheltered Wave Water Characteristics** and EC 1110-2-27 **Policies & Procedures Pertaining to Determination of Spillway Capacities and Freeboard Allowances for Dams** which referenced ETL 1110-2-8 **Computation of Freeboard Allowances for Waves in Reservoirs.** (Wind speeds were estimated from ETL 1110-2-221 and Civil Works Investigations Project CW-178 "Freeboard Criteria for Dams and Levees," Tech. Bulletin No. 1) 0.5 foot was added to this freeboard calculation for estimated increase in stages due to sedimentation over the project life. (0.5′ estimate from the report prepared by Brown & Butler). If this study had continued, a sedimentation analysis would have been completed allowing a more detailed estimate of freeboard needs due to sedimentation.

Service Spillway The service spillway is a tainter gate control structure that enables rapid lowering of lake levels. While this type of control is desirable in the operation of "wet" reservoirs, it is not a necessity. As in the case of "dry" reservoirs, "wet" reservoirs can function without a service spillway. In this initial design phase, all service spillway designs consisted of 3 approximate 40'x40' tainter gates.

Low Level Outlets The low level outlets for all reservoir alternatives were designed to pass a maximum of 10,000 cfs at the flood control pool elevation. These sizes were checked for drawdown requirements as specified in ER 1110-2-50 (Low Level Discharge Facilities for Drawdown of Impoundments). (Low level outlets were assumed to be closed at stages above the Flood Control Pool Elevation due to high velocities through these culverts).

Design of Spillways and Reservoir Outlet Works

Due to the large number of alternatives, only the 50-year reservoir alternatives were initially evaluated determine project costs for economic feasibility. The designs of the other levels of protection were determined to be similar enough to the 50-year reservoirs to allow this generalization. The designs of the spillways and low-level outlets were done using COE Engineering Manuals. EM 1110-2-1603 **"Hydraulic Design of Spillways"** was used to design the spillways. EM 1101-2-1602 **"Hydraulic Design of Reservoir Outlet Works"** was used to design the low level outlet works. Several of the CORPS computer programs were used for the design of the spillway crest and nappe profiles. Flows from a PMP were used in this design. Low level outlets were aligned within the service spillway gate bays for the wet reservoir alternatives and had their own outlet works for those alternatives without a service spillway.

ALTERNATIVE OPTIMIZATIONS

Once the hydraulic design data for each of the reservoir alternatives had been developed and costs of the proposed project were estimated, the designs were revisited to determine if refinements could reduce the total cost of the project. At this time, most of the refinements were

applied to the dry reservoir design because it appeared that the wet reservoir would be far too costly to be economically justified. The refinements included redesign of the emergency spillway, optimization of the low-level outlets, and methods for reducing the height of the dam and thereby reduce the total cross-sectional needs of the dam. The following paragraphs summarize the refinements analyzed and their results.

Emergency Spillway The initial design of the emergency spillway involved an ogee type concrete spillway similar to the type provided in the EH&A report. This structure proved to be extremely expensive. The design was refined to provide a concrete chute spillway which, while still costly, provided substantial reductions in cost.

A 1000 foot long emergency spillway was provided for each of the initial 15 reservoir designs. In an effort to reduce the height of the dam (thereby reducing the cross-sectional areas of the dam), the length of the spillway was varied to determine if the reduced cost of the dam embankment would offset the increased cost of the emergency spillway. However, this refinement was limited by the PMP discharge over the spillway. To prevent induced stages downstream of the dam, longer spillways which resulted in increased PMP discharges from existing conditions were eliminated from further consideration. A shorter emergency spillway was also investigated to determine if the cost savings in a shorter spillway would offset the higher cost of the resulting higher dam and embankment. The results of these iterations are provided in Table 33 (NOTE: Some of the Table 33 data is not available. When the iteration reached the point where it was apparent that this would not be a more acceptable alternative, further analysis was ceased). None of the refinements produced cost savings (in fact, some were more expensive) large enough to produce an economically feasible alternative.

Low-Level Outlets An additional attempt was made to lower the top of the dam by increasing the amount of low-level discharges from the reservoir. In the initial 15 alternatives, a maximum low-level discharge of 10000 cfs was used. As a possible refinement, low-level discharges of 20000 cfs and 30000 cfs were analyzed for some of the

dry reservoirs. These refinements resulted in lowering the top of the dam from 4 to 10 feet depending on the level of protection of the reservoir. However, these refinements required additional concrete box culverts to provide the larger low-level discharges thereby reducing the potential cost savings. The potential impacts downstream of the reservoir from the increased low-level discharges (i.e., reduced benefits) were not determined in this analysis. Table 34 provides the results of this analysis.

TABLE 26
HYDROLOGIC AND HYDRAULIC
ANALYSES OF FINAL ALTERNATIVES

	LARGE WET RESERVOIR W/SERVICE SPILLWAY			DRY RESERVOIR			REDUCED WET RESERVOIR W/SERVICE SPILLWAY		
	25-YR Design	50-YR Design	100-YR Design	25-YR Design	50-YR Design	100-YR Design	25-YR Design	50-YR Design	100-YR Design
Conservation Pool Elev	171.0	171.0	171.0	N/A	N/A	N/A	150.0	150.0	150.0
- Storage (acre-ft)	213,000	213,000	213,000	0	0	0	39,000	39,000	39,000
- Surface Area (ac)	12,800	12,800	12,800	0	0	0	4,400	4,400	4,400
Flood Control Pool Elev	184.0	186.9	189.4	171.0	175.3	178.7	172.8	176.9	180.2
- Storage (acre-ft)	198,000	255,000	304,000	213,000	270,000	320,000	198,000	254,000	303,000
- Surface Area (ac)	18,900	20,500	22,100	12,800	14,600	16,200	13,600	15,300	17,000
Surcharge Pool Elev (PMF max. stage)	200.0	202.0	204.0	193.5	197.0	200.5	190.5	193.0	196.0
- Storage (acre-ft)	371,000	373,000	386,000	399,000	433,000	469,000	298,000	308,000	325,000
- Surface Area (ac)	29,300	30,500	32,000	25,000	27,300	29,500	23,000	24,600	26,600
Total Peak Storage (acre-ft)	782,000	841,000	903,000	612,000	703,000	789,000	535,000	601,000	667,000
Dam Height:									
Earth Embank. w/riprap									
Dam Side slope	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H
Freeboard, ft	7.5'	7.5'	7.5'	7.5'	7.5'	7.5'	7.5'	7.5'	7.5'
Top Elev of Dam	207.5	209.5	211.5	201.0	204.5	208.0	198.0	200.5	203.5
Service Spillway									
No. of Tainter Gates	3	3	3	N/A	N/A	N/A	3	3	3
Size of Gates	40'x37'	40'x40'	40'x42'	N/A	N/A	N/A	40'x36'	40'x40'	40'x44'
Crest Elev	147.5	147.5	147.5	N/A	N/A	N/A	137.0	137.0	137.0
Emergency Spillway									
Crest Elev	184.0	186.9	189.4	171.0	175.3	178.7	172.8	176.9	180.2
Length of Spillway	1000'	1000'	1000'	1000'	1000'	1000'	1000'	1000'	1000'
Low Level Outlet									
Concrete Box Culverts (Inv. Elev = 120.0)									
No. of Culverts	2	2	2	3	3	3	2	2	2
Size of Culverts	10'x12'	10'x12'	10'x12'	10'x10'	10'x10'	9'x10'	10'x12'	10'x12'	10'x12'
Outflow Emerg. Spillway	249,000	235,000	218,000	425,000	410,000	396,000	286,000	263,000	240,000
Outflow Service Spillway	164,000	173,000	179,000	0	0	0	166,000	179,000	188,000
Outflow Low Level Outlet	0	0	0	12,000	12,000	11,000	0	0	0
Max. Total Outflow (cfs)	413,000	408,000	397,000	437,000	422,000	407,000	452,000	442,000	428,000

Note: All Elevations are in Feet NGVD
Storage Pool quantities are incremental

TABLE 26 (Continued)
HYDROLOGIC AND HYDRAULIC
ANALYSES OF FINAL ALTERNATIVES

	LARGE WET RESERVOIR W/O SERVICE SPILLWAY			REDUCED WET RESERVOIR W/O SERVICE SPILLWAY		
	25-YR Design	50-YR Design	100-YR Design	25-YR Design	50-YR Design	100-YR Design
Conservation Pool Elev	171.0	171.0	171.0	150.0	150.0	150.0
- Storage (acre-ft)	213,000	213,000	213,000	39,000	39,000	39,000
- Surface Area (ac)	12,800	12,800	12,800	4,400	4,400	4,400
Flood Control Pool Elev	184.0	186.9	189.4	172.8	176.9	180.2
- Storage (acre-ft)	198,000	255,000	304,000	198,000	254,000	303,000
- Surface Area (ac)	18,900	20,500	22,100	13,600	15,300	17,000
Surcharge Pool Elev (PMF max. stage)	205.0	207.3	209.9	195.3	198.5	201.7
- Storage (acre-ft)	525,000	541,000	572,000	421,000	449,000	490,000
- Surface Area (ac)	32,500	34,300	36,000	26,100	28,000	30,300
Total Peak Storage (acre-ft)	936,000	1,008,000	1,089,000	658,000	742,000	832,000
Dam Height:						
Earth Embank. w/riprap						
Dam Side slope	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H
Freeboard, ft	7.5'	7.5'	7.5'	7.5'	7.5'	7.5'
Top Elev of Dam	212.5	214.8	217.4	202.8	206.0	209.2
Service Spillway						
No. of Tainter Gates	N/A	N/A	N/A	N/A	N/A	N/A
Size of Gates	N/A	N/A	N/A	N/A	N/A	N/A
Crest Elev	N/A	N/A	N/A	N/A	N/A	N/A
Emergency Spillway						
Crest Elev	184.0	186.9	189.4	172.8	176.9	180.2
Length of Spillway	1000'	1000'	1000'	1000'	1000'	1000'
Low Level Outlet						
Concrete Box Culverts (Inv. Elev = 120.0)						
No. of Culverts	3	3	3	3	3	3
Size of Culverts	10'x10'	10'x10'	10'x10'	10'x10'	10'x10'	10'x10'
Outflow Emerg. Spillway	392,000	386,000	375,000	432,000	423,000	408,000
Outflow Service Spillway	0	0	0	0	0	0
Outflow Low Level Outlet	0	0	0	0	0	0
Max. Total Outflow (cfs)	392,000	386,000	375,000	432,000	423,000	408,000

Note: All Elevations are in Feet NGVD
Storage Pool quantities are incremental

TABLE 27

ALTERNATIVE RIVER STAGES (FT. NGVD) @ PORT VINCENT GAGE (MILE 32.9)

Conditions	Frequency Events								
	1-Yr	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	200-Yr	500-Yr
Existing Conditions	6.2	8.7	10.8	12.1	13.2	13.9	14.6	15.2	15.7
Dry Reservoir									
25-Yr Design	6.0	7.2	8.5	9.8	10.9	11.9	12.9	14.2	15.4
50-Yr Design	6.0	7.2	8.5	9.8	10.9	11.7	12.6	13.6	15.0
100-Yr Design	6.0	7.2	8.5	9.8	10.9	11.7	12.2	12.9	14.4
Large Wet Reservoir W/Service Spillway									
25-Yr Design	6.0	7.2	8.5	9.8	10.9	11.7	12.9	14.3	15.4
50-Yr Design	6.0	7.2	8.5	9.8	10.9	11.4	12.4	13.7	15.0
100-Yr Design	6.0	7.2	8.5	9.8	10.9	11.4	12.0	12.9	14.4
Large Wet Reservoir W/O Service Spillway									
25-Yr Design	6.0	7.2	8.5	9.8	10.9	11.9	12.9	14.2	15.4
50-Yr Design	6.0	7.2	8.5	9.8	10.9	11.7	12.6	13.6	15.0
100-Yr Design	6.0	7.2	8.5	9.8	10.9	11.7	12.2	12.9	14.4
Reduced Wet Reservoir W/Service Spillway									
25-Yr Design	6.0	7.2	8.5	9.8	10.9	11.7	12.9	14.3	15.4
50-Yr Design	6.0	7.2	8.5	9.8	10.9	11.4	12.4	13.7	15.0
100-Yr Design	6.0	7.2	8.5	9.8	10.9	11.4	12.0	12.9	14.4
Reduced Wet Reservoir W/O Service Spillway									
25-Yr Design	6.0	7.2	8.5	9.8	10.9	11.9	12.9	14.2	15.4
50-Yr Design	6.0	7.2	8.5	9.8	10.9	11.7	12.6	13.6	15.0
100-Yr Design	6.0	7.2	8.5	9.8	10.9	11.7	12.2	12.9	14.4

TABLE 28

ALTERNATIVE RIVER STAGES (FT. NGVD) @ DENHAM SPRINGS GAGE (MILE 50.1)

Conditions	Frequency Events								
	1-Yr	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	200-Yr	500-Yr
Existing Conditions	29.2	31.9	35.7	38.8	40.5	42.6	43.5	44.4	45.1
Dry Reservoir									
25-Yr Design	27.0	28.8	31.0	32.6	34.0	35.4	37.2	39.2	42.0
50-Yr Design	27.0	28.8	31.0	32.6	34.0	34.8	36.1	38.1	40.8
100-Yr Design	27.0	28.8	31.0	32.6	34.0	34.8	35.6	36.9	39.7
Large Wet Reservoir W/Service Spillway									
25-Yr Design	28.2	29.7	31.7	33.2	34.7	36.3	38.5	41.3	43.5
50-Yr Design	28.2	29.7	31.7	33.2	34.7	35.6	37.0	39.8	42.5
100-Yr Design	28.2	29.7	31.7	33.2	34.7	35.6	36.5	38.3	41.5
Large Wet Reservoir W/O Service Spillway									
25-Yr Design	28.2	29.7	31.7	33.2	34.7	36.0	37.5	39.5	41.2
50-Yr Design	28.2	29.7	31.7	33.2	34.7	35.5	36.8	38.3	39.8
100-Yr Design	28.2	29.7	31.7	33.2	34.7	35.5	36.3	37.2	38.5
Reduced Wet Reservoir W/Service Spillway									
25-Yr Design	28.2	29.7	31.7	33.2	34.7	36.3	38.5	41.3	43.5
50-Yr Design	28.2	29.7	31.7	33.2	34.7	35.6	37.0	39.8	42.5
100-Yr Design	28.2	29.7	31.7	33.2	34.7	35.6	36.5	38.3	41.5
Reduced Wet Reservoir W/O Service Spillway									
25-Yr Design	28.2	29.7	31.7	33.2	34.7	36.0	37.5	39.5	41.2
50-Yr Design	28.2	29.7	31.7	33.2	34.7	35.5	36.8	38.3	39.8
100-Yr Design	28.2	29.7	31.7	33.2	34.7	35.5	36.3	37.2	38.5

TABLE 29

ALTERNATIVE RIVER STAGES (FT. NGVD) @ COMITE RIVER JUNCTION (MILE 50.9)

Conditions	Frequency Events								
	1-Yr	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	200-Yr	500-Yr
Existing Conditions	29.5	32.5	36.0	38.9	41.7	43.3	44.4	45.0	46.8
Dry Reservoir									
25-Yr Design	27.2	29.0	31.2	32.8	34.2	35.6	37.5	39.6	42.7
50-Yr Design	27.2	29.0	31.2	32.8	34.2	35.0	36.4	38.5	41.4
100-Yr Design	27.2	29.0	31.2	32.8	34.2	35.0	35.8	37.4	40.0
Large Wet Reservoir W/Service Spillway									
25-Yr Design	28.4	29.9	31.9	33.4	34.9	36.5	38.9	41.9	44.2
50-Yr Design	28.4	29.9	31.9	33.4	34.9	35.8	38.1	40.2	43.3
100-Yr Design	28.4	29.9	31.9	33.4	34.9	35.8	37.4	38.6	42.1
Large Wet Reservoir W/O Service Spillway									
25-Yr Design	28.4	29.9	31.9	33.4	34.9	36.2	37.8	39.9	41.7
50-Yr Design	28.4	29.9	31.9	33.4	34.9	35.7	37.1	38.7	40.2
100-Yr Design	28.4	29.9	31.9	33.4	34.9	35.7	36.4	37.5	38.9
Reduced Wet Reservoir W/Service Spillway									
25-Yr Design	28.4	29.9	31.9	33.4	34.9	36.5	38.9	41.9	44.2
50-Yr Design	28.4	29.9	31.9	33.4	34.9	35.8	38.1	40.2	43.3
100-Yr Design	28.4	29.9	31.9	33.4	34.9	35.8	37.4	38.6	42.1
Reduced Wet Reservoir W/O Service Spillway									
25-Yr Design	28.4	29.9	31.9	33.4	34.9	36.2	37.8	39.9	41.7
50-Yr Design	28.4	29.9	31.9	33.4	34.9	35.7	37.1	38.7	40.2
100-Yr Design	28.4	29.9	31.9	33.4	34.9	35.7	36.4	37.5	38.9

TABLE 30

ALTERNATIVE RIVER STAGES (FT. NGVD) @ MAGNOLIA GAGE (MILE 56.2)

	Frequency Events								
Conditions	1-Yr	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	200-Yr	500-Yr
Existing Conditions	40.8	44.5	47.7	49.2	51.0	52.1	53.2	54.2	55.3
Dry Reservoir									
25-Yr Design	38.6	39.5	40.7	41.5	42.5	43.3	45.1	48.7	52.1
50-Yr Design	38.6	39.5	40.7	41.5	42.5	42.7	43.8	46.3	50.1
100-Yr Design	38.6	39.5	40.7	41.5	42.5	42.7	43.0	44.6	48.0
Large Wet Reservoir W/Service Spillway									
25-Yr Design	40.6	41.0	41.8	42.8	43.5	45.1	47.8	51.4	54.6
50-Yr Design	40.6	41.0	41.8	42.8	43.5	44.0	45.3	49.0	53.0
100-Yr Design	40.6	41.0	41.8	42.8	43.5	44.0	44.3	46.3	51.3
Large Wet Reservoir W/O Service Spillway									
25-Yr Design	40.6	41.0	41.8	42.8	43.5	44.5	45.8	49.3	52.6
50-Yr Design	40.6	41.0	41.8	42.8	43.5	43.8	44.8	46.8	50.6
100-Yr Design	40.6	41.0	41.8	42.8	43.5	43.8	44.3	45.4	48.5
Reduced Wet Reservoir W/Service Spillway									
25-Yr Design	40.6	41.0	41.8	42.8	43.5	45.1	47.8	51.4	54.6
50-Yr Design	40.6	41.0	41.8	42.8	43.5	44.0	45.3	49.0	53.0
100-Yr Design	40.6	41.0	41.8	42.8	43.5	44.0	44.3	46.3	51.3
Reduced Wet Reservoir W/O Service Spillway									
25-Yr Design	40.6	41.0	41.8	42.8	43.5	44.5	45.8	49.3	52.6
50-Yr Design	40.6	41.0	41.8	42.8	43.5	43.8	44.8	46.8	50.6
100-Yr Design	40.6	41.0	41.8	42.8	43.5	43.8	44.3	45.4	48.5

TABLE 31

ALTERNATIVE RIVER STAGES (FT. NGVD) @ BEAVER BAYOU (COMITE RIVER MILE 3.63)

Conditions	Frequency Events								
	1-Yr	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	200-Yr	500-Yr
Existing Conditions	34.0	35.9	38.4	40.4	42.9	44.9	46.1	47.1	48.0
Dry Reservoir									
25-Yr Design	33.7	35.7	36.1	38.0	39.6	41.7	43.6	44.7	46.5
50-Yr Design	33.7	35.7	36.1	38.0	39.6	41.6	43.3	44.4	46.1
100-Yr Design	33.7	35.7	36.1	38.0	39.6	41.6	43.1	44.2	45.6
Large Wet Reservoir									
W/Service Spillway									
25-Yr Design	33.7	35.8	36.2	38.1	39.8	42.1	43.9	45.6	46.9
50-Yr Design	33.7	35.8	36.2	38.1	39.8	41.9	43.6	44.9	46.6
100-Yr Design	33.7	35.8	36.2	38.1	39.8	41.9	43.4	44.6	46.4
Large Wet Reservoir									
W/O Service Spillway									
25-Yr Design	33.7	35.8	36.2	38.1	39.8	42.0	43.7	44.9	46.0
50-Yr Design	33.7	35.8	36.2	38.1	39.8	41.8	43.4	44.6	45.7
100-Yr Design	33.7	35.8	36.2	38.1	39.8	41.8	43.2	44.3	45.3
Reduced Wet Reservoir									
W/Service Spillway									
25-Yr Design	33.7	35.8	36.2	38.1	39.8	42.1	43.9	45.6	46.9
50-Yr Design	33.7	35.8	36.2	38.1	39.8	41.9	43.6	44.9	46.6
100-Yr Design	33.7	35.8	36.2	38.1	39.8	41.9	43.4	44.6	46.4
Reduced Wet Reservoir									
W/O Service Spillway									
25-Yr Design	33.7	35.8	36.2	38.1	39.8	42.0	43.7	44.9	46.0
50-Yr Design	33.7	35.8	36.2	38.1	39.8	41.8	43.4	44.6	45.7
100-Yr Design	33.7	35.8	36.2	38.1	39.8	41.8	43.2	44.3	45.3

TABLE 32

ALTERNATIVE RIVER STAGES (FT. NGVD) @ HURRICANE CREEK (COMITE RIVER MILE 6.45)

Conditions	Frequency Events								
	1-Yr	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	200-Yr	500-Yr
Existing Conditions	39.0	41.8	44.6	46.9	49.4	51.2	52.5	52.9	53.2
Dry Reservoir									
25-Yr Design				NO STAGE LOWERINGS					
50-Yr Design				NO STAGE LOWERINGS					
100-Yr Design				NO STAGE LOWERINGS					
Large Wet Reservoir									
W/Service Spillway									
25-Yr Design				NO STAGE LOWERINGS					
50-Yr Design				NO STAGE LOWERINGS					
100-Yr Design				NO STAGE LOWERINGS					
Large Wet Reservoir									
W/O Service Spillway									
25-Yr Design				NO STAGE LOWERINGS					
50-Yr Design				NO STAGE LOWERINGS					
100-Yr Design				NO STAGE LOWERINGS					
Reduced Wet Reservoir									
W/Service Spillway									
25-Yr Design				NO STAGE LOWERINGS					
50-Yr Design				NO STAGE LOWERINGS					
100-Yr Design				NO STAGE LOWERINGS					
Reduced Wet Reservoir									
W/O Service Spillway									
25-Yr Design				NO STAGE LOWERINGS					
50-Yr Design				NO STAGE LOWERINGS					
100-Yr Design				NO STAGE LOWERINGS					

TABLE 33

HYDROLOGIC AND HYDRAULIC ANALYSES

WITH ALTERNATIVE EMERGENCY SPILLWAY LENGTHS

DRY RESERVOIR

	10-YR Design			25-YR Design				50-YR Design	
Conservation Pool Elev	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
- Storage (acre-ft)	0	0	0	0	0	0	0	0	0
- Surface Area (ac)	0	0	0	0	0	0	0	0	0
Flood Control Pool Elev	164.5	164.5	164.5	171.0	171.0	171.0	171.0	175.3	175.3
- Storage (acre-ft)				213,000	213,000	213,000	213,000	270,000	270,000
- Surface Area (ac)				12,800	12,800	12,800	12,800	14,600	14,600
Surcharge Pool Elev (PMF max. stage)	187.9	183.6	180.9	193.5	187.0	183.8	181.8	212.0	197.0
- Storage (acre-ft)				399,000					433,000
- Surface Area (ac)				25,000					27,300
Total Peak Storage (acre-ft)				612,000				1,158,000	703,000
Dam Height:									
Earth Embank. w/riprap									
Dam Side slope	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H
Freeboard, ft	7.5'	7.5'	7.5'	7.5'	7.5'	7.5'	7.5'	7.5'	7.5'
Top Elev of Dam	195.5	191.5	188.5	201.0	194.5	191.5	189.5	219.5	204.5
Service Spillway									
No. of Tainter Gates	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Size of Gates	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Crest Elev	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Emergency Spillway									
Crest Elev	164.5	164.5	164.5	171.0	171.0	171.0	171.0	175.3	175.3
Length of Spillway	1000'	1500'	2000'	1000'	2000'	3000'	4000'	200'	1000'
Low Level Outlet									
Concrete Box Culverts (Inv. Elev = 120.0)									
No. of Culverts	3	3	3	3	3	3	3	3	3
Size of Culverts	10'x11'	10'x11'	10'x11'	10'x10'	10'x10'	10'x10'	10'x10'	10'x10'	10'x10'
Outflow Emerg. Spillway				425,000				185,000	410,000
Outflow Service Spillway				0				0	0
Outflow Low Level Outlet				12,000				13,000	12,000
Max. Total Outflow (cfs)	461,000	497,000	516,000	437,000	503,000	524,000	536,000	198,000	422,000

Note: All Elevations are in Feet NGVD

Storage Pool quantities are incremental

Exist. PMP Flow at the Reservoir Site = 495,000 cfs

TABLE 34
HYDROLOGIC AND HYDRAULIC ANALYSES
WITH ALTERNATIVE LOW LEVEL OUTLET FLOWS

********** DRY RESERVOIR **********

	10-YR Design		25-YR Design			50-YR Design	100-YR Design
Maximum Flood Control Release (cfs)	10,000	30,000	10,000	20,000	30,000	10,000	10,000
Flood Control Pool Elev	164.5	153.7	171.0	166.5	162.9	175.3	178.7
Surcharge Pool Elev (PMF max. stage)	187.9	177.8	193.5	189.3	185.7	197.0	200.5
Dam Height:							
Earth Embank. w/riprap							
Dam Side slope	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H	1V:5H
Freeboard, ft	7.5'	7.5'	7.5'	7.5'	7.5'	7.5'	7.5'
Top Elev of Dam	195.5	185.5	201.0	197.0	193.5	204.5	208.0
Emergency Spillway							
Crest Elev	164.5	153.7	171.0	166.5	162.9	175.3	178.7
Length of Spillway	1000'	1000'	1000'	1000'	1000'	1000'	1000'
Low Level Outlet							
Concrete Box Culverts (Inv. Elev = 120.0)							
No. of Culverts	3	9	3	6	9	3	3
Size of Culverts	10'x11'	11'x12'	10'x10'	10'x10'	10'x10'	10'x10'	9'x10'
Max. Total Outflow (cfs)	461,000	490,000	437,000	454,000	466,000	422,000	407,000

Exist. PMP Flow at the Reservoir Site = 495,000 cfs

Note: All Elevations are in Feet NGVD
Storage Pool quantities are incremental

GEOLOGY AND DAM LOCATION

Geology

The proposed reservoir area, located in East Feliciana and St. Helena Parishes, Louisiana is an area of moderate relief consisting of broad uplands and alluvial valleys. These uplands are formed into terraces along the rivers and streams.

Soil borings were taken east-west across the reservoir basin along the centerline of the proposed dam site (see Plate 15). From these borings it was determined that the uplands on both ends of the area consist of interbedded silts and clays in the upper 18-20' and interbedded silty sands to the depths of the borings in the Citronelle terrace formations. The Tertiary contact is below the depths of the borings taken in these areas. The erosional Citronelle terrace surface is of Pliocene to Pleistocene age.

The central portion of the project area, approximately from Stations 40+00 to 120+00, consists of Pleistocene terrace formations and Holocene alluvial deposits along the rivers and streams. The portion above the Tertiary contact consists of interbedded silts, silty sands and sands with occasional lenses of clays and gravels (see Plate 15). The thickness of this upper portion varies from 18 to 70' which is approximately elevation +138 to +60' NGVD. The Miocene deposits (below the Tertiary contact) consist of interbedded clays, silts, and silty sands and one gravel lense in boring DD-AU. These deposits extend to an unknown depth.

Should the project proceed additional borings will need to be taken during the design phase to better define the geology of the area.

DAM LOCATION

The dam is to cross a deep entrenchment of the Amite River between the approximate elevation 200 contours. Locations were evaluated both upstream and downstream of the selected site. It was determined that there is no geologic advantage to moving the site from it's current

location. Roadway and utility relocations are about the same at any alternate location. The site that resulted in the shortest dam length was therefore selected.

GEOTECHNICAL ANALYSIS AND STRUCTURAL DESIGN

GENERAL CRITERIA

In order for the proposed reservoir to be safe and functional, the proposed dam must both serve the hydraulic design needs and be structurally stable. To accomplish this, the design of the dam must satisfy the following criteria:

- The dam must withstand the forces induced by the storage pool under all possible hydrologic conditions and occurrences. It must have sufficient cross-section and material content in order not to slide, settle or slough.

- The dam must not allow excessive seepage underneath or through the structure. Failure to do so will result in undermining which in-turn could cause a catastrophic failure of the dam.

- The dam must be able to pass the probable maximum flood. This requires the inclusion of an emergency spillway to allow controlled release, thereby preventing overtopping the dam. Failure to adequately provide such an outlet could result in the crevassing and catastrophic failure of the structure during this flood event.

- The dam must allow normal river flow. This requires low flow outlets to be built as part of the dam structure.

SITE INVESTIGATION Surveys of the dam centerline were performed resulting in a centerline profile and centerline traverse. Seven soil borings were taken along the centerline and related to the stationing along that traverse. In previous investigations three soil borings were taken on the east side of the Amite River approximately on the centerline traverse. Index type testing was performed upon the representative samples from the soil borings to aide in the classification of the soils. No detailed shear testing was performed pending the

outcome of initial analysis of the dam cross-section and cost estimates. General soil shear strength properties were assumed based upon experience with that soil type. Plotted logs of the seven soil borings are included on Plate 16. Based upon the seven soil borings taken for this study and the three previous soil borings a Geologic cross-section of the valley along the alignment was developed as shown on Plate 15.

Preliminary Design Considerations and Selection of Dam Type.

Two types of dam structures are appropriate for this application - roller compacted concrete and earthen section. Analysis revealed that while the roller compacted concrete dam is smaller in material section than an earthen structure, its heavier unit weight requires a pile supported foundation given the existing soil conditions. The cost associated with the required pile foundation makes the roller compacted concrete dam's overall cost far excessive relative to an earthen structure of equal height. The earthen section dam structure was therefore selected for further study.

DESIGN ANALYSES. Detailed design analyses based upon assumed foundation and embankment strengths were performed to determine dimensions for the full size wet reservoir dam to be constructed to a crest elevation of 211.5 ft. NGVD (National Geodetic Vertical Datum). Dimensions were estimated and randomly checked for two alternative dam heights constructed to elevations of 201.0 and 191.5 ft. NGVD. This corresponds to smaller sized dry reservoir alternatives.

General Design. Soil environments were subdivided as shown on the Geologic cross-section based upon height of embankment and soil conditions. Similar soils deposits and thickness of soil type were used to select the soils reaches.

Shear Stability Analyses. These analyses were performed using the method of planes, based upon best estimates of the stratification and shear strength as determined from the logs of soil borings, as shown on the analysis. See Plates 17-23. Construction case analyses were performed

and a cross-section necessary to obtain a 1.30 safety factor was developed as shown on Plates 17-23. These cross-sections were developed for a crown elevation of 211.5 and a 28 foot crown width, for the soil environments/reaches shown on the geologic cross-section. Sudden draw-down and steady seepage analyses were also performed. The design safety factor was 1.0 for the sudden drawdown analyses. Additionally, a long term steady state seepage condition was analyzed for a selected reach. The design assumptions for this long term analysis were: friction angel in the silt (random back fill and first layer beneath the dam) of 28 degrees, unit weigh of 115 pounds per cubic foot, and cohesion of 0 pounds per square foot. The minimum design safety factor for this case was 1.5 and a minimum safety factor of 2.6 was found. In all cases analyzed, the construction case (short term) governed the cross-sections of the dams.

Through and Under Seepage Design. The predominant soil type in the area available for borrow is silt and that soil is pervious to water flow. A core of impervious clay was considered to reduce through seepage of the dam as shown on the stability analysis plates and alternate design Plates 24-27. In addition, a collector drain of gravel will be required to reduce the hydraulic gradient through the dam as shown on Plates 24-27. The collector drain will be separated from the adjacent soils by a geotextile. These collector drains will exit on the downstream toe into a collection ditch. When the design of the collector drain is finalized, a thinner drain section will likely result, however at this time it did not appear prudent to pursue further. To reduce under seepage a slurry trench cut-off wall is proposed to extend 5 feet into the stiff to hard clay shown on the geologic cross-section for the full length of the dam.

Spillway Sliding Analysis and Design. The spillway was originally planned on the east bank of the river, near the center of the dam, to facilitate entrance and exit conditions of the water. The CASE computer program "CSLIDE" was used to analyze the sliding stability of the spillway. It was found that in order to provide a safety factor against sliding of 1.30 in the S-Case (long term), with an ineffective drainage blanket, the heel of the structure had to be placed at elevation 42.0. The large

amount of concrete required to construct at this location required the spillway to be relocated nearer to the eastern abutment on the terrace at approximate elevation 170. Although this location required a longer entrance and exit channel to be excavated, the base elevation could be raised significantly as shown on Plate 28 thereby reducing the overall spillway cost. The spillway has a 30 foot deep cut-off wall of steel sheet pile at both the entrance side and the exit side to prevent local undermining. An underdrain system of collector pipes placed in an inverted filter 2 feet thick was also planned to relieve uplift conditions. The filter is placed upon a separator of geotextile. The collector pipes will drain into manholes then exit into the tail bay.

The spillway is to be constructed of roller compacted concrete. The top surface is to be lined with reinforced concrete in order to provide a durable wearing surface. At design flows, which will produce velocities of 40-60 feet per second, the roller compacted concrete would be subject to fast erosion and possible failure without this wearing surface.

Summary of Dam Structure Design Earthen Sections.

Design cross-sections for a dam with top elevation 211.5 are shown on Plates 17-23. In general, the side and berm slopes shown must be used for the section design of all other dam alternative heights in order to achieve required structural safety and stability. For all alternatives, the clay core and slurry trench wall shown in these drawings are also required to sufficiently block through and under structure seepage. The gravel drain system shown on Plates 24-27 is also required in all dam alternatives.

Spillways and Outlets.

The cost of a service spillway for the Wet and Reduced Wet Alternatives was determined to be in excess of $100,000,000. Costs associated with higher dam heights required without the service spillways were determined to be for less. As discussed above is the Hydraulics Analyses, service spillways are not essential to the operation of a reservoir with a permanent lake. In

consideration of the above, it was determined that the Wet and Reduced Wet Dam Alternatives should not include service spillways.

Again, as discussed above in the Hydraulics Analyses, the design optimization of the emergency spillway and low level outlets indicates that a 1,000 foot long emergency spillway and (3) 10 foot by 10 foot concrete box culverts low level outlet provide the most cost-effective means to convey both peak and normal flows for all dam alternatives under consideration.

Design Summary. A plan view of a typical dam structure is shown on Plate 29. As discussed above, the Wet, Reduced Wet and Dry Dam Alternatives will essentially differ only in height and overall section width. All dams will have a 1,000 foot long emergency spillway and (3) 10 foot by 10 foot concrete box culverts low flow outlet. A typical cross-section of the emergency spillway is shown on Plate 28 and a typical low flow outlet cross-section is shown on Plate 30. No service spillway is included in any final dam alternative.

COSTS

Structure Costs. Itemized costs for the earthen dam structure for dam heights of 211.5, 201.0 and 191.5 are shown respectively in Tables 35-37. Cost breakdowns for the 1,000 foot long emergency spillway and the (3) 10 foot by 10 foot concrete box culvert low flow outlet, which are part of all dam alternatives, are shown in Tables 38 and 39, respectively.

Cost estimates were prepared based upon an evaluation of historical data (including previous M-CACES projects) or abbreviated (non-detailed) estimating procedures.

From the above set of three dam structure costs, a cost curve was developed as shown in Figure 1. Alternative dam designs are illustrated on this curve. It can be seen that the dam structure cost rises geometrically relative to design height. This is not surprising since the material volume of the dam structure also increases geometrically relative to height.

Reservoir Clearing Costs. It will be necessary that a substantial amount of timber and debris be removed from the reservoir area prior to its operation. A brief investigation was done to evaluate this cost. It was determined that the salvage value of the usable timber will offset most of the costs associated with its removal as well as the removal of other debris. No net clearing cost was therefore included in the final estimates. The removal cost of utilities and structures is included in relocations and real estate costs.

Annual Operation and Maintenance Costs. Since tainter gates are not included in the final dam designs for all alternatives, annual operation and maintenance for the dam structure itself will be limited to grass cutting and erosion repairs. Relative to each other, there will be more erosion repair needed with a wet reservoir due to constant wave action associated with the permanent lake.

Annual operation and maintenance costs for a full size wet reservoir dam structure, not including recreational facilities and environmental mitigation items, is estimated at $175,000. Annual operation and maintenance for a dry reservoir dam structure is estimated at $85,000.

TABLE 35

ESTIMATED DAM STRUCTURE COSTS-ELEVATION 211.5

Item	Quantity	Unit	Unit Price	Amount	Contingencies	Project Cost
MAIN DAM						
MOB & DEMOB	LS	——	100,000.00	$100,000	$20,000	$120,000
CARE AND DIVERSION OF WATER						
COFFERDAM						
[Clearing & Grubbing						
Semicompacted Fill						
Seeding & Fertilizing	LS	——	500,000.00	$500,000	$150,000	$650,000
Dewatering System						
Cofferdam Removal]						
EARTHWORK FOR STRUCTURE						
SITE WORK (EXCAVATION)						
Clearing & Grubbing	345.0	ACRE	1,200.00	$414,000	$124,200	$538,200
Common Excavation -						
Adjacent Disposal	450,000.0	CUYD	1.50	$675,000	$202,500	$877,500
EARTHEN DAM						
SITE WORK (DRAINAGE)						
Slurry Trench Excavation-						
70' Avg. Depth	1,260,000.0	SQFT	15.00	$18,900,000	$5,670,000	$24,570,000
Gravel Filter Material	1,650,000.0	CUYD	20.00	$33,000,000	$9,900,000	$42,900,000
Filter Fabric	810,000.0	SQYD	4.50	$3,645,000	$1,093,500	$4,738,500
SITE WORK (BACKFILL)						
Semicompacted Random	14,000,000.0	CUYD	3.50	$49,000,000	$14,700,000	$63,700,000
Fully Compacted Select						
Clay	1,400,000.0	CUYD	5.00	$7,000,000	$2,100,000	$9,100,000
SITE WORK (MISCELLANEOUS)						
Seeding & Fertilizing	350.0	ACRE	500.00	$175,000	$52,500	$227,500
SUBTOTAL				$113,409,000	$34,012,700	$147,421,700
ENGINEERING AND DESIGN (6%)						$8,845,300
CONSTRUCTION MANAGEMENT (6%)						$8,845,300
TOTAL						$165,112,300

TABLE 36

ESTIMATED DAM STRUCTURE COSTS-ELEVATION 201.0

Item	Quantity	Unit	Unit Price	Amount	Contingencies	Project Cost
MAIN DAM						
MOB & DEMOB	LS	——	100,000.00	$100,000	$20,000	$120,000
CARE AND DIVERSION OF WATER						
COFFERDAM						
[Clearing & Grubbing						
Semicompacted Fill						
Seeding & Fertilizing	LS	——	500,000.00	$500,000	$150,000	$650,000
Dewatering System						
Cofferdam Removal]						
EARTHWORK FOR STRUCTURE						
SITE WORK (EXCAVATION)						
Clearing & Grubbing	270.0	ACRE	1,200.00	$324,000	$81,100	$405,100
Common Excavation -						
Adjacent Disposal	255,000.0	CUYD	1.50	$382,500	$95,700	$478,200
EARTHEN DAM						
SITE WORK (DRAINAGE)						
Slurry Trench Excavation-						
70' Avg. Depth	1,260,000.0	SQFT	15.00	$18,900,000	$4,731,300	$23,631,300
Gravel Filter Material	1,165,000.0	CUYD	16.00	$18,640,000	$4,665,600	$23,305,600
Filter Fabric	635,000.0	SQYD	4.50	$2,857,500	$715,200	$3,572,700
SITE WORK (BACKFILL)						
Semicompacted Random	9,010,000.0	CUYD	2.50	$22,525,000	$5,638,600	$28,163,600
Fully Compacted Select						
Clay	1,040,000.0	CUYD	5.00	$5,200,000	$1,301,600	$6,501,600
SITE WORK (MISCELLANEOUS)						
Seeding & Fertilizing	275.0	ACRE	500.00	$137,500	$34,400	$171,900
SUBTOTAL				$69,566,500	$17,433,500	$87,000,000
ENGINEERING AND DESIGN (6%)						$5,220,000
CONSTRUCTION MANAGEMENT (6%)						$5,220,000
TOTAL						$97,440,000

TABLE 37

ESTIMATED DAM STRUCTURE COSTS-ELEVATION 191.5

Item	Quantity	Unit	Unit Price	Amount	Contingencies	Project Cost
MAIN DAM						
MOB & DEMOB	LS	——	100,000.00	$100,000	$20,000	$120,000
CORE AND DIVERSION OF WATER						
COFFERDAM						
[Clearing & Grubbing						
Semicompacted Fill						
Seeding & Fertilizing	LS	——	500,000.00	$500,000	$150,000	$650,000
Dewatering System						
Cofferdam Removal]						
EARTHWORK FOR STRUCTURE						
SITE WORK (EXCAVATION)						
Clearing & Grubbing	200.0	ACRE	1,200.00	$240,000	$60,400	$300,400
Common Excavation -						
Adjacent Disposal	250,000.0	CUYD	1.50	$375,000	$94,300	$469,300
EARTHEN DAM						
SITE WORK (DRAINAGE)						
Slurry Trench Excavation-						
70' Avg. Depth	1,260,000.0	SQFT	15.00	$18,900,000	$4,754,000	$23,654,000
Gravel Filter Material	860,000.0	CUYD	16.00	$13,760,000	$3,461,000	$17,221,000
Filter Fabric	460,000.0	SQYD	4.50	$2,070,000	$520,600	$2,590,600
SITE WORK (BACKFILL)						
Semicompacted Random	5,300,000.0	CUYD	2.50	$13,250,000	$3,332,500	$16,582,500
Fully Compacted Select						
Clay	765,000.0	CUYD	5.00	$3,825,000	$962,000	$4,787,000
SITE WORK (MISCELLANEOUS)						
Seeding & Fertilizing	200.0	ACRE	500.00	$100,000	$25,200	$125,200
SUBTOTAL				$53,120,000	$13,380,000	$66,500,000
ENGINEERING AND DESIGN (6%)						$3,990,000
CONSTRUCTION MANAGEMENT (6%)						$3,990,000
TOTAL						$74,480,000

TABLE 38

ESTIMATED EMERGENCY SPILLWAY COSTS - ELEVATION 201.0 DAM

Item	Quantity	Unit	Unit Price	Amount	Contingencies	Project Cost
SPILLWAY						
MOB & DEMOB	LS	——	100,000.00	$100,000	$20,000	$120,000
ACCESS ROADS	LS	——	50,000.00	$50,000	$12,500	$62,5000
CARE AND DIVERSION OF WATER						
SITE WORK (MISCELLANEOUS)						
Dewatering System - Sumps and Pumps	LS	——	150,000.00	$150,000	$45,000	$195,000
EARTHWORK FOR STRUCTURE						
SITE WORK (EXCAVATION)						
Clearing & Grubbing	20.0	ACRE	1,200.00	$24,000	$6,000	$30,000
Structural Excavation - Adjacent Disposal	600,000.0	CUYD	3.50	$2,100,000	$527,100	$2,627,100
SITE WORK (BACKFILL)						
Semicompacted Random	15,000.0	CUYD	2.50	$37,500	$9,400	$46,900
Fully Compacted Select Clay	115,000.0	CUYD	5.00	$575,000	$144,600	$719,600
Fully Compacted Select Sand	26,000.0	CUYD	7.00	$182,000	$45,800	$227,800
SITE WORK (RIPRAP - DRY)						
42" Riprap	123,000.0	TONS	25.00	$3,075,000	$773,400	$3,848,400
6" Bedding Material	12,000.0	CUYD	25.00	$300,000	$75,500	$375,500
FOUNDATION WORK						
SITE WORK (PILING)						
PZ-27 Sheet Pile	33,000.0	SQFT	13.50	$445,500	$111,800	$557,300
CONCRETE						
Sand Cement Foundation Treatment	9,000.0	CUYD	18.00	$162,000	$40,700	$202,700

TABLE 38 (Continued)

ESTIMATED EMERGENCY SPILLWAY COSTS - ELEVATION 201.0 DAM

Item	Quantity	Unit	Unit Price	Amount	Contingencies	Project Cost
DRAINAGE						
SITE WORK (MISCELLANEOUS)						
Slurry Trench Excavation - 75' Avg. Depth	76,000.0	SQFT	15.00	$1,140,000	$286,700	$1,426,700
Gravel Filter Material	34,000.0	CUYD	20.00	$680,000	$171,000	$851,000
6" Perforated PVC Pipe	46,000.0	LNFT	10.00	$460,000	$115,700	$575,700
12" PVC Pipe	1,800.0	LNFT	14.00	$25,200	$6,300	$31,500
SPILLWAY CHANNEL						
SITE WORK (EXCAVATION)						
Clearing & Grubbing	100.0	ACRE	1,200.00	$120,000	$30,200	$150,200
Common Excavation	6,200,000.0	CUYD	1.50	$9,300,000	$2,343,200	$11,643,200
CONCRETE OVERFLOW SECTION						
CONCRETE						
Reinforced Concrete Overlay	50,000.0	CUYD	300.00	$15,000,000	$3,779,800	$18,779,800
Roller Compacted Concrete	180,000.0	CUYD	70.00	$12,600,000	$3,175,000	$15,775,000
Dowels	290,000.0	LBS	.70	$203,000	$51,100	$254,100
SUBTOTAL				$46,729,200	$11,770,800	$58,500,000
ENGINEERING AND DESIGN (6%)						$3,510,000
CONSTRUCTION MANAGEMENT (6%)						$3,510,000
TOTAL						$65,520,000

TABLE 39

ESTIMATED LOW LEVEL OUTLET COSTS - ELEVATION 211.5 DAM

Item	Quantity	Unit	Unit Price	Amount	Contingencies	Project Cost
ACCESS ROADS	LS	——	50,000.00	$50,000	$12,500	$62,500
CARE AND DIVERSION OF WATER						
SITE WORK (MISCELLANEOUS)						
Dewatering System-	LS	——	150,000.00	$150,000	$45,000	$195,000
Sumps and Pumps						
EARTHWORK FOR STRUCTURE						
SITE WORK (EXCAVATION)						
Clearing & Grubbing	****					
Structural Excavation -	120,000.0		3.50	$420,000	$105,000	$525,000
Adjacent Disposal						
SITE WORK (BACKFILL)						
Semicompacted Random	****					
Fully Compacted Select Clay	****					
Fully Compacted Select Sand	****					
FOUNDATION WORK						
SITE WORK (PILING)						
PZ-22 Sheet Pile	5,000.0	SQFT	12.50	$62,500	$15,600	$78,100
INLET AND OUTLET CHANNELS						
SITE WORK (RIPRAP - DRY)						
36" Riprap	15,000.0	TONS	25.00	$375,000	$93,800	$468,800
24" Riprap	4,700.0	TONS	25.00	$117,500	$29,400	$146,900
6" Bedding Material	2,500.0	CUYD	20.00	$50,000	$12,500	$62,500
Filter Fabric	22,000.0	SQYD	4.50	$99,000	$24,800	$123,800
SITE WORK (EXCAVATION)						
Clearing & Grubbing	10.0	ACRE	1,200.00	$12,000	$3,000	$15,000
Common Excavation	120,000.0	CUYD	1.50	$180,000	$45,000	$225,000
SEEPAGE CONTROL						
SITE WORK (MISCELLANEOUS)						
Slurry Trench Excavation -	****					
75' Avg. Depth						

TABLE 39 (Continued)

ESTIMATED LOW LEVEL OUTLET COSTS - ELEVATION 211.5 DAM

Item	Quantity	Unit	Unit Price	Amount	Contingencies	Project Cost
CULVERT STRUCTURE						
REINFORCED CONCRETE						
Slab	7,300.0	CUYD	200.0	$1,460,000	$365,000	$1,825,000
Wall & Roof	10,400.0	CUYD	330.0	$3,432,000	$858,000	$4,290,000
Tower	380.0	CUYD	350.0	$133,000	$34,000	$167,000
Collars	750.0	CUYD	330.0	$247,500	$61,900	$309,400
Stop Logs	60.0	CUYD	250.0	$15,000	$4,000	$19,000
UNREINFORCED CONCRETE						
Stab. Slab	650.0	CUYD	70.0	$45,500	$11,400	$56,900
MISCELLANEOUS ITEMS						
24" Vent Pipe	1,600.0	LNFT	50.0	$80,000	$20,000	$100,000
3-Bulb Waterstop	3,500.0	LNFT	10.0	$35,000	$8,800	$43,800
Expansion Joint Filler	11,000.0	SQFT	2.0	$22,000	$5,500	$27,500
METALS						
Trash Racks	30,000.0	LBS	1.50	$45,000	$11,300	$56,300
GATES & EQUIPMENT						
METALS						
Sluice Gates	3.0	EACH	100,000.00	$300,000	$75,000	$375,000
MECHANICAL						
Gate Operating Machinery	3.0	EACH	10,000.00	$30,000	$7,500	$37,500
SUBTOTAL				$7,461,000	$1,869,000	$9,210,000
Engineering and Design (6%)						$552,600
Construction Management (6%)						$552,600
TOTAL						$10,315,200

**** (Included in dam structure estimate)

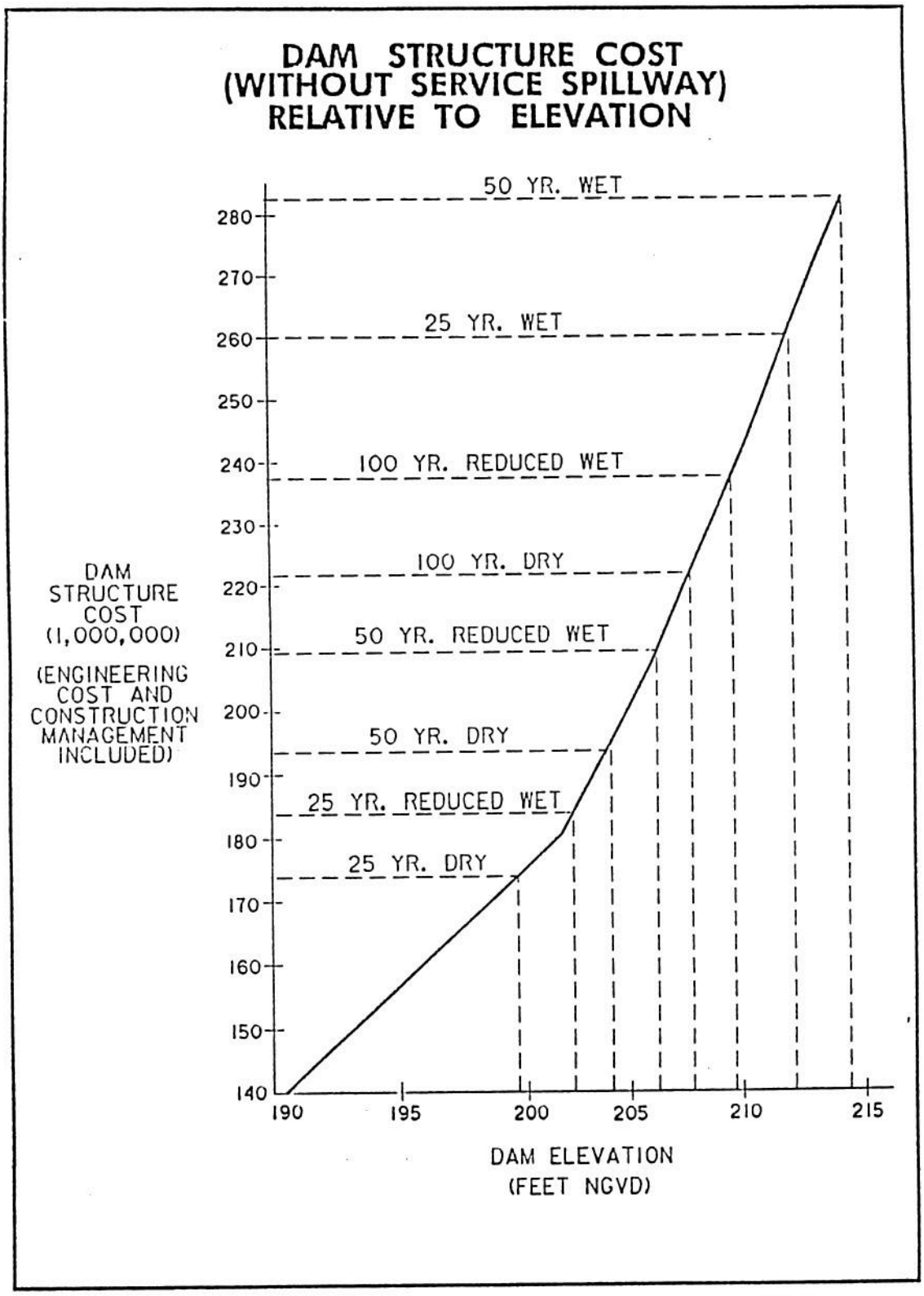

DAM STRUCTURE COST
(WITHOUT SERVICE SPILLWAY)
RELATIVE TO ELEVATION
50 YR. WET
25 YR. WET
100 YR. REDUCED WET
100 YR. DRY
50 YR. REDUCED WET
50 YR. DRY
25 YR. REDUCED WET
25 YR. DRY
DAM STRUCTURE COST (1,000,000)
(ENGINEERING COST AND CONSTRUCTION MANAGEMENT INCLUDED)
280
270
260
250
240
230
220
210
200
190
180
170
160
150
140
190
195
200
205
210
215
DAM ELEVATION (FEET NGVD)

FIGURE 1

ROADWAY AND UTILITY RELOCATIONS

SUMMARY

The estimated total roadway and utility relocation costs for each 50 year design project alternative are:

Alternative	Total Estimated Relocation Costs (with complete in-kind replacement of all roadways)	Total Estimated Relocation Costs (with all optional reductions in road service)
Wet Reservoir; with and without service spillway	$69,734,000	$32,142,000
Reduced Wet Reservoir; with and without service spillway	$42,799,000	$16,752,000
Dry Reservoir	$33,631,000	$9,209,000

These costs include 12% for the owner's engineering, 10% for the owner's construction administration, plus, 25% for contingencies. Itemized relocation costs for each reservoir alternative are shown in Tables 40-42.

In accordance with project authority the local sponsor (State of Louisiana) would fund and effect relocations and alterations of buildings, utilities, highways, bridges (except railroads), sewers, and related special facilities required for construction of a selected plan.

CRITERIA

Roads and Bridges.

Existing highways and roads that traverse the proposed reservoir will either be re-routed or raised to accommodate a 50 year flood event in accordance with Louisiana State

Department of Transportation and Development standards. Roads that only provide access to areas inside the reservoir limits will be abandoned.

The proposed design elevation of the top surface of replacement roads and the bottom of the stringer beams of replacement bridges will be the design 50 year flood elevation plus an additional two feet of freeboard.

Railroads.

There are no railroads located in the project limits.

Utilities.

(Pipelines)

Pipelines located under proposed permanent water will require weighting to offset negative buoyancy. The most feasible method of in-situ pipeline weighting appears to be by installing pre-cast set-on concrete river weights.

Since they are continuously buoyant, natural gas carriers require weighting up to a reasonable flood event elevation. The 100 year flood elevation plus two feet was chosen as this limit. Liquid carriers that are not normally buoyant will require weighting in frequently flooded areas only. Five feet above the proposed permanent water level was chosen for this limit.

Pipeline reaches under existing river and creek beds that are already weighted were discounted.

(Power and Communication Lines)

Vertical clearances for power and communication lines must be in accordance with ER 1110-2-4401, dated 5 September 1986. Briefly, this regulation states that minimum vertical clearances are to be set above the total

design capacity level of the reservoir excluding surcharge. For the purposes of this study, this reference level is the 50 year flood event pool elevation. This regulation lists specific minimum clearances where sailboats are commonly operated. Without sailboating, minimum clearances are to be set above the reference level in accordance with the National Electric Safety Code. In the "Wet" and "Reduced Wet" Alternatives, sailboating is expected in pool areas south of LA Highway 10.

FACILITY DESCRIPTION, PROPOSED RELOCATIONS, AND COSTS

Wet Reservoirs Alternative (with and without service spillway).

(Since flood elevations affecting relocation criteria are the same for both Wet Alternatives, i.e., with and without the proposed service spillway, the required relocations for each are identical.)

(Roads and Bridges)

The estimated total cost for complete in-kind relocations of all affected roads in this alternative is $42,333,000. Possible reductions in road service, such as detours and class downgrading, can reduce this cost down to $4,741,000. The significant trade-offs associated with these options are discussed below. Roadway relocations for this plan are illustrated on Plate 31.

- LA Highway 960

In all proposed reservoir alternatives the west end of the dam will intersect this highway. A one mile relocation, that includes new rights-of-way, around the dam will be required. The estimated cost is $1,254,000.

- LA Highway 448

In all proposed reservoir alternatives the dam and flood pool will affect this road which connects LA Hwy 37 and LA Hwy 10. The most feasible relocation of LA 448 is to permanently re-route traffic to an existing parish road located about three miles east of its current location. About 8.5 miles of this road can be upgraded from class RC-1 to RC-2 to match the existing classification of LA 448, or, this road can be utilized as is. The estimated cost of this optional upgrading, that includes new rights-of-way, is $14,700,000.

- LA Highway 10; Darling Creek Reach

About one mile of this highway must be raised approximately 10 feet to accommodate the 50 year flood event in this alternative. There is one, 350 foot long bridge in this reach that will require replacement and raising. The estimated cost of raising this reach, including the replacement bridge, is $4,148,000.

- LA Highway 10; Amite River Basin Reach

There are several relocation options available for this reach which is about 2.7 miles long. First, the existing highway can be replaced with a causeway bridge, about 25 feet above ground on average, for its entire length. While this option is the most expensive at an estimated $46,970,000, it will provide an in-kind replacement of this highway reach that will require very low maintenance.

The second option is to raise this reach by means of embankment in conjunction with replacing and raising five existing bridges. This plan will also provide an in-kind replacement of the highway and has a lower estimated first cost of $21,638,000. With this option, however, there would be significant maintenance costs associated with grass cutting and periodic bank repairs. The significant first cost savings makes this the most cost effective in-kind replacement plan.

A third relocation plan is to permanently re-route traffic by detouring on LA 38 from Coleman Town to LA 432, to LA 961 back to LA 10. About twelve miles total of LA 432 and LA 961 must be upgraded from RC-2 to RC-3 to match the existing classification of LA 10. The estimated cost of this option with the above upgrading that includes new rights-of-way is $19,093,000. There are significant reductions in road way service with this relocations option. This detour route is about eight miles longer than this existing reach of LA 10 and will require an additional traffic stop at the intersection of LA 38 and 432. The significant increase in traffic volume at this intersection will also be a problem to consider.

The above traffic re-routing can also be accomplished without upgrading LA 432 and 961. In addition to the above listed shortcomings of this detour, this reach will also be downgraded in classification. This minimum relocation can be done at virtually no cost.

- East Feliciana Parish Road 42; Collins Creek Crossing

This reservoir alternative will affect this bridge and about 1,000 feet of approach roadway. A replacement bridge and approach road raising will be required to accommodate a 50 year flood event. This road provides access to LA Hwy 10 for the remaining residents south of this location. This estimated cost is $593,000.

(Utilities)

The estimated cost for the relocation of all affected utilities for this alternative is $27,401,000. Utility relocations for this plan are shown on Plate 32.

- Colonial Pipeline Co.

This owner has two, 36-inch petroleum product pipelines

that cross the proposed permanent pool. These pipelines will require weighting to offset negative buoyancy if and when emptied of product. The most feasible method of accomplishing this appears to be by installing set-on pre-cast concrete river weights. About 6,000 feet* of each line must be weighted to accommodate pipeline evacuation up to a level 5 feet above the proposed normal level of the lake. The estimated cost is $3,050,000.

Permanent submersion of these lines may cause future repairs to be significantly more costly than would be under existing conditions. While it is not reasonable to assume that a pipeline failure will occur, it is possible. This will require further consideration if this alternative is selected.

Associated with these pipelines, Colonial also owns a major pump station located just inside the western boundary of the proposed reservoir. It appears that a flood elevation up to 202.0 feet will not damage this facility. This corresponds to a very infrequent event of 200± years. While it may not be cost effective to relocate or protect this facility for such an infrequent event, there is the possibility of property and commerce loss. This item will also require further consideration if this alternative is selected.

- Transcontinental Gas Pipeline Co.

This owner has one, 30-inch and two, 36-inch natural gas pipelines that cross the proposed reservoir. These lines will require weighting up to a reasonable flood event elevation. About 9,500 feet* of each line must be weighted to accommodate a 100 year flood event plus two feet. The estimated cost is $6,469,000.

As with the above pipeline owner, the possibility of underwater line repair costs require further consideration if this alternative is selected.

* existing weighted reaches discounted

o Removal of Local Distribution Power and Communication Lines

In this alternative approximately 23 miles of local distribution aerial power and phone lines, including poles, will require removal at an estimated cost of $107,000.

o Relocation of Power and Communication Lines Along LA Hwy 960

In all project alternatives, about one mile of aerial power and communication lines will require relocation in association with this highway relocation. Relocated lines would be placed in new highway rights-of-way. The estimated cost is $137,000.

o Gulf States Utilities (GSU)

GSU owns a major 230/500 kv transmission powerline that runs north-south through the entire proposed reservoir length close to the western boundary.

In the area south of LA Hwy 10, sailboating is possible in the proposed lake. In accordance with ER 1110-2-4401, this line would require raising of about 70 feet to accommodate this activity. It appears to be far more feasible to relocate this line outside the reservoir limits in this area. The slightly longer route and the cost of extra right-of-way is offset by the excessive height requirements for this line to remain in its present alignment.

North of LA Hwy 10 sailboating is not anticipated because of shallow water depths. In this area the transmission line must be raised above the 50 year flood elevation to meet National Electric Code Standards. It

appears that relocating this portion of the line just inside the western boundary of the reservoir is more feasible than raising it along its present location.

The total length of new transmission line for both sections is about 10 miles. The estimated cost of removing the affected transmission line and installing both replacement sections is $15,719,000. This cost includes new rights-of-way.

- Dixie Electric Membership Company (DEMCO)

DEMCO owns a 69 kv local transmission powerline that runs across the proposed reservoir from Hatchersville northeast to Darlington. Like the above GSU line, it appears to be far more feasible to relocate this line outside the reservoir limits south of LA Hwy 10 than raising it to accommodate sailboating. The replacement line can share the new GSU towers up to LA Hwy 10. New poles, which can also accommodate other local lines, will be required along the north side of LA Hwy 10 to bring this line east to Darlington.

The total length of replacement is about 11 miles. The estimated cost of removal and relocation is $1,728,000.

- Relocation of Power and Communication Lines Along LA Hwy 10

About 3.5 miles of local distribution aerial power and phone lines must be raised along LA Hwy 10. These lines can be attached to the new poles required for the above DEMCO transmission powerline at an estimated cost of $142,000.

- Relocation of Power and Communication Lines Along LA Hwy 432

About one mile of local distribution aerial power and

phone lines must be raised about five feet. Extensions to existing poles can be used at an estimated cost of $49,000.

Reduced Wet Reservoir Alternative (with and without service spillway)

(Since flood elevations affecting relocation criteria are the same for both Wet Alternatives, i.e., with and without the proposed service spillway, the required relocations for each are identical.)

(Roads and Bridges)

The estimated total cost for complete in-kind relocations of all affected roads in this alternative is $27,301,000. Possible reductions in road service, such as detours and class downgrading, can reduce this cost down to $1,254,000. Significant trade-offs associated with these options are discussed below. Roadway relocations for this plan are shown on Plate 33.

- LA Highway 960

In all proposed reservoir alternatives the west end of the dam will intersect this highway. A one mile relocation, that includes new rights-of-way, around the dam will be required. The estimated cost is $1,254,000.

- LA Highway 448

In all proposed reservoir alternatives the dam and flood pool will affect this road which connects LA Hwy 37 and LA Hwy 10. The most feasible relocation of LA 448 is to permanently re-route traffic to an existing parish road located about three miles east of its current location. About 8.5 miles can be upgraded from class RC-1 to RC-2 to match the existing classification of LA 448, or, this road can be utilized as is. The estimated cost of this optional upgrading, that includes new rights-of-way, is $14,700,000.

- LA Highway 10; Darling Creek Reach

This road elevation in this reach is above the design 50 year flood elevation. No raising is required.

- LA Highway 10; Amite River Basin Reach

There are several relocation options available for this reach which is about 2.2 miles long. First, the existing highway can be raised, about 15 feet above existing ground on average, by means of embankment in conjunction with replacing and raising five existing bridges. This option will provide an in-kind replacement of the highway and has an estimated cost of $11,347,000.

For the height required for this raising, replacement with a causeway bridge completely across the basin is not feasible.

As with the Wet Alternative, it is also possible to permanently re-route traffic by detouring on LA 38 from Coleman Town to LA 432, to LA 961 back to LA 10. About twelve miles total of LA 432 and LA 961 must be upgraded from RC-2 to RC-3 to match the existing classification of LA 10. The estimated cost of this option with the above upgrading that includes new rights-of-way is $19,093,000. Aside from being more expensive than the above plan, there are significant reductions in roadway service with this relocation option. This detour route is about eight miles longer than this reach of LA 10 and will require an additional traffic stop at the intersection of LA 38 and LA 432. The significant increase in traffic volume at this intersection will also be problem to consider.

The above traffic re-routing can also be accomplished without upgrading LA 432 and 961. In addition to the above shortcomings of this detour, this reach will also be downgraded in classification. This minimum relocation can

be done at virtually no cost.

- East Feliciana Parish Road 42; Collins Creek Crossing

This reservoir alternative will not affect this bridge.

(Utilities)

The estimated total cost for the relocation of all affected utilities for this alternative is $27,401,000. Utility relocations for this plan are shown on Plate 34.

- Colonial Pipeline Co.

This owner's two petroleum product pipelines are not normally buoyant and will be submerged only by infrequent flood events in this alternative. Weight installation will therefore not be required.

- Transcontinental Gas Pipeline Co.

This owner has one, 30-inch and two, 36-inch natural gas pipelines that cross the proposed reservoir. Since these lines are continuously buoyant, they will require weighting up to a reasonable flood event elevation. Discounting existing weighted lengths, about 9,000 feet of each line must be weighted to accommodate a 100 year flood event. The estimated cost is $6,129,000.

- Removal of Local Distribution Power and Communication Lines

In this alternative about 22 miles of local distribution aerial power and phone lines, including poles, will require removal at a cost of $101,000.

- Relocation of Power and Communication Lines Along LA Hwy 960

In all project alternatives, about one mile of aerial power and communication lines will require relocation in association with this highway relocation. Relocated lines will be placed in new highway rights-of-way. The estimated cost is $137,000.

- Gulf States Utilities (GSU)

GSU owns a major 230/500 kv transmission powerline that runs north-south through the entire proposed reservoir length close to the western boundary.

In the area south of LA Hwy 10, sailboating is possible in the proposed lake. In accordance with ER 1110-2-4401, this line would require raising of about 60 feet to accommodate this activity. It appears to be far more feasible to relocate this line outside the reservoir limits in this area. The slightly longer route and the cost of extra right-of-way is offset by the excessive height requirements for this line to remain in its present alignment.

There will be no sailboating north of LA Highway 10 in this project alternative. In this area the transmission line must accommodate the 50 year flood elevation to meet National Electric Code Standards. It appears that this portion of the line has adequate clearance and that relocation or raising will not be needed.

The estimated total length of new transmission line is approximately 5 miles. The estimated cost of removing the affected transmission line and installing the replacement section is $7,421,000. This cost includes new rights-of-way.

- Dixie Electric Membership Company (DEMCO)

DEMCO owns a 69 kv local transmission powerline that runs across the proposed reservoir from Hatchersville

northeast to Darlington. Like the above GSU line, it appears to be far more feasible to relocate this line outside the reservoir limits south of LA Hwy 10 than raising it to accommodate sailboating. The replacement line can share the new GSU towers up to LA Hwy 10. New poles, which can also accommodate other local lines, will be required along the north side of LA Hwy 10 to bring this line east to Darlington.

The estimated total length of replacement is about 10.5 miles. The estimated cost of removal and relocation is $1,609,000.

o Relocation of Power and Communication Lines Along LA Hwy 10

About 2.5 miles of local distribution aerial power and phone lines must be raised along LA Hwy 10. These lines can be attached to the new poles required for the above DEMCO transmission powerline at an estimated cost of $101,000.

o Relocation of Power and Communication Lines Along LA Hwy 432

Adequate clearance currently exists and no raising is required for this alternative.

Dry Reservoir Alternative.

(Roads and Bridges)

We estimate that the total cost for complete in-kind relocations of all affected roads in this alternative is $25,676,000. Possible reductions in road service, such as detours and class downgrading, can reduce this cost down to $1,254,000. Significant trade-offs associated with these options are discussed below. Roadway relocations for this plan are shown on Plate 35.

- LA Highway 960

In all proposed reservoir alternatives the west end of the dam will intersect this highway. A one mile relocation, that includes new rights-of-way, around the dam is required. The estimated cost is $1,254,000.

- LA Highway 448

In all proposed reservoir alternatives the dam and flood pool affects this road which connects LA Hwy 37 and LA Hwy 10. The most feasible relocation of LA 448 is to permanently re-route traffic to an existing parish road located about three miles east of its current location. About 8.5 miles of this road can be upgraded from class RC-1 to RC-2 to match the existing classification of LA 448, or, this road can be utilized as is. The estimated cost of this optional upgrading, that includes new rights-of-way, is $14,700,000.

- LA Highway 10; Darling Creek Reach

This road elevation in this reach is above the design 50 year flood elevation. No raising will be required.

- LA Highway 10; Amite River Basin Reach

There are several relocation options available for this reach which is about 2.2 miles long. First, the existing highway can be raised, about 13 feet above existing ground on average, by means of embankment in conjunction with replacing and raising five existing bridges. This option will provide an in-kind replacement of the highway and has an estimated cost of $9,722,000.

For the height required for this raising, replacement with a causeway bridge completely across the basin is not feasible.

As with the other alternatives, it is also possible to permanently re-route traffic by detouring on LA 38 from Coleman Town to LA 432, to LA 961 back to LA 10. About twelve miles total of LA 432 and 961 must be upgraded from RC-2 to RC-3 to match the existing classification of LA 10. The estimated cost of this plan, that includes new rights-of-way, is $19,093,000. Aside from being more expensive than the above plan, there are significant reductions in road service with this relocation option. This detour route is about eight miles longer than this existing reach of LA 10 and will require an additional traffic stop at the intersection of LA 38 and LA 432. The significant increase in traffic volume at this intersection will also be a problem to consider.

The above traffic re-routing can also be accomplished without upgrading LA 432 and 961. In addition to the above shortcomings of this detour, this reach will also be downgraded in classification. This minimum relocation can be done at virtually no cost.

- East Feliciana Parish Road 42; Collins Creek Crossing

This reservoir alternative does not affect this bridge.

(Utilities)

The estimated total cost for the relocation of all affected utilities for this alternative is $7,955,000. Utility relocations for this plan are shown on Plate 36.

- Colonial Pipeline Co.

This owner's two petroleum project pipelines are not normally buoyant and will be submerged only by infrequent flood events in this alternative. Weight installation will not therefore be required.

- Transcontinental Gas Pipeline Co.

This owner has one, 30-inch and two, 36-inch natural gas pipelines that cross the proposed reservoir. Since these lines are continuously buoyant, they will require weighting up to a reasonable flood event elevation. Discounting existing weighted lengths, about 9,000 feet of each line must be weighted to accommodate a 100 year flood event. The estimated cost is $6,129,000.

- Removal of Local Distribution Power and Communication Lines

In this alternative about 22 miles of local distribution aerial power and phone lines, including poles, will require removal at a cost of $101,000.

- Relocation of Power and Communication Lines Along LA Hwy 960

In all project alternatives, about one mile of aerial power and communication lines require relocation in association with this highway relocation. Relocated lines will be placed in new highway rights-of-way. The estimated cost is $137,000.

- Gulf States Utilities (GSU)

This line appears to have adequate clearance above the design 50 year flood elevation. No raising or relocation will be required for this alternative.

- Dixie Electric Membership Company (DEMCO)

DEMCO owns a 69 kv local transmission powerline that runs across the proposed reservoir from Hatchersville northeast to Darlington. It appears to be more feasible to relocate this line outside the reservoir in the area south of LA 10 as opposed to raising it along its present

alignment to accommodate the design 50 year flood level. This section can possibly be connected to the existing GSU towers along the western boundary of the reservoir. New poles, which can also accommodate other local lines, will be required along the north side of LA Hwy 10 to connect this line east to Darlington.

The estimated total length of replacement is about 9.5 miles. The estimated cost of removal and relocation is $1,487,000.

- Relocation of Power and Communication Lines Along LA Hwy 10

About 2.5 miles of local distribution aerial power and phone lines must be raised along LA Hwy 10. These lines can be attached to the new poles required for the above DEMCO transmission powerline at an estimated cost of $101,000.

- Relocation of Power and Communication Lines Along LA Hwy 432

Adequate clearance currently exists and no raising will be required for this alternative.

TABLE 40

ROADWAY AND UTILITY ESTIMATED RELOCATION COSTS
WET RESERVOIR ALTERNATIVE

A. ROADS AND BRIDGES.

1. LA 960 @ West End of Dam; Relocate 1.0 mile around dam. Cost includes 14.5 acres new right-of-way.

Estimated Cost $1,254,000

2. LA Hwy 448; Permanently re-route traffic to existing parish road 3 miles to the east; option to upgrade 8.5 miles of parish road from class RL-1 to RC-2. Cost includes 41 acres new right-of-way.

(Optional) Estimated Cost $14,700,000

3. LA Hwy 10 Darling Creek Reach; Raise 4,700 feet (by embankment) and construct 350 ft replacement bridge.

Estimated Cost $4,148,000

4. LA Hwy 10 Amite River Basin Reach; Raise 12,500 ft (by embankment) and construct (5) replacement bridges.

(Optional) Estimated Cost $21,638,000

5. E. Feliciana Parish Road 42 Collins Creek Bridge; Construct new 60 ft. long bridge and raise 1,100 feet of approach roadway.

Estimated Cost $593,000

(Other optional items not included in subtotals)

4.-a. LA 10 Amite River Basin Reach; Construct 14,000 ft causeway bridge completely across the basin.

Estimated Cost $46,970,000

4.-b. LA 10 Amite River Basin Reach; Relocate to existing hwys LA 432 and 961; upgrade 15 miles from RC-2 to RC-3. Cost includes 45 ac. new right-of-way.

Estimated Cost $19,093,000

TABLE 40 (cont'd)

Subtotal Roadways	$42,333,000
Subtotal Roadways (with all optional service reductions)	$4,741,000

B. UTILITIES.

1. Colonial pipeline Co.; Install set-on concrete river weights on two, 36-inch petroleum product pipelines. Each require weights for a distance of 6,000 feet across the Amite River Basin.

Estimated Cost $3,050,000

2. Transcontinental Gas Pipeline Co.; Install set-on concrete river weights on two, 36-in. and one, 30-in. gas pipelines. Each require weights for a distance of 9,500 feet across the Amite River Basin.

Estimated Cost $6,469,000

3. Remove 23 miles of local distribution aerial power and phone lines including poles.

Estimated Cost $107,000

4. Along new LA 960 reach; relocate 1 mile of local distribution aerial power and phone lines.

Estimated Cost $137,000

5. Gulf States Utilities (GSU); Relocate 10 miles of 500/230 kv transmission powerline; Included:
   - clearing cost for all 10 miles with 200 ft wide r-o-w
   - new towers spaced @ 1,000 ft with min. line vertical clearance of 36 ft

TABLE 40 (cont'd)

- cost to remove lines and dismantle towers on 10 miles of existing line
- lines share same tower; new 230 kv line to be installed on 6 miles only.
- 133 acres new right-of-way

Estimated Cost $15,719,000

6. Dixie Electric Membership Co. (DEMCO); Relocate 11 miles of 69 kv transmission powerline; Included:
   - 5.5 miles to share the above new (GSU) towers
   - 2 miles to be on new single poles with min. line vertical clearance of 46 ft across Amite R. Basin
   - 1 mile to be on new single poles with min. line vertical clearance of 36 ft across Darling Cr. Basin
   - 2.5 miles to be on new single poles with min. line vertical clearance of 24 ft along LA 10 outside reservoir
   - cost to remove 8.5 miles of existing lines and poles.

   Estimated Cost $1,728,000

7. Along LA 10 reach; Raise 3.5 miles of local distribution aerial power and phone lines; lines to share the above new DEMCO poles.

   Estimated Cost $142,000

8. Along LA 432 reach; Raise 1 mile of local distribution aerial power and phone lines; use existing poles and extend 5 ft vertical.

   Estimated Cost $49,000

Subtotal Utilities	$27,401,000
TOTAL WET ALTERNATIVE RELOCATIONS	**$69,734,000**
TOTAL WET ALTERNATIVE (with all optional reductions in road service)	**$32,142,000**

Note: Costs include 12% for the owner's engineering, 10% for the owner's construction administration, plus, 25% for contingencies.

TABLE 41

ROADWAY AND UTILITY ESTIMATED RELOCATION COSTS
REDUCED WET ALTERNATIVE

A. ROADS AND BRIDGES.

1. LA 960 @ West End of Dam; Relocate 1.0 mile around dam Cost includes 14.5 acres new right-of-way.

Estimated Cost $1,254,000

2. LA 448; Permanently re-route traffic to existing parish road 3 miles to the east; option to upgrade 8.5 miles of parish road from class RL-1 to RC-2. Cost includes 41 acres new right-of-way.

(Optional) Estimated Cost $14,700,000

3. LA 10 Amite River Basin Reach; Raise 11,500 ft (by embankment) and construct (5) replacement bridges.

(Optional) Estimated Cost $11,347,000

(Other optional item not included in subtotal)

3.-a. LA 10 Amite River Basin Reach; Relocate to existing hwys LA 432 and 961; upgrade 15 miles from RC-2 to RC-3. Cost includes 45 acres new right-of-way.

Estimated Cost $19,093,000

Subtotal Roadways $28,301,000

Subtotal Roadways (with all optional service reductions) $1,254,000

B. UTILITIES.

1. Transcontinental Gas Pipeline Co.; Install set-on concrete river weights on two, 36-in. and one, 30-in. gas pipelines. Each require weights for a distance of 9,000 feet across the Amite River Basin.

Estimated Cost $6,129,000

TABLE 41 (cont'd)

2. Remove 22 miles of local distribution aerial power and phone lines including poles.

Estimated Cost $101,000

3. Along new LA 960 reach; relocate 1 mile of local distribution aerial power and phone lines.

Estimated Cost $137,000

4. Gulf States Utilities (GSU); Relocate 5 miles of 500/230 kv transmission powerline; Included:
   - clearing cost for all 5 miles with 200 ft wide r-o-w
   - new towers spaced @ 1,000 ft with min. line vertical clearance of 36 ft
   - cost to remove lines and dismantle towers on 4 miles of existing line.
   - 121 acres new right-of-way

Estimated Cost $7,421,000

5. Dixie Electric Membership Co. (DEMCO); Relocate 10.5 miles of 69 kv transmission powerline; Included:
   - 5 miles to share the above new (GSU) towers - 2 miles to be on new single poles with min. line vertical clearance of 36 ft across Amite R. Basin
   - 3.5 miles to be on new single poles with min. line vertical clearance of 24 ft across Darling Cr. Basin and outside of reservoir
   - cost of to remove 7 miles of existing line and poles.

Estimated Cost $1,609,000

6. Along LA 10 reach; raise 2.5 miles of local distribution aerial power and phone lines; lines to share the above new DEMCO poles.

Estimated Cost $101,000

Subtotal Utilities $15,498,000

TABLE 41 (cont'd)

TOTAL REDUCED WET ALTERNATIVE RELOCATIONS	$42,799,000
TOTAL REDUCED WET ALTERNATIVE RELOCATIONS (with all optional reductions in road service)	$16,752,000

Note: Costs include 12% for the owner's engineering, 10% for the owner's construction administration, plus, 25% for contingencies.

TABLE 42
ROADWAY AND UTILITY ESTIMATED RELOCATION COSTS
DRY ALTERNATIVE

a. ROADS AND BRIDGES.

1. LA 960 @ West End of Dam; Relocate 1.0 mile around dam. Cost includes 14.5 acres new right-of-way.

Estimated Cost $1,254,000

2. LA 448; Permanently re-route traffic to existing parish road 3 miles to the east; option to upgrade 8.5 miles of parish road from class RL-1 to RC-2. Cost includes 41 acres new right-of-way.

(Optional) Estimated Cost $14,700,000

3. LA 10 Amite River Basin Reach; Raise 11,500 ft (by embankment) and construct (5) replacement bridges.

(Optional) Estimated Cost $9,722,000

(Other optional item not included in subtotal)

3.-a. LA 10 Amite River Basin Reach; Relocate to existing hwys LA 432 and 961; upgrade 15 miles from RC-2 to RC-3. Cost includes 45 acres new right-of-way.

Estimated Cost $19,093,000

Subtotal Roadways	$26,676,000
Subtotal Roadways (with all optional service reductions)	$1,254,000

B. UTILITIES.

1. Transcontinental Gas Pipeline Co.; Install set-on concrete river weights on two, 36-in. and one, 30-in. gas pipelines. Each requires weights for a distance of 9,000 feet across the Amite River Basin.

Estimated Cost $6,129,000

TABLE 42 (cont'd)

2. Remove 22 miles of local distribution aerial power and phone lines including poles.

Estimated Cost $101,000

3. Along LA 960 reach; relocate 1 mile of local distribution aerial power and phone lines along new LA 960 reach.

Estimated Cost $137,000

4. Relocate 9.5 miles 69 kv Dixie Electric Membership Co. (DEMCO) transmission powerline; Included:
   - 4 miles to be installed on existing GSU towers
   - 2 miles to be on new single poles with min. line vertical clearance of 34 ft across Amite R. Basin
   - 3.5 miles to be on new single poles with min. vertical clearance of 24 ft across Darling Cr. and outside of reservoir
   - cost to remove 7 miles of existing line and poles.

Estimated Cost $1,487,000

5. Along LA 10 reach; raise 2.5 miles of local distribution aerial power and phone lines; lines to share the above new DEMCO poles.

Estimated Cost $101,000

Subtotal Utilities	$7,955,000
TOTAL DRY ALTERNATIVE RELOCATIONS	**$33,631,000**
TOTAL DRY ALTERNATIVE RELOCATIONS (with all optional reductions in road service)	**$9,209,000**

*** Costs include 12% for the owner's engineering, 10% for the owner's construction administration, plus, 25% for contingencies.**

ENVIRONMENTAL IMPACT AND MITIGATION

CATEGORIZATION OF RESOURCES

Woodland habitat. Woodland habitat is considered a significant resource in the study area. Bottomland hardwoods, a particular kind of wooded lands exhibit a significance, not only regionally, but nationally, as well. The Water Resources Development Act (WRDA) of 1986 states that "impacts to bottomland hardwoods shall be mitigated in-kind to the extent possible" when impacted by Federal actions. Therefore, an attempt was made to quantify all woodlands, and particularly bottomland hardwoods, impacted by each alternative.

Free-flowing river. Free-flowing river habitat is an aquatic habitat that is considered to be significant to the study area and the nation. Many rivers such as the Amite get their productivity not from within, but from sources away from the river itself. The rise and fall of these rivers in response to the influx of storm waters results in a constant recharge of organic material including decomposing leaves, grasses, and twigs from forested areas not within the river channel. Many of our nation's rivers have been modified by channelization and by dams and reservoirs for flood control and navigation which has resulted in significant adverse impacts to this particular habitat.

EVALUATION PROCEDURES

Methodologies. Evaluation procedures were utilized to quantify the losses of the two significant resources impacted by the Darlington reservoir project. There are two evaluation systems completely acceptable for evaluating the effects of actions on terrestrial resources, i.e., wildlife habitat. The Habitat Evaluation Procedures (HEP) developed by the U.S. Fish and Wildlife Service (USFWS) is

a system that is accepted for use nationally. The Habitat Evaluation System (HES) developed by the U.S. Army Corps of Engineers (Corps) is accepted for use in the Lower Mississippi Valley. There is no commonly utilized standardized system for evaluating impacts of flood control projects on aquatic habitat. An evaluation system was developed jointly by the biologists of the USFWS and the Corps.

Woodland Analysis.

Both the HEP and the HES utilize a system of habitat evaluation that is based on all habitat having some value that can be quantified somewhere between 0.0 and 1.0. That is, the maximum value any habitat, i.e., perfect habitat, could have would be 1.0. The HEP utilizes individual species for determining value while the HES measures habitat from an overall value approach. The method used in this analysis consisted of a combination of the two methods. Both methods utilize measurement of a group of parameters that are designed to reflect the value of the habitat measured for either the certain species in the case of HEP or the habitat in general in case of HES. Parameters measured include for instance, canopy closure, canopy closure of mast-bearing trees, percent coverage of understory growth, percent coverage of ground cover, number of s: s, number of trees of a certain size, tree and shrub spec makeup, etc..

Habitat Sampling. This evaluation utilized HEP habitat sampling done early in the permit application process by the team of Espey Houston, USFWS, and the Louisiana Department of Wildlife and Fisheries (LDWF). The values were summed and averaged to establish a value that could be used in a screening type analysis. The values were then inserted into a formula that included habitat value multiplied by acreage impacted over a given time period, i.e., the project economic life. The resulting number is then converted to an average number and is expressed as

average annual habitat unit (AAHU) within the HEP and annualized habitat unit value (HUV) in the HES. For the purposes of this analysis, the AAHU term was used.

Source of losses. Terrestrial losses were made up of losses of woodland habitat caused by three categories of project actions. These categories were 1) direct construction, 2) relocations, and 3) permanent pool inundation. That woodland habitat covered by periodic inundation was not considered a lost resource.

Calculated Losses. The losses were determined for three reservoir types. They were the Full Wet Reservoir, the Reduced Size Wet Reservoir, and the Dry Reservoir. Losses of all woodland habitat for the three reservoir types were 8,228; 3,102; and 500 acres, respectively. It was considered appropriate to include all woodland in order not to produce an overly conservative mitigation cost estimate. Bottomland hardwoods, which would be used in a more detailed analysis, make up approximately 88 percent of the total woodland losses. A projected project economic life of 50 years was utilized for the period of analysis. Utilizing the methodology described previously, habitat losses of 7,148; 2,739; and 450 AAHU's were determined to be attributable to construction of the three respective reservoirs. Tables 43-45 display the procedures by which the habitat losses were calculated for the 50 year Wet, Reduced Wet, and Dry Reservoirs. The same procedure was utilized for both other alternatives. It was assumed in the assessment of losses that the current Habitat Suitability Index (HSI) value, i.e., 0.65, would be maintained in the future without the project (no action) for the duration of the analysis period. This assumption was based upon the thought that the same factors that resulted in the current HSI would continue to be exhibited; therefore no changes would occur. No reductions in acreage of the woodland habitat in the no action condition was assumed. The project area is not in an area of urban development that would provide reason to project a

reduction of the amount of wooded lands. Also, USDA programs in recent years have been providing incentives for landowners to take land out of commodity crop production which may eventually result in increased amounts of forested areas.

Mitigation Needs. Mitigation of project impacts is required by the National Environmental Policy Act (NEPA), the Fish and Wildlife Coordination Act (FWCA), WRDA of 1986, and evolving environmental awareness of the country. Mitigation is a process, the goal of which is to minimize significant environmental effects of actions. It's steps consist of avoiding, minimizing, and, at last, compensating all unavoidable losses. Avoidance could be thought of as avoiding impacting significant resources or not implementing the project at all. Minimization would consist of choosing an alternative with less damages to significant resources. Mitigation by minimization for the Darlington study would consist of choosing the dry reservoir over the other alternatives. After all steps are taken to avoid and minimize adverse impacts, a process is used to determine appropriate compensation for those losses. The same system utilized in evaluating effects of actions upon wildlife habitat can be utilized in determining appropriate compensation. This was done for the Darlington study.

Mitigation Objectives. The objectives to mitigate environmental losses are:

- compensate by replacement, 100% of the unavoidable woodland habitat losses due to implementation of the flood control actions, and

- compensate by replacement or habitat productivity increases, 100% of the unavoidable losses of free-flowing river habitat due to implementation of the flood control actions.

Woodland Mitigation Alternatives. The goal of the compensation analysis in this study was to determine the amount of area upon which habitat value equal to that lost due to the action could be produced. Two scenarios were developed for the Darlington study in order to explain the basis for recommending a certain mitigation plan. Scenario (a) utilized existing open lands and scenario (b) utilized existing wooded lands. An analysis was planned to determine the overall habitat value of a tract of each habitat type utilizing certain target years for the period of analysis. This was done for the area with the mitigation plan being implemented and for the same area with no action to mitigate the habitat losses, i.e., with no change in the current conditions. The output of the analysis was to produce the most cost-effective mitigation plan.

Woodland Mitigation Assumptions.

(Open Land Scenario) The scenario that utilized existing open land was based upon the premise that the open tract analyzed would remain in an open condition under the no action plan (no mitigation implemented) for the entirety of the analysis period. Thus the HSI would have been 0.0 for the entire period; and the product of 0.0 times any acreage for the analysis period would have been 0.0. Therefore, there would be no habitat value of the acreage of the no action plan to subtract from the habitat value produced by the selected acreage of the action plan. With the mitigation plan in place, the area would be planted with seedlings of trees that would produce habitat value fairly rapidly and would be managed in such a way that a much higher HSE would be reached over the analysis period than that of woodlands for which actual habitat sampling was completed. For example, approximately 4,019 acres of open land would be required for planting and protection to produce the AAHUs necessary (2,739) to mitigate the habitat losses incurred by the implementation of the Reduced Wet Reservoir. Tables 46-48 illustrate these calculations for

the 50 Year Wet, Reduced Wet, and Dry Reservoirs.

(Wooded Land Scenario) If the scenario utilizing a tract of wooded land was implemented, it can be seen that a considerable larger tract would be required. The idea of this feature would be to create an amount of habitat value on the mitigation tract above and beyond the no action value in the amount equal to those losses caused by implementation of the flood control plan, i.e., the AAHUs of losses for the analysis period. Under the no action plan (future without condition), it was assumed that the HSE values would remain the same as they were when sampled and the acreage would not change. Under the mitigation action plan, habitat improvement measures (development) would be implemented to result in eventual higher quality habitat. This would include items such as creation of clearints, selective deadening of intermediate sized non-mast-producing trees to release suppressed mast producers, and limited plantings. This would cause a reduction of HSI values initially, but would later result in much higher HSI values on the tract than existed prior to the development features. However, as Table 47 displays for the Reduced Wet Reservoir, approximately 16,130 acres of existing wooded land would be required to replace the 2,739 AAHUs lost. Stewardship or protection of the area would be required to assure that the habitat quality projected, actually does occur. Tables 46-48 also illustrate these calculations for the 50 year Wet, Reduced Wet, and Dry Reservoirs.

Woodland Cost Development. Estimates were made for costs associated with the implementation of each mitigation measure. The costs included real estate, habitat development, and required annual operation and maintenance. Table 49 displays estimated terrestrial mitigation costs for the three alternative reservoir plans. The costs displayed are based upon the original AAHUs lost and compensation acreages required and not upon the corrected and revised AAHUs lost and compensation acreages required.

While there are a number of available mitigation sites, a site located in the West Atchafalaya Floodway in St. Landry Parish was chosen for the purposes of this analysis. This area is shown on Plate 37.

Free-flowing River Mitigation.

(River Mitigation Concerns)

Mitigation for impacted aquatic resources has not been developed to the extent as for terrestrial resources. In many cases, planners are at a loss as to what can be done to compensate the losses to the impacted resources. The frequent rise and fall of the river in its natural condition with its out-of-banks flooding, in most cases, cannot be reproduced. The input from leaf litter, twigs, and small branches is the source of productivity to the Amite River. Replacement of that lost productivity for the extent of free-flowing river impacted by the construction of any dam alternative is the goal of mitigation for the Darlington study.

(River Losses)

The loss that would be mitigated is the 38 miles of free-flowing river both upstream (from approximately 10 miles north of the structure) and downstream of the proposed dam structure (to about 10 miles south of Denham Springs) that would be effected by either alternative. The full wet reservoir would cause the greatest amount of reduction in extent of alteration to the free-flowing nature of the river as any alternative evaluated. The reduced wet reservoir would be less detrimental and was estimated to be approximately 70 percent as detrimental as the full wet. The dry reservoir would be even less detrimental and was estimated to be approximately 50 percent as detrimental as the full wet. Out-of-bank flooding would not be reduced but would in fact be increased in the area upstream of the dam by the dry

reservoir alternative. Mitigation would, therefore, consist of the definite protection through acquisition of that 3-foot zone on each side of the river for the entirety of the 38 river miles for the full wet alternative.

(River Mitigation Assumptions)

Productivity of a free-flowing sandy-bottomed stream is directly related to the productivity of surrounding riparian forests. Approximately 97 percent of the materials (leaf litter, twigs, etc.) contributing to biological productivity supplied to warm water flowing streams by frequent flooding of out-of-bank areas is derived from the first 300 feet from the stream bank (see Amite River and Tributaries, Louisiana, Comite River Basin, Feasibility Study (ARTCRBFS) and Environmental Impact Statement (EIS), Appendix B-1). Therefore, the area on either side and within 300 feet of the stream is the significant area and, for the purposes of the study, should be protected.

(River Mitigation Costs) Related costs for this measure include the assumption that the 300-foot strip would be bought and that a large portion of the area (56%) would be in a sand and gravel mining area and a smaller portion (44%) would be in wooded land. Sand and gravel mining is extensive along the Amite River in this area. The amount of $2,300 per acre was used for this considering the amount of each land use in the total length, and 1.32 was used as a multiple to convert actual land costs to obtain a very general total cost estimate which includes acquisition costs. However, it is stressed that this is definitely not an approved Corps estimate since it was not requested of NOD Real Estate Division and has definite limitations. The acreage required was based upon the 300-foot strip on each side or 600 feet in total width, times 5,280 feet per mile, multiplied by the 38-mile strip. Land costs for this feature would total approximately $8,371,182.

Mitigation for the reduced wet reservoir alternative would consist of protection through acquisition for all of the 38-mile area but for approximately 70 percent of the total losses. Therefore, these land costs were calculated to be 70 percent of the amount for the full wet reservoir or approximately $5,859,828. Similarly, since the dry reservoir was said to be 50 percent as detrimental as the full wet reservoir for this purpose, the land costs for this mitigation feature were calculated to be approximately $4,185,591.

Another cost associated with the aquatic mitigation feature is for fencing. The increase in productivity of the 300-foot area on each side of the river can be assured only if there is a guarantee of no development or grazing on the strip. A fence would be required to restrict free-ranging cattle and unauthorized individuals that may enter to cut firewood wherever restriction is not evident. Fencing is included as a construction cost and is based upon a cost of $3.50 per foot for fence construction through wooded lands. The fencing cost is shown in Table 49.

Combined Terrestrial and Aquatic Mitigation Costs.

Table 49 presents a display of the costs of all components of both features of the overall combined mitigation plan. As is described previously, the scenarios of using cleared lands or existing wooded land for terrestrial mitigation are presented. It should be understood that one or the other scenario is to be used, but not combined. A combination could be done with the goal simply to fully compensate losses by replacing the number of AAHUs lost due to the implementation of the alternative being evaluated. It is obvious that the utilization of cleared land rather than existing wooded land to compensate unavoidable losses would be the mitigation plan that would be recommended for selection. For both the Wet and Reduced Wet Alternatives, cleared area

real estate located in the West Atchafalaya Floodway in St. Landry Parish was considered for mitigation purchase. A cost breakdown for this real estate is listed in the following section. Acreages used in this breakdown differ slightly from those shown in Tables 46-48. In the Dry Alternative cleared areas quantities are available within the reservoir limits to be purchased to accommodate the flood control pool. This cost is included in the Real Estate estimate for this alternative and therefore not charged a second time. No specific sites were chosen and analyzed for wooded mitigation. Costs were estimated as being comparable to cleared land. As with the cleared area option, wooded land for mitigation is available within the proposed reservoir limits for the dry alternative and again costs were not double charged.

Replacements and Operation and Maintenance. Fencing would require replacement at intervals of approximately every 17 years. Stewardship would be required to see that the area is protected from vandalism and unauthorized activities. A projected cost of $30,000 is estimated for a salary of one individual to provide the stewardship that the area would require.

TABLE 43

HABITAT IMPACT ANALYSIS SCREENING COMPUTATION
WET RESERVOIR

WOODLAND HABITAT LOSSES

WITHOUT PROJECT					
TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	8628	0.65	5608		
1	8628	0.65	5608	5608	
2	8628	0.65	5608	5608	
3	8628	0.65	5608	5608	
4	8628	0.65	5608	5608	
5	8628	0.65	5608	5608	
6	8628	0.65	5608	5608	
13	8628	0.65	5608	39257	
23	8628	0.65	5608	56082	
33	8628	0.65	5608	56082	
43	8628	0.65	5608	56082	
63	8628	0.65	5608	112164	
83	8628	0.65	5608	112164	
103	8628	0.65	5608	336492	
				801973 (SUM)	7786 (CUM/TY)

WITH PROJECT					
0	8628	0.65	5608		
1	8628	0.65	5608	5608	
2	8628	0.65	5608	5608	
3	8193	0.65	5325	5467	
4	8193	0.65	5325	5325	
5	8193	0.65	5325	5325	
6	400	0.65	260	2793	
13	400	0.65	260	1820	
23	400	0.65	260	2600	
33	400	0.65	260	2600	
43	400	0.65	260	2600	
63	400	0.65	260	5200	
83	400	0.65	260	5200	
103	400	0.65	260	15600	
				65747 (SUM)	638 (CUM/TY)

Project impact in habitat units= 7148

TABLE 44

HABITAT IMPACT ANALYSIS SCREENING COMPUTATION
REDUCED WET RESERVOIR
WOODLAND HABITAT LOSSES

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
WITHOUT PROJECT					
0	3502	0.65	2276		
1	3502	0.65	2276	2276	
2	3502	0.65	2276	2276	
3	3502	0.65	2276	2276	
4	3502	0.65	2276	2276	
5	3502	0.65	2276	2768	
6	3502	0.65	2276	2276	
13	3502	0.65	2276	15934	
23	3502	0.65	2276	22763	
33	3502	0.65	2276	22763	
43	3502	0.65	2276	22763	
63	3502	0.65	2276	45526	
83	3502	0.65	2276	45526	
103	3502	0.65	2276	136578	
				325511 (SUM)	3160 (CUM/TY)
WITH PROJECT					
0	3502	0.65	2276		
1	3079	0.65	2001	2139	
2	3079	0.65	2001	2001	
3	3079	0.65	2001	2001	
4	400	0.65	260	1131	
5	400	0.65	260	260	
6	400	0.65	260	260	
13	400	0.65	260	1820	
23	400	0.65	260	2600	
33	400	0.65	260	2600	
43	400	0.65	260	2600	
63	400	0.65	260	5200	
83	400	0.65	260	5200	
103	400	0.65	260	15600	
				43412 (SUM)	421 (CUM/TY)

Project impact in habitat units= 2739

TABLE 45

HABITAT IMPACT ANALYSIS SCREENING COMPUTATION
DRY RESERVOIR

WOODLAND HABITAT LOSSES

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
FUTURE WITHOUT PROJECT					
0	802	0.65	521		
1	802	0.65	521	521	
2	802	0.65	521	521	
3	802	0.65	521	521	
4	802	0.65	521	521	
5	802	0.65	521	521	
6	802	0.65	521	521	
13	802	0.65	521	3649	
23	802	0.65	521	5213	
33	802	0.65	521	5213	
43	802	0.65	521	5213	
63	802	0.65	521	10426	
83	802	0.65	521	10426	
103	802	0.65	521	31278	
				74546 (SUM)	724 (CUM/TY)
FUTURE WITH PROJECT					
0	802	0.65	521		
1	400	0.65	260	391	
2	400	0.65	260	260	
3	400	0.65	260	260	
4	300	0.65	195	228	
5	300	0.65	195	195	
6	300	0.65	195	195	
13	300	0.65	195	1365	
23	300	0.65	195	1950	
33	300	0.65	195	1950	
43	300	0.65	195	1950	
63	300	0.65	195	3900	
83	300	0.65	195	3900	
103	300	0.65	195	11700	
				28243 (SUM)	274 (CUM/TY)

Project impact in habitat units= 450

TABLE 46

GENERAL HABITAT (WOODLAND) MITIGATION AREA
WET RESERVOIR
(Assume reforesting cleared land)

FUTURE
WITH MITIGATION

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	10488	0.00	0		
1	10488	0.00	0	0	
2	10488	0.15	1573	787	
3	10488	0.20	2098	1835	
4	10488	0.20	2098	2098	
5	10488	0.25	2622	2360	
6	10488	0.38	3985	3304	
13	10488	0.45	4720	30468	
23	10488	0.48	5034	48769	
33	10488	0.55	5768	54013	
43	10488	0.65	6817	62928	
63	10488	0.85	8915	157320	
83	10488	0.90	9439	183540	
103	10488	0.90	9439	188784	
			TOTAL	736205	7148

GENERAL HABITAT (WOODLAND) MITIGATION AREA
(Managing existing wooded lands, i.e., HSI's increase.)

FUTURE
WITHOUT MITIGATION

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	42094	0.65	27361		
1	42094	0.65	27361	27361	
2	42094	0.65	27361	27361	
3	42094	0.65	27361	27361	
4	42094	0.65	27361	27361	
5	42094	0.65	27361	27361	
6	42094	0.65	27361	27361	
13	42094	0.65	27361	191528	
23	42094	0.65	27361	273611	
33	42094	0.65	27361	273611	
43	42094	0.65	27361	273611	
63	42094	0.65	27361	547222	
83	42094	0.65	27361	547222	
103	42094	0.65	27361	547222	
AAHU'S mitigated = 7148			TOTAL	2818193	27361

TABLE 46 (Continued)

GENERAL HABITAT (WOODLAND) MITIGATION AREA
WET RESERVOIR

FUTURE
WITH MITIGATION

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	42094	0.65	27361		
1	42094	0.65	27361	27361	
2	42094	0.65	27361	27361	
3	42094	0.55	23152	25256	
4	42094	0.58	24415	23783	
5	42094	0.61	25677	25046	
6	42094	0.65	27361	26519	
13	42094	0.75	31571	206261	
23	42094	0.78	32833	322019	
33	42094	0.80	33675	332543	
43	42094	0.82	34517	340961	
63	42094	0.85	35780	702970	
83	42094	0.90	37885	736645	
103	42094	0.90	37885	757692	
			TOTAL	3554417	34509
			Impact in habitat units=		7148

TABLE 47

GENERAL HABITAT (WOODLAND) MITIGATION AREA
50-YEAR REDUCED WET RESERVOIR
(Assume reforesting cleared land)

FUTURE
WITH MITIGATION

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	4019	0.00	0		
1	4019	0.00	0	0	
2	4019	0.15	603	301	
3	4019	0.20	804	703	
4	4019	0.20	804	804	
5	4019	0.25	1005	904	
6	4019	0.38	1527	1266	
13	4019	0.45	1809	11675	
23	4019	0.48	1929	18688	
33	4019	0.55	2210	20698	
43	4019	0.65	2612	24114	
63	4019	0.85	3416	60285	
83	4019	0.90	3617	70333	
103	4019	0.90	3617	72342	
			TOTAL	282114	2739

GENERAL HABITAT (WOODLAND) MITIGATION AREA
(Managing existing wooded lands, i.e., HSI's increase.)

FUTURE
WITHOUT MITIGATION

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	16130	0.65	10485		
1	16130	0.65	10485	10485	
2	16130	0.65	10485	10485	
3	16130	0.65	10485	10485	
4	16130	0.65	10485	10485	
5	16130	0.65	10485	10485	
6	16130	0.65	10485	10485	
13	16130	0.65	10485	73392	
23	16130	0.65	10485	104845	
33	16130	0.65	10485	104845	
43	16130	0.65	10485	104845	
63	16130	0.65	10485	209690	
83	16130	0.65	10485	209690	
103	16130	0.65	10485	209690	
AAHU'S mitigated = 2739			TOTAL	1079904	10485

TABLE 47 (Continued)

GENERAL HABITAT (WOODLAND) MITIGATION AREA
50-YEAR REDUCED WET RESERVOIR
(Assume reforesting cleared land)

FUTURE
WITH MITIGATION

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	16130	0.65	10485		
1	16130	0.65	10485	10485	
2	16130	0.65	10485	10485	
3	16130	0.55	8872	9678	
4	16130	0.58	9355	9113	
5	16130	0.61	9839	9597	
6	16130	0.65	10485	10162	
13	16130	0.75	12098	79037	
23	16130	0.78	12581	123395	
33	16130	0.80	12904	127427	
43	16130	0.82	13227	130653	
63	16130	0.85	13711	269371	
83	16130	0.90	14517	282275	
103	16130	0.90	14517	290340	
			TOTAL	1362017	13223

Impact in habitat units= 2739

TABLE 48

GENERAL HABITAT (WOODLAND) MITIGATION AREA
50-YEAR DRY RESERVOIR
(Assume reforesting cleared land)

FUTURE
WITH MITIGATION

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	660	0.00	0		
1	660	0.00	0	0	
2	660	0.15	99	50	
3	660	0.20	132	116	
4	660	0.20	132	132	
5	660	0.25	165	149	
6	660	0.38	251	208	
13	660	0.45	297	1917	
23	660	0.48	317	3069	
33	660	0.55	363	3399	
43	660	0.65	429	3960	
63	660	0.85	561	9900	
83	660	0.90	594	11550	
103	660	0.90	594	11880	
			TOTAL	46329	450

GENERAL HABITAT (WOODLAND) MITIGATION AREA
(Managing existing wooded lands, i.e., HSI's increase.)

FUTURE
WITHOUT MITIGATION

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	2650	0.65	1723		
1	2650	0.65	1723	1723	
2	2650	0.65	1723	1723	
3	2650	0.65	1723	1723	
4	2650	0.65	1723	1723	
5	2650	0.65	1723	1723	
6	2650	0.65	1723	1723	
13	2650	0.65	1723	12058	
23	2650	0.65	1723	17225	
33	2650	0.65	1723	17225	
43	2650	0.65	1723	17225	
63	2650	0.65	1723	34450	
83	2650	0.65	1723	34450	
103	2650	0.65	1723	34450	
AAHU'S mitigated = 450			TOTAL	177418	1723

TABLE 48 (Continued)

GENERAL HABITAT (WOODLAND) MITIGATION AREA
50-YEAR DRY RESERVOIR
(Assume reforesting cleared land)

FUTURE
WITH MITIGATION

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	2650	0.65	1723		
1	2650	0.65	1723	1723	
2	2650	0.65	1723	1723	
3	2650	0.55	1458	1590	
4	2650	0.58	1537	1497	
5	2650	0.61	1617	1577	
6	2650	0.65	1723	1670	
13	2650	0.75	1988	12985	
23	2650	0.78	2067	20273	
33	2650	0.80	2120	20935	
43	2650	0.82	2173	21465	
63	2650	0.85	2253	44255	
83	2650	0.90	2385	46375	
103	2650	0.90	2385	47700	
			TOTAL	223766	2172
			Impact in habitat units=		450

TABLE 49

ENVIRONMENTAL MITIGATION COST ESTIMATES

	Full Wet Reservoir	Reduced Wet Reservoir	Dry Reservoir
REAL ESTATE			
I.TERRESTRIAL			
Woodland Losses	8,228	3,102	500
Acres of Cleared Lands for Mitigation	11,448	4,019	660
Costs	$10,232,000	$3,601,000	$0*
Acres of Wooded Lands for Mitigation	41,300	15,900	2,700
Costs (estimated)	$36,907,000	$14,314,000	$0*
II.AQUATIC			
38 Miles Upstream & Downstream	$8,371,000	$5,860,000	$4,186,000
III.TOTAL REAL ESTATE			
Cleared Option	$18,603,000	$9,461,000	$4,186,000
Wooded Option	$45,278,000	$20,174,000	$4,186,000

* Mitigation lands exist within the acquisition limits of the reservoir.

TABLE 49 (Continued)

ENVIRONMENTAL MITIGATION COST ESTIMATES

	Full Wet Reservoir	Reduced Wet Reservoir	Dry Reservoir
CONSTRUCTION			
I.TERRESTRIAL			
Fencing Costs			
Cleared Option	$5,372,000	$3,335,000	$0*
Wooded Option	$10,730,000	$6,000,000	$0*
**Reforestation Costs			
Cleared Option	$2,070,000	$798,000	$136,000
Wooded Option	N/A	N/A	N/A
II.AQUATIC			
Total Aquatic Fencing @ $3.50 River mi.=38	$1,404,000	$1,404,000	$1,404,000
III.TOTAL CONSTRUCTION			
Cleared Option	$8,846,000	$5,537,000	$1,540,000
Wooded Option	$12,134,000	$8,062,000	$1,540,000

* Mitigation lands exist within the acquisition limits of the reservoir.

** Reforestation cost estimate is $200 per acre

NOTE: All design and construction management costs are included.

TABLE 49 (Continued)

ENVIRONMENTAL MITIGATION COST ESTIMATES

	Full Wet Reservoir	Reduced Wet Reservoir	Dry Reservoir
TOTAL FIRST COSTS (REAL ESTATE AND CONSTRUCTION)			
Cleared Option	$27,449,000	$14,998,000	$5,726,000
Wooded Option	$57,412,000	$28,236,000	$5,726,000
ANNUAL OPERATION & MAINTENANCE COSTS			
Cleared Option	$269,000	$197,000	$79,000
Wooded Option	$457,000	$314,000	$79,000

REAL ESTATE

Summary

The estimated total real estate costs required for the proposed dam structure and flood storage area for each (50 year design) project alternative are:

Alternative	Total Acres Required for Dam and Reservoir	Cost
Wet Reservoir	20,150	$30,861,000
Dry Reservoir	13,750	$21,420,000

The above costs include land, improvements, 25 percent added contingencies, Federal and non-Federal administrative costs for tract acquisitions, and added funds for residential relocations (PL 91-646). Itemized cost breakdowns for each 50 year design alternative are shown in Tables 50 and 51.

Estimated real estate costs for the purchase of areas for environmental mitigation are shown in Tabels 52 and 53. Real estate costs for new rights-of-way required for roadway and utility relocations are listed in Tables 54 and 55. These costs are not included in the above listed totals, but are included as part of the "Environmental Impact and Mitigation" and "Roadway and Utility Relocations" estimates.

Criteria

All habitable structures require acquisition from all areas within the boundary elevation of the "probable maximum flood event." This elevation also defines the area required for either a flowage easement or an in fee property purchase. Since there is no significant cost

difference with either option, and, since the entire reservoir area would include public recreational development, it was determined that direct in fee property purchase would be appropriate.

Assumptions and Limitations

Acreages and land losses for each project alternative were obtained from the Draft Environmental Impact Statement (EIS) for the Darlington Dam and Reservoir prepared by Espey Huston and Associates (EHA) for the State, December 1989. See copy of Tables 3.9-2 and 3.9-8 of this EIS in Appendix B. The land class listed as "Other" in this EIS was assumed to represent public roads and waterbodies. The number of tracts used to estimate acquisition costs were also obtained from this draft EIS.

It was assumed that all agricultural land would be purchased after crop harvest. No value was therefore given to said crops in this estimate. An estimated contributing value for timber was included in the final per acre values.

Specific cost data on sand and gravel operations were not available during this analysis. Values for the sand and gravel pits include only raw land costs. Further analysis of a selected plan should include appropriate revisions to account for commercial use losses. No costs were included for a required cemetery relocation. Again, revisions should be made should a plan be selected.

Due to the nature of this cost estimate, inspection of the project area was limited to areas accessible by paved or dirt roads.

Wet Reservoir Alternative

Based on data compiled in the EHA Draft EIS, this alternative would affect 20,150 acres that would require in fee purchase of 629 tracts of land. Most of this area

(15,000 acres) is woodland with about 4,300 acres in pasture/agricultural lands. The remaining acres are shared among residential, sand and gravel pits, roads and waterbodies. Acquisition would be required for 101 identified improvements. The total estimated real estate cost for this alternative is $30,861,000. See Table 50 for unit cost breakdown. This area is shown by the maximum flood level illustrated on Plate 12.

Dry Reservoir Alternative

Again, based on data compiled in the EHA Draft EIS, this alternative would affect 13,750 acres that would require in fee purchase of 386 tracts of land. Most of this area (10,000 acres) is woodland with about 3,000 acres in pasture/agricultural lands. The remaining acres are shared among residential, sand and gravel pits, roads and waterbodies. Acquisition would be required for 77 identified improvements. The total estimated real estate cost for this alternative is $21,420,000. See Table 51 for unit cost breakdown. This area is shown by the maximum flood level illustrated on Plate 14.

TABLE 50

REAL ESTATE COST ESTIMATE
WET RESERVOIR

Lands and Damages	Acres	Unit Value	Total Value
Fee			
Pasture/Agricultural	4,345	$1,100	$4,779,500
Woodland	15,058	$900	$13,552,200
Residential	40	$1,800	$72,000
Sand & Gravel Pits	559	$2,000	$1,118,000
Roads & Waterbodies	148	N/A	
Improvements (101)*			$1,968,500
Severance Damage			0
Total			$21,490,000
Contingencies 25%			$5,373,000
Acquisition Costs (629 Tracts)			
Federal 629 @ $1,250 per tract			$786,000
Non-Federal 629 @ $2,500 per tract			$1,573,000
PL 91-646			$1,639,000
Total Estimated Real Estate Costs			$30,861,000

* The number of improvements includes two churches.

TABLE 51

REAL ESTATE COST ESTIMATE
DRY RESERVOIR

Lands and Damages	Acres	Unit Value	Total Value
Fee			
Pasture/Agricultural	2,996	$1,100	$3,295,600
Woodland	10,149	$900	$9,134,100
Residential	36	$1,800	$64,800
Sand & Gravel Pits	424	$2,000	$848,000
Roads & Waterbodies	145	N/A	
Improvements (77)*			$1,605,000
Severance Damage			0
Total			$14,948,000
Contingencies 25%			$3,737,000
Acquisition Costs (386 Tracts)			
Federal 386 @ $1,250 per tract			$483,000
Non-Federal 386 @ $2,500 per tract			$965,000
PL 91-646			$1,287,000
Total Estimated Real Estate Costs			$21,420,000

* The number of improvements includes two churches.

TABLE 52

REAL ESTATE COST ESTIMATE FOR ENVIRONMENTAL MITIGATION SITE IN ST.LANDRY PARISH, LOUISIANA

	WET ALTERNATIVE		
Lands and Damages	Acres	Unit Value	Total Value
Fee			
Pasture/Agricultural	10,350	$700	$7,245,000
Improvements			0
Severance Damage			0
Total			$7,245,000
Contingencies 25%			$1,811,000
Acquisition Costs (52 Tracts)			
Federal 52 @ $1,250 per tract			$65,000
Non-Federal 52 @ $2,500 per tract			$130,000
PL 91-646			0
Total Estimated Real Estate Costs			$9,251,000

NOTE:

This land is located within the West Atchafalaya Floodway. The floodway was acquired in the 1930's, and it has never been used. As a result, the market does not recognize the flowage easement encumbering the land. For this reason, full fee value has been estimated for this area.

TABLE 53

REAL ESTATE COST ESTIMATE FOR ENVIRONMENTAL MITIGATION SITE IN ST.LANDRY PARISH, LOUISIANA

	REDUCED WET ALTERNATIVE		
Lands and Damages	Acres	Unit Value	Total Value
Fee			
Pasture/Agricultural	4,000	$700	$2,800,000
Improvements			0
Severance Damage			0
Total			$2,800,000
Contingencies 25%			$700,000
Acquisition Costs (20 Tracts)			
Federal 20 @ $1,250 per tract			$25,000
Non-Federal 20 @ $2,500 per tract			$50,000
PL 91-646			0
Total Estimated Real Estate Costs			$3,575,000

NOTE:

This land is located within the West Atchafalaya Floodway. The floodway was acquired in the 1930's, and it has never been used. As a result, the market does not recognize the flowage easement encumbering the land. For this reason, full fee value has been estimated for this area.

TABLE 54

REAL ESTATE COST ESTIMATES FOR ROADWAY RELOCATIONS

Louisiana Highway 960 (All Alternatives)

Lands and Damages	Acres	Unit Value	Total Value
Perpetual Road Easement			
Woodland	14.5	$900	$13,000
Improvements			0
Severance Damage			0
Total			$13,000
Contingencies 25%			$3,000
Acquisition Costs (5 Tracts)			
Federal 5 @ $2,000 per tract			$10,000
Non-Federal 5 @ $2,500 per tract			$20,000
PL 91-646			0
Total Estimated Real Estate Costs			$46,000

TABLE 54 (Continued)

REAL ESTATE COST ESTIMATES FOR ROADWAY RELOCATIONS

LA Highway 448 (All Alternatives-Optional Relocation)

Lands and Damages	Acres	Unit Value	Total Value
Perpetual Road Easement			
Woodland	35	$900	$31,500
Pasture/Agriculture	5	$1,100	$5,500
Residential	1	$1,800	$1,800
Improvements			$340,000
Severance Damage			0
Total			$379,000
Contingencies 25%			$95,000
Acquisition Costs (43 Tracts)			
Federal 43 @ $1,250 per tract			$54,000
Non-Federal 43 @ $2,500 per tract			$108,,000
PL 91-646			$174,000
Total Estimated Real Estate Costs			$810,000

TABLE 54 (Continued)

REAL ESTATE COST ESTIMATES FOR ROADWAY RELOCATIONS

LA Highway 432 (All Alternatives-Optional Relocation)

Lands and Damages	Acres	Value	Value
Perpetual Road Easement			
Woodland	15.5	$900	$13,950
Residential	0.5	$1,800	$900
Improvements			$165,000
Severance Damage			0
Total			$180,000
Contingencies 25%			$45,000
Acquisition Costs (23 Tracts)			
Federal 23 @ $1,500 per tract			$35,000
Non-Federal 23 @ $3,000 per tract			$69,000
PL 91-646			$65,000
Total Estimated Real Estate Costs			$394,000

NOTE: There are a church and cemetery located within the right-of-way. This estimate does not include the costs of relocating the cemetery.

TABLE 54 (Continued)

REAL ESTATE COST ESTIMATES FOR ROADWAY RELOCATIONS

LA Highway 961 (All Alternatives-Optional Relocation)

Lands and Damages	Acres	Value	Value
Fee			
Woodland	11	$900	$9,900
Pasture/Agriculture	14	$1,100	$15,400
Residential	4	$1,800	$7,200
Improvements			$219,000
Severance Damage			0
Total			$252,000
Contingencies 25%			$63,000
Acquisition Costs (40 Tracts)			
Federal 40 @ $1,250 per tract			$50,000
Non-Federal 40 @ $2,500 per tract			$100,000
PL 91-646			$117,000
Total Estimated Real Estate Costs			$582,000

TABLE 55

REAL ESTATE COST ESTIMATES FOR UTILITY RELOCATIONS

GSU and DEMCO New Powerlines (Wet Alternative)

Lands and Damages	Acres	Value	Value
Perpetual Utility Easement			
Woodland	133	$900	$119,700
Improvements			0
Severance Damage			0
Total			$120,000
Contingencies 25%			$30,000
Acquisition Costs (28 Tracts)			
Federal 28 @ $1,250 per tract			$35,000
Non-Federal 28 @ $2,500 per tract			$70,000
PL 91-646			0
Total Estimated Real Estate Costs			$255,000

NOTE: Existing improvements will be avoided.

TABLE 55 (Continued)

REAL ESTATE COST ESTIMATES FOR UTILITY RELOCATIONS

GSU and DEMCO New Powerlines (Reduced Wet Alternative)

Lands and Damages	Acres	Value	Value
Perpetual Utility Easement Woodland	121	$900	$108,900
Improvements			0
Severance Damage			0
Total			$109,000
Contingencies 25%			$27,000
Acquisition Costs (25 Tracts)			
Federal 25 @ $1,500 per tract			$38,000
Non-Federal 25 @ $3,000 per tract			$75,000
PL 91-646			0
Total Estimated Real Estate Costs			$249,000

NOTE: Existing improvements will be avoided.

RECREATION RESOURCES

EXISTING RECREATION USE

There are no major outdoor recreational facility developments such as boat launching areas, campgrounds, or picnic areas in the study area. Public access to the Amite River is limited to the few major bridges and roadway crossings found along its course. Mostly all the lands adjacent to the river are held in private ownership. The river itself is accessible to the public because it is a state-owned water bottom.

The major recreational activities occurring in the study area are sport hunting, bank and some boat fishing, and canoeing and tubing on the river. Mostly all the lands suitable for hunting are privately owned and have either been leased by hunting clubs for the exclusive use of their members or posted against all uses by the land owners. Hunters pursue big game species such as white-tail deer, small game such as rabbits and squirrels, and migratory birds and waterfowl. Bank fishing occurs near bridge crossings, on private lands near the river, and where landowners have given permission for public access. Fishermen use shallow draft flat or john boats that are launched near bridge crossings or from private lands to gain access to a river that is sinewy, heavily shoaled, and sand-barred in many places. Typical fish species caught include large-mouth bass, sunfish, and catfish. Tubing and canoeing often begin at a point upriver and end at a pickup or rendezvous point down river. Other regional outdoor recreation areas include parish, city, and school facilities. They include the bulk of ball fields, playgrounds, and swimming pools in the area.

No existing use figures are available for the activities for the study area. The most recent license data obtained from the Louisiana Department of Wildlife and Fisheries indicates that 26,263 resident hunting licenses;

45,778 resident fishing licenses; and 33,442 motorboat registrations have been issued for 1990 in the five parishes (see list below) in the study area.

RECREATIONAL NEED, DEVELOPMENT, AND USE POTENTIAL

The 1988 Louisiana Statewide Comprehensive Outdoor Recreation Plan (SCORP) indicates that a substantial need exists to satisfy the demand for public recreation in State Planning Region 2 which includes the five parishes that border the Amite River - Ascension, East Baton Rouge, East Feliciana, Livingston, and St. Helena. A result of an analysis for the five parishes in the study area shows a net need requirement for camping (2015 tent sites, 876 recreational vehicle sites, and 1000 cabins) and picnicking areas (2217 picnic tables), fishing and boating areas (1238 boat ramp lanes), and big game and small game hunting areas (947,097 hunting acres). The close proximity to Baton Rouge and its growing suburbs makes the area more viable for recreational development. Travel time to the New Orleans metropolitan area is only an hour to an hour and a half by car.

The potential for developing and using reclaimed sand and gravel mining areas for public outdoor recreation is illustrated in some of the abandoned sand and gravel mining operations that have been inactive for a period of years. Many of these sites have naturally re-vegetated with pines and hardwoods over time. The sand and gravel pits have retained water from the water table after hydraulic mining and from rainfall forming clear, scenic lakes that have been stocked naturally or manually with a variety of game fish. A variety of game species that include deer, turkey, and squirrel is found on the lands surrounding these lakes. Areas where an attempt was made to clean the site and redistribute piles of overburden after mining appear more esthetically pleasing and natural.

Some of these re-vegetated areas that have not been

mined for ten to fifteen years offer a very natural environment for outdoor recreational use such as primitive camping, boating, hunting, and fishing. Most of these areas were leased by sand and gravel companies from the land owner during mining operations and are still privately owned. Recreational use is therefore restricted to the landowner and whomever he may permit. Many of the existing mining operations have the same land reclamation potential.

From the public use perspective, these reclaimed lands could be developed and managed for recreation purposes by a State, regional or local public entity (i.e., Louisiana State Parks, Louisiana Wildlife and Fisheries, Amite River Basin Commission, St. Helena Parish), or by a commercial operation interested in providing outdoor recreation or overnight camping. The development schemes could be as elaborate as regional parks with overnight cabins and day use areas offering picnicking and boat and bank fishing or as simple as a fishing lake surrounded by woods. The potential to develop these areas for public outdoor recreation use should be considered to meet future needs as population size and urban sprawl continue to increase.

PROPOSED RECREATIONAL DEVELOPMENT FOR DARLINGTON RESERVOIR

Wet Reservoir.

The development and construction of a full-sized wet reservoir would provide approximately 12,000 surface acres of lake for water-based recreational purposes and for recreational shoreline development. A conceptual recreation plan was developed to meet some of the needs for State Planning Region 2 described in the SCORP. This plan calls for the development of ten sites. A conceptual location for these sites is shown on Plate 38. The plan includes five major campgrounds with picnic areas and boat launches; four access areas with a boat launch; primitive camping and picnicking; and a high intensity day use area with tail water fishing, hiking, and nature trails. Annual use for the wet reservoir recreation plan is projected at approximately 1,800,000 visitors yielding approximately $9,000,000 in average annual benefits based upon an intermediate general recreation unit day value of $5.00/day over a 50 year project life. The $5.00/day unit day value was determined from planning guidance and procedures listed in ER 1105-2-100, dated 28 Dec 90. The average annual costs are projected at approximately $5,000,000 yielding a benefit-cost ratio of 1.9 to 1. A breakdown of costs and benefits for this conceptual recreation plan is shown in Table 56.

Dry Reservoir.

The development of a dry reservoir would provide approximately 14,240 acres for land and water-based recreation. A conceptual recreation plan was developed to meet some of the needs for State Planning Region 2 as detailed in the SCORP. This conceptual plan calls for the development of six sites shown on Plate 39. It includes a major park with camping and picnic areas, hiking and backpacking trails, a boat launch, and areas for canoeing, fishing, and swimming; a boat access area with picnicking,

hiking, canoeing, and fishing; a designated area developed for off-road vehicle use; a fishing impoundment with beach; an overnight cabin development close to several fishing lakes; and a high intensity day use area with canoeing, hiking, and nature trails. Annual use for the dry reservoir recreation plan is projected at approximately 1,100,000 visitors yielding approximately $5,500,000 in average annual benefits, again based upon an intermediate general recreation unit day value of $5.00/day over a 50 year project life. The average annual costs are projected at approximately $3,600,000 yielding a benefit-cost ratio of 1.5 to 1. A breakdown of benefits and costs for this conceptual recreation plan is shown in Table 57.

TABLE 56

ESTIMATED BENEFITS AND COSTS FOR THE FULL-SIZE WET RESERVOIR CONCEPTUAL RECREATION PLAN

SUMMARY:

TOTAL CONSTRUCTION COST		$41,425,000
GROSS INVESTMENT	$3,886,000	($45,000,000 GROSS INVESTMENT)
ANNUALIZED		
ANNUAL O&M COSTS	$900,000	
TOTAL ANNUAL COSTS	$4,786,000	
TOTAL ANNUAL BENEFITS	$8,900,000	
BENEFIT TO COST RATIO	1.86 : 1	

CONSTRUCTION COST ITEMS:

SITE #	ESTIMATED CONSTRUCTION COST*
1	$4,325,000
2	$6,275,000
3	$1,450,000
4	$4,475,000
5	$1,350,000
6	$5,525,000
7	$1,450,000
8	$7,875,000
9	$2,225,000
10	$6,475,000
TOTAL	$41,425,000

*Engineering design and construction management costs are included.

TABLE 56 (Continued)

CALCULATED BENEFITS:

SITE #	ESTIMATED ANNUAL VISITATION	UNIT DAY VALUE	CALCULATED ANNUAL VALUE
1	200,000	$5.00	$1,000,000
2	275,000	"	$1,375,000
3	40,000	"	$200,000
4	180,000	"	$900,000
5	50,000	"	$250,000
6	210,000	"	$1,050,000
7	45,000	"	$225,000
8	400,000	"	$2,000,000
9	30,000	"	$150,000
10	350,000	"	$1,750,000
TOTALS	1,780,000		$8,900,000

TABLE 57
ESTIMATED BENEFITS AND COSTS FOR THE
DRY RESERVOIR CONCEPTUAL RECREATION PLAN

SUMMARY:

TOTAL CONSTRUCTION COST		$31,375,000
GROSS INVESTMENT	$2,946,000	($34,250,000
ANNUALIZED		GROSS INVESTMENT)
ANNUAL O&M COSTS	$680,000	
TOTAL ANNUAL COSTS	$3,626,000	
TOTAL ANNUAL BENEFITS	$5,404,000	
BENEFIT TO COST RATIO	1.49 : 1	

CONSTRUCTION COST ITEMS:

SITE #	ESTIMATED CONSTRUCTION COST*
1	$4,725,000
2	$6,100,000
3	$3,800,000
4	$3,675,000
5	$5,350,000
6	$7,725,000
TOTAL	$31,375,000

* Engineering design and construction management costs are included.

CALCULATED BENEFITS:

SITE #	ESTIMATED ANNUAL VISITATION	UNIT DAY VALUE	CALCULATED ANNUAL VALUE
1	215,000	$5.00	$1,075,000
2	440,000	"	$2,200,000
3	95,000	"	$475,000
4	20,000	"	$100,000
5	230,000	"	$1,150,000
6	70,000	"	$350,000
Hunting	7,000	$7.75	$54,000
TOTALS	1,077,000		$5,404,000

NATIONAL ECONOMIC DEVELOPMENT BENEFITS

GENERAL In Appendix A of ER 1105-2-40, three primary categories of benefits for urban flood control plans are recognized: inundation reduction, intensification, and location benefits. Inundation reduction is the only primary category of National Economic Development (NED) benefits considered, since no significant differences in land or structure value in the study area can be readily observed strictly on the basis of flood risk. Lands and other properties with significantly different theoretical flood risks, but similar quality and size, command the same market values; thus, past price differentiation has been on the basis of factors other than flood risks.

Inundation reduction was considered only on existing structures. The calculation of inundation reduction benefits requires knowledge of the values and types of structures which would be inundated, and while an estimate is available of the number of acres expected to be developed in the flood plain during the project period, any projection of the values and types of structures to be built would be highly speculative at best.

No consideration was given to inundation on agricultural lands is the floodplain. Of the 357,800 acres flooded in the 1983 flood only 53,200 acres were agricultural and over half of this land was fallow at the time of the flood. Approximately $900,000 of agricultural damages were incurred. Average annual benefits from agricultural damages prevented would be very low and would not have a significant impact on plan selection.

INUNDATION REDUCTION BENEFITS Flood damages considered under this category were confined to physical damages and emergency costs. Average annual benefits derived from the prevention of flood damages were computed by determining the difference between expected annual losses without a project and the losses remaining after construction of proposed improvements. This benefit applies to those activities whose location decisions are unaffected by any proposed plan, i.e., those which would use the flood plain without a protection project. To the extent that land use is the same with or without a project, benefits accrue primarily through the reduction in actual or potential damages associated with existing land use characteristics.

EMERGENCY COSTS AND BENEFITS A significant cost that is incurred with a major storm is the emergency costs of fighting the flood and clean-up afterwards. Included with this is the damage inflicted on public property such as roads and bridges. Certain public property damages can be prevented, and some emergency costs reduced based on protection provided by the project. Reduction of these costs are a benefit creditable to the project.

OTHER BENEFITS When a protection project is completed, certain indirect benefits often result, and they contribute to the NED account. These other benefits which are considered in the study include a reduction in Flood Insurance Administration (FIA) costs, a reduction in the amount of fill material needed under the slabs of houses to be built in future years, and recreational benefits.

The net national cost of the flood insurance program is the cost of administering the program. Potential benefits would arise from a reduction in this administrative overhead, which consists of agent commissions, the costs of servicing, and claims adjustment. The current administrative cost per policy is $77.

Fill reduction benefits will accrue to future homeowners to the extent that a project lowers the elevation of the 100-year flood event, since for every one foot decrease in the 100-year event, one foot less of fill material will be required.

BENEFIT COMPUTATION - INUNDATION REDUCTION

Survey of Structures. In April of 1987 Gulf South Research Corporation (GSRC) of Baton Rouge was contracted to produce an inventory of structures within the 500-year flood plain of the Amite River Basin. Type, value, and first floor elevation were noted for each structure. In addition, ground elevation, type of foundation, number of stories, first floor square footage, type of construction, and the condition of the structure were recorded.

Residential Structures. This category was subdivided into single-story, two-story, and mobile homes. Values of these structures were determined using current real estate transfers, or current prices, or homes for sale minus lot prices. Computerized Real Estate Transfers (COMPTRAN) for the year 1985 was utilized in determining these values.

Ground elevations were determined using aerial photographs containing 2-foot contour lines. First floor elevations were determined by adding between 1.0 and 4.0 feet to ground elevations based upon visual observation.

In areas where housing types varied considerably, as in rural areas, information was obtained by surveying structure by structure. In urban areas, such as in homogeneous subdivisions, house value, number of stories, and floor elevations were generated from sampling.

Nonresidential Structures. These structures were surveyed by small subareas for pertinent characteristics such as type of business, number of stories, type of foundation, construction, physical condition, and dimensions. After all the field work was accomplished, the businesses were classified and placed into one of the fifteen following categories.

NONRESIDENTIAL CATEGORIES

1. Business Services
2. Public Gathering Places, Communications, Transportation, Utilities
3. Cleaning, Maintenance, Grooming
4. Contractor Operations
5. Department Stores
6. Eating and Drinking Establishments
7. Gas Service Stations
8. Grocery Stores
9. Food Stores (Specialty)
10. Home and Auto Supplies, Appliances
11. Medical Buildings
12. Proprietary (Drug) Stores
13. Repair Services
14. Ready-to-Wear
15. Miscellaneous

Each commercial structure was then assigned a value per square foot based on data obtained through the Marshall and Swift Valuation Service. Buildings were classified by construction types in order to determine the base cost per square foot. This base cost was then adjusted for factors such as heating and cooling, local construction cost, current cost conditions, age, and life expectancy of the building. The price per square foot was multiplied by the

square footage size of the building to determine a total value for each commercial structure. An average ground elevation was established based on structure location as pinpointed on the contour maps. First floor elevations were determined based on visual observations for each individual nonresidential structure.

Delineation of Subareas. The Amite River Basin contains 60 sub-basins. Plans studied in this analysis provide reduced damages in 23 of these sub-basins. Each sub-basin was broken into reaches based upon the hydrologic make-up of that particular sub-basin. Each hydrologic reach was further subdivided into segments called mini-reaches. Each mini-reach represents approximately a one-foot change in the elevation of the 100-year flood level at the center of the mini-reach. Plate 10 delineates each sub-basin, while Table 58 provides total structural value by reach within each sub-basin.

Depth-Damage Relationships for Structures and Contents. Depth-damage relationships are estimates of damages that would occur at varying depths of flooding. The relationships for the study area were compiled by 1/2-foot increments of flooding to a depth of 15 feet over the ground flood elevation. Damages are expressed as a percent of depreciated pre-flood structure and contents values.

(Residential) Under Corps of Engineers contract DACW38-79-C-0023 conducted in 1979-1980 for the Lake Pontchartrain Hurricane Protection Project (LPHPP) study, depth-damage relationships were developed which showed the relationship among contents values, structure values, and damages that would be sustained in both categories with various depths of flooding. Structure values were aggregated into three types: single-story, two-story, and mobile home. Table 59 presents depth-damages relationships for freshwater damage, the type which would occur by overflow within the Amite Basin. As construction types and general value ranges within the Amite Basin are similar to those found in the Lake Pontchartrain Basin, use of this data is appropriate.

For the LPHPP analysis, detailed room-by-room inventories of 125 sample residential structures were conducted. The contents were valued using standardized prices obtained from catalogs of major nationwide distributors. All items were depreciated in value to allow

for age, wear, and tear. Depreciated contents values were then related to depreciated structure values in order to develop relationships which express contents as a percent of structure value by ranges, as shown in Table 60. Depth-damage relationships for contents are shown in Table 61.

(Nonresidential) Nonresidential depth-damage relationships were based on contractor engineering expertise and field interviews which were carried out during the LPHPP analysis. Structural values were based on construction cost data published by Marshall and Swift Publication Company. Depth-damage relationships for nonresidential structures are shown in Table 62.

Relationships for nonresidential contents as a percent of structure values were also developed. These relationships were based on interviews with management personnel of sample structures, and included the value of all contents, i.e., machinery and equipment, supplies, furnishings, stock, goods in process, and finished goods. Table 63 displays the nonresidential contents/structure value relationships, while Table 64 shows the depth-damage relationships for nonresidential contents.

Stage Frequency Curves. Stage-frequency curves, which express the annual probability of various levels of flooding under each plan analyzed, were developed for each reach (See Hydrologic and Hydraulic Design Section for stage calculated frequency data for selected points in the river basin).

DAMAGE ANALYSIS

Damage Computations. Damage was calculated using the Hydrologic Engineering Center's (HEC) Flood Damage Analysis Package, flood plain structure inventories, and the depth-damage relationships previously described, together with stage-frequency curves for each hydraulically consistent reach. Stage-frequency curves were adjusted for slope within each reach by one foot incremental changes in the 100-year reference flood water surface elevations. These reference flood elevations were assigned to structure cards according to minibands, each representing a one foot increment in the reference flood elevation at its center, and designed to cut a path perpendicular to the flows of the various rivers and tributaries while taking into

account hydraulic obstructions and geographic inconsistencies.

The Structure Inventory for Damage Analysis (SID) computer program was used to calculate elevation-damage curves. Adjustments were made to the data to correct for unnaturally extreme or frequent occurrences of flooding, and the hydraulic data was adjusted in coordination with the Hydraulics and Hydrology Branch (H&H) in order to account for the occurrence of natural levees along stream banks. Zero damage elevations were established at or above the two year storm elevation in most cases and the appropriate increment established to develop well-defined elevation-damage curves which were stored in a random access file, the HEC Data Storage System (HECDSS) file, under pathnames matching those of the stored elevation-probability data provided by H&H.

The Expected Annual Flood Damage Computation (EAD) computer program was used to access the HECDSS file and weight the damage corresponding to each magnitude of flooding by the percent chance of exceedance, and then sum the weighted damage to determine the expected annual damage. Damages were calculated for single-family one-story and two-story homes; mobile homes; the contents of all residential structures; commercial structures; and the contents of commercial structures, for each reach and for all with and without project conditions considered for each basin.

DAMAGE PRESENTATION. Average annual damages to existing development and damages prevented for each hydrologic condition for the wet and dry reservoir are displayed in Table 65. Based on comparable data in the Comite River Diversion Report, an additional 20% of benefits was added to these inundation reduction benefits to account for other benefit categories such as vehicle inundation reduction, reduction in emergency cost, Federal insurance administration costs saved, and fill reduction benefits.

TABLE 58

DARLINGTON RESERVOIR
TOTAL STRUCTURAL VALUE BY SUB-BASIN

SUB-BASIN NUMBER	SUB-BASIN NAME	CATEGORY OF STRUCTURE	NUMBER OF STRUCTURES	VALUE OF STRUCTURES (IN THOUSANDS)	EXPECTED ANNUAL DAMAGE (IN THOUSANDS)
14	BEAVER BAYOU	1-STORY	1,247	69,938	6,673.69
		2-STORY	53	3,549	196.49
		MOBILE HOME	252	3,011	159.76
		COMMERCIAL	247	49,567	1,209.66
		TOTAL	1,799	126,065	8,239.60
18	COMITE RIVER & SHOE CREEK	1-STORY	1,024	55,353	716.33
		2-STORY	80	4,153	81.82
		MOBILE HOME	73	785	4.68
		COMMERCIAL	53	8,856	69.21
		TOTAL	1,200	69,147	872.04
19	COMITE RIVER & DRAUGHAN CREEK	1-STORY	845	38,037	225.29
		2-STORY	84	5,492	5.75
		MOBILE HOME	26	302	0.52
		COMMERCIAL	63	4,883	12.69
		TOTAL	1,018	48,714	244.25
22	JONES CREEK	1-STORY	1,503	116,806	519.81
		2-STORY	301	29,352	75.28
		MOBILE HOME	9	99	2.15
		COMMERCIAL	380	53,393	183.58
		TOTAL	2,193	199,650	779.52
28	WEINER CREEK	1-STORY	295	38,285	63.26
		2-STORY	42	7,575	1.27
		MOBILE HOME	1	10	0
		COMMERCIAL	22	3,708	0.19
		TOTAL	360	50,548	64.72
29	BAYOU FOUNTAIN	1-STORY	494	27,169	327.38
		2-STORY	332	25,220	96.83
		MOBILE HOME	6	72	0.04
		COMMERCIAL	215	29,154	608.38
		TOTAL	1,047	81,615	1,032.63

TABLE 58

DARLINGTON RESERVOIR
TOTAL STRUCTURAL VALUE BY SUB-BASIN

SUB-BASIN NUMBER	SUB-BASIN NAME	CATEGORY OF STRUCTURE	NUMBER OF STRUCTURES	VALUE OF STRUCTURES (IN THOUSANDS)	EXPECTED ANNUAL DAMAGE (IN THOUSANDS)
31	CLAYCUT BAYOU	1-STORY	1,040	57,821	613.28
		2-STORY	107	8,661	187.71
		MOBILE HOME	114	1,037	14.58
		COMMERCIAL	87	38,521	198.99
		TOTAL	1,348	106,040	1,014.56
32	LOWER WARD'S CREEK	1-STORY	337	15,809	381.55
		2-STORY	27	2,597	26.46
		MOBILE HOME	22	898	5.82
		COMMERCIAL	78	8,309	437.03
		TOTAL	464	27,613	851.86
42	AMITE RIVER	1-STORY	180	7,517	8.37
		2-STORY	19	1,089	1.70
		MOBILE HOME	121	1,343	0.06
		COMMERCIAL	28	1,718	5.93
		TOTAL	348	11,657	16.66
43	MUDDY CREEK	1-STORY	183	8,513	102.40
		2-STORY	15	922	14.00
		MOBILE HOME	105	1,183	29.51
		COMMERCIAL	13	752	1.40
		TOTAL	316	11,370	147.31
44	AMITE RIVER	1-STORY	939	42,368	1,372.51
		2-STORY	61	5,007	155.32
		MOBILE HOME	589	6,608	173.94
		COMMERCIAL	79	9,560	108.69
		TOTAL	1,688	63,543	1,810.46
45	COLYELL CREEK	1-STORY	357	16,356	734.67
		2-STORY	34	2,052	14.64
		MOBILE HOME	233	2,479	76.42
		COMMERCIAL	36	1,436	140.23
		TOTAL	660	22,323	965.96

TABLE 56

DARLINGTON RESERVOIR
TOTAL STRUCTURAL VALUE BY SUB-BASIN

SUB-BASIN NUMBER	SUB-BASIN NAME	CATEGORY OF STRUCTURE	NUMBER OF STRUCTURES	VALUE OF STRUCTURES (IN THOUSANDS)	EXPECTED ANNUAL DAMAGE (IN THOUSANDS)
46	GRAY'S CREEK	1-STORY	645	31,637	1,138.68
		2-STORY	49	2,954	95.78
		MOBILE HOME	385	4,482	73.39
		COMMERCIAL	64	2,536	47.12
		TOTAL	1,143	41,609	1,354.97
47	AMITE RIVER	1-STORY	786	44,439	1,082.17
		2-STORY	85	6,940	130.51
		MOBILE HOME	102	1,177	24.85
		COMMERCIAL	145	22,078	191.55
		TOTAL	1,118	74,634	1,429.08
48	HONEY CUT	1-STORY	621	40,650	295.67
		2-STORY	44	5,564	5.38
		MOBILE HOME	9	117	0.58
		COMMERCIAL	125	32,853	98.19
		TOTAL	799	79,184	399.82
49	MILLER CANAL	1-STORY	509	25,904	2,292.84
		2-STORY	38	2,447	148.94
		MOBILE HOME	346	3,828	212.22
		COMMERCIAL	147	23,344	1,314.29
		TOTAL	1,040	55,523	3,968.29
50	UPPER GRAY'S CREEK	1-STORY	600	27,469	332.29
		2-STORY	12	809	9.87
		MOBILE HOME	135	1,413	48.47
		COMMERCIAL	165	25,487	120.21
		TOTAL	912	55,175	570.84
51	UPPER AMITE RIVER	1-STORY	1,037	47,724	321.27
		2-STORY	51	3,181	11.57
		MOBILE HOME	287	3,104	44.46
		COMMERCIAL	178	22,541	60.71
		TOTAL	1,553	76,530	438.01

TABLE 58

DARLINGTON RESERVOIR
TOTAL STRUCTURAL VALUE BY SUB-BASIN

SUB-BASIN NUMBER	SUB-BASIN NAME	CATEGORY OF STRUCTURE	NUMBER OF STRUCTURES	VALUE OF STRUCTURES (IN THOUSANDS)	EXPECTED ANNUAL DAMAGE (IN THOUSANDS)
52	UPPER AMITE RIVER	1-STORY	200	8,939	52.83
		2-STORY	14	840	3.33
		MOBILE HOME	105	1,132	4.16
		COMMERCIAL	10	420	1.04
		TOTAL	329	11,331	61.36
54	UPPER AMITE RIVER	1-STORY	272	12,689	83.44
		2-STORY	24	1,837	17.04
		MOBILE HOME	130	1,444	9.12
		COMMERCIAL	59	28,655	7.12
		TOTAL	485	44,625	116.72
55	UPPER AMITE RIVER	1-STORY	22	1,560	5.66
		2-STORY	4	240	0
		MOBILE HOME	15	165	0.32
		COMMERCIAL	2	118	0.16
		TOTAL	43	2,083	6.14
59	UPPER AMITE RIVER	1-STORY	899	30,390	376.84
		2-STORY	54	2,645	31.63
		MOBILE HOME	471	4,844	121.74
		COMMERCIAL	88	4,973	5.58
		TOTAL	1,512	42,852	535.79
64	BAYOU MANCHAC	1-STORY	268	12,784	212.73
		2-STORY	42	2,131	50.19
		MOBILE HOME	154	1,633	55.02
		COMMERCIAL	17	1,752	18.95
		TOTAL	481	18,300	336.89
TOTALS FOR ALL BASINS		1-STORY	14,303	779,127	17,992.26
		2-STORY	1,562	125,234	1,361.51
		MOBILE HOME	3,690	41,166	1,062.81
		COMMERCIAL	2,301	374,614	4,840.90
		TOTAL	21,856	1,320,141	25,257.48

Table 59
FRESHWATER DEPTH-DAMAGE RELATIONSHIPS
for
Residential Structures

Depth of Flooding	Percent Damage to Structure		
	One Story	Two Story	Mobile Home
(Ft.)	(%)	(%)	(%)
-1.0	0.0	0.0	0.0
-0.5	0.0	0.5	2.0
0.0	9.0	7.5	30.0
0.5	18.7	12.3	63.5
1.0	26.0	15.7	76.8
1.5	32.3	18.3	87.0
2.0	36.7	20.7	92.0
2.5	40.4	22.8	95.6
3.0	43.0	24.4	97.8
3.5	45.6	25.8	99.0
4.0	48.0	26.8	99.8
4.5	49.8	27.9	100.0
5.0	51.4	28.6	100.0
5.5	53.0	29.2	100.0
6.0	54.2	29.9	100.0
6.5	55.3	30.0	100.0
7.0	56.0	30.0	100.0
7.5	57.0	30.0	100.0
8.0	57.8	30.7	100.0
8.5	58.4	33.2	100.0
9.0	59.0	37.0	100.0
9.5	59.5	40.0	100.0
10.0	59.9	42.3	100.0
10.5	60.0	44.3	100.0
11.0	60.0	46.1	100.0
11.5	60.0	47.5	100.0
12.0	60.0	48.2	100.0
12.5	60.0	49.3	100.0
13.0	60.0	49.9	100.0
13.5	60.0	50.1	100.0
14.0	60.0	50.3	100.0
14.5	60.0	50.4	100.0
15.0	60.0	50.5	100.0

Source: DACW38-79-C-0023 Output, October 1980, US Army Engineer District, New Orleans, contract with CH2MHill.

Table 60

CONTENTS AS A PERCENT
OF STRUCTURAL VALUE
Residential

Structure Value Range ($)	Contents as a Percent of Structure Value (%)	Contents Value Range ($)
000 - 10,000	75	000 - 7,500
10,001 - 20,000	73	7,300 - 14,600
20,001 - 30,000	70	14,600 - 21,000
30,001 - 40,000	66	21,000 - 26,400
40,001 - 50,000	62	26,400 - 31,000
50,001 - 60,000	57	31,000 - 34,200
60,001 - 70,000	53	34,200 - 37,100
70,001 - 80,000	51	37,100 - 40,800
80,001 - 90,000	49	40,800 - 44,100
90,001 -100,000	48	44,100 - 48,000
>100,000	47	>48,000

Source: DACW-38-79-C-0023 Output, October 1980, US Army Engineer District, New Orleans contract with CH2MHill, updated to 1983 pric

Table 61
FRESHWATER DEPTH-DAMAGE RELATIONSHIPS
for
Residential Contents

Depth of Flooding	Percent Damage to Contents	
	One Story & Mobile Homes	Two Story
(Ft.)	(%)	(%)
0.0	0.0	0.0
0.5	11.5	10.0
1.0	21.5	16.0
1.5	31.0	20.0
2.0	39.7	23.6
2.5	46.8	26.2
3.0	52.5	28.1
3.5	57.5	30.0
4.0	61.7	30.3
4.5	64.8	32.4
5.0	67.3	33.5
5.5	69.3	34.3
6.0	70.7	35.0
6.5	71.7	35.5
7.0	72.6	36.0
7.5	73.6	36.5
8.0	74.1	37.0
8.5	74.8	37.4
9.0	75.7	37.9
9.5	76.1	38.5
10.0	76.6	39.3
10.5	77.1	41.6
11.0	77.5	44.8
11.5	77.8	47.5
12.0	78.0	50.5
12.5	78.1	53.0
13.0	78.2	55.9
13.5	78.4	59.0
14.0	78.6	62.0
14.5	78.8	64.6
15.0	79.0	66.9

Source: DACW38-79-C-0023 Output, October 1980, US Army Engineer District, New Orleans, contract with CH2MHill.

Table 62
FRESHWATER DEPTH-DAMAGE RELATIONSHIP
for
Non-Residential Structures

Depth of Flooding (Ft.)	Percent Damage to Structures (%)
0.0	0.0
0.5	4.9
1.0	8.6
1.5	11.9
2.0	14.3
2.5	16.7
3.0	18.4
3.5	19.5
4.0	20.6
4.5	21.4
5.0	22.0
5.5	22.1
6.0	22.2
6.5	22.3
7.0	22.4
7.5	23.0
8.0	23.8
8.5	24.7
9.0	25.9
9.5	27.3
10.0	29.0
10.5	30.8
11.0	33.0
11.5	35.2
12.0	37.0
12.5	38.8
13.0	40.0
13.5	41.2
14.0	41.9
14.5	42.1
15.0	42.3

Source: DACW38-79-C-0023 Output, October 1980, US Army Engineer District, New Orleans, contract with CH2MHill.

Table 63

NON-RESIDENTIAL CONTENTS VALUE
EXPRESSED AS A PERCENT OF STRUCTURAL VALUE

Category	Contents Value (% of Structural Value)
	(%)
Business Services	109
Public Gathering Places	24
Cleaning, Maintenance and Grooming	209
Contractor Operations	97
Department Stores	205
Eating and Drinking Establishments	102
Gas Service Stations	83
Grocery Stores	84
Food Stores (Specialty)	98
Home and Auto Supplies, Appliances	127
Medical Buildings	41
Proprietary (Drug) Stores	129
Repair Service	152
Ready-to-Wear	190
Miscellaneous	113

Source: DACW38-79-C-0023 Output, October 1980, US Army Engineer District, New Orleans contract with CH2MHill.

Table 64
FRESHWATER DEPTH-DAMAGE RELATIONSHIPS
for
Non-Residential Contents

Depth of Flooding Over Ground Floor	Business Services	Pub Gathering, Communications, Transportation, Utilities	Cleaning, Maintenance, Grooming	Contractor Operations	Department Stores	Eating and Drinking Establishments	Gas Service Stations	Grocery Stores
(Ft.)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.5	14.0	4.0	21.0	14.5	2.0	3.0	10.0	30.0
1.0	25.0	7.8	36.0	21.0	9.0	13.8	15.5	40.0
1.5	32.5	10.7	46.8	27.0	15.0	24.0	21.3	47.0
2.0	39.5	13.2	55.0	33.0	20.0	32.0	25.6	52.0
2.5	45.0	15.2	61.0	37.5	25.0	39.0	32.5	56.8
3.0	50.0	17.0	66.0	42.0	29.0	45.0	39.9	60.3
3.5	54.0	18.9	69.0	45.9	33.1	50.5	44.9	64.0
4.0	57.5	20.0	71.0	50.5	37.3	54.7	50.5	67.0
4.5	60.0	21.5	72.8	54.0	40.5	58.8	55.5	70.0
5.0	62.0	22.9	74.0	57.0	43.5	62.0	60.0	72.5
5.5	63.2	24.0	75.0	61.0	46.0	64.3	64.5	74.8
6.0	64.0	25.0	76.0	64.5	48.7	66.0	69.0	76.8
6.5	64.7	26.0	77.5	67.5	51.0	67.7	72.8	78.0
7.0	65.2	27.0	78.0	70.0	53.0	69.0	76.5	79.5
7.5	65.8	28.0	78.7	72.0	55.0	70.2	80.3	80.7
8.0	66.0	29.2	79.4	74.0	57.0	71.0	84.0	81.8
8.5	66.0	30.1	79.8	75.5	59.0	72.0	87.0	82.3
9.0	65.0	31.0	80.0	77.5	60.8	72.7	90.0	83.3
9.5	66.2	32.0	80.0	78.5	62.3	73.5	92.0	84.0
10.0	66.3	32.7	80.0	80.0	63.8	74.0	93.8	84.5
10.5	66.4	33.5	80.0	81.0	65.0	74.8	94.7	85.3
11.0	66.5	34.0	80.1	82.0	66.1	75.7	95.9	85.7
11.5	66.6	34.5	80.1	82.5	67.5	76.0	95.2	86.0
12.0	66.8	34.9	80.1	83.0	68.3	76.2	95.7	86.1
12.5	67.0	35.5	80.1	83.5	69.7	76.4	97.0	86.1
13.0	67.3	35.8	80.2	84.0	70.6	76.6	97.4	86.2
13.5	67.4	35.9	80.2	84.5	71.5	76.8	97.5	86.2
14.0	67.6	36.0	80.3	85.0	72.1	77.0	97.6	86.3
14.5	67.8	36.2	80.3	85.4	73.0	77.2	97.7	86.4
15.0	68.0	36.4	80.4	85.8	73.9	77.4	97.8	86.5

Table 64
FRESHWATER DEPTH-DAMAGE RELATIONSHIPS
for
Non-Residential Contents

Depth of Flooding Over Ground Floor	Specialty Food Stores	Home and Auto Supplies, Appliances	Medical Buildings	Proprietary (Drug) Stores	Repair Services	Ready-to-Wear	Miscellaneous
(Ft.)	(%)	(%)	(%)	(%)	(%)	(%)	(%)
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.5	13.0	26.0	29.0	11.0	14.0	16.0	14.7
1.0	28.0	31.5	33.2	18.0	22.0	33.3	24.0
1.5	36.7	37.3	35.5	26.6	30.0	46.0	31.5
2.0	43.7	42.4	37.7	34.5	37.4	57.1	38.2
2.5	50.0	48.0	39.4	43.0	43.3	65.0	43.8
3.0	55.0	53.8	40.9	51.0	54.0	72.0	48.9
3.5	58.4	59.2	42.2	60.0	62.8	77.5	53.3
4.0	61.4	63.7	43.7	68.4	71.0	82.0	57.4
4.5	64.4	67.0	44.8	77.0	77.9	85.0	60.8
5.0	66.7	69.5	46.0	85.0	83.8	87.7	63.6
5.5	69.2	71.8	47.0	93.1	89.1	90.1	66.1
6.0	70.8	73.6	47.9	97.2	92.8	92.0	68.1
6.5	72.3	75.2	48.9	99.0	95.6	93.6	69.8
7.0	73.7	76.5	49.7	99.0	97.8	94.8	71.3
7.5	75.0	77.8	50.3	99.1	99.0	96.0	72.4
8.0	76.0	78.8	51.0	99.3	100.0	97.1	73.5
8.5	77.0	79.8	51.8	99.5	100.0	98.2	74.3
9.0	78.0	80.3	52.0	99.7	100.0	98.3	75.1
9.5	78.7	80.9	52.6	99.8	100.0	98.9	75.7
10.0	79.4	81.3	53.0	99.9	100.0	99.5	76.3
10.5	80.0	81.6	53.5	99.9	100.0	100.0	76.8
11.0	80.5	81.8	53.9	100.0	100.0	100.0	77.3
11.5	81.3	82.0	54.2	100.0	100.0	100.0	77.6
12.0	81.7	82.1	54.4	100.0	100.0	100 0	77.8
12.5	82.0	82.1	54.8	100.0	100.0	100.0	78.1
13.0	82.1	82.2	55.1	100.0	100.0	100.0	78.4
13.5	82.2	82.2	55.4	100.0	100.0	100.0	78.5
14.0	82.3	82.2	55.7	100.0	100.0	100.0	78.8
14.5	82.3	83.3	55.8	100.0	100.0	100.0	79.0
15.0	82.3	83.3	55.9	100.0	100.0	100.0	79.2

TABLE 65
CALCULATED ANNUAL FLOOD DAMAGES AND PROJECT FLOOD CONTROL BENEFITS

BASIN #	AVERAGE ANNUAL DAMAGES W/O PROJECT	AVERAGE ANNUAL BENEFITS (000'S) DRY RESERVOIR 25-YR	DRY RESERVOIR 50-YR	DRY RESERVOIR 100-YR	REDUCED WET RESERVOIR W/ SERVICE SPILLWAY 25-YR	REDUCED WET RESERVOIR W/ SERVICE SPILLWAY 50-YR	REDUCED WET RESERVOIR W/ SERVICE SPILLWAY 100-YR	REDUCED WET RESERVOIR W/O SERVICE SPILLWAY 25-YR	REDUCED WET RESERVOIR W/O SERVICE SPILLWAY 50-YR	REDUCED WET RESERVOIR W/O SERVICE SPILLWAY 100-YR	LARGE WET RESERVOIR W/ SERVICE SPILLWAY 25-YR	LARGE WET RESERVOIR W/ SERVICE SPILLWAY 50-YR	LARGE WET RESERVOIR W/ SERVICE SPILLWAY 100-YR	LARGE WET RESERVOIR W/O SERVICE SPILLWAY 25-YR	LARGE WET RESERVOIR W/O SERVICE SPILLWAY 50-YR	LARGE WET RESERVOIR W/O SERVICE SPILLWAY 100-YR
14	$8,239.60	$30.89	$32.55	$33.99	$27.57	$30.10	$31.39	$31.13	$32.74	$34.01	$27.57	$30.10	$31.39	$31.13	$32.74	$34.01
18	$872.04	$104.09	$109.44	$113.07	$93.80	$102.01	$105.60	$128.64	$133.42	$136.97	$93.80	$102.01	$105.60	$104.19	$108.97	$112.52
19	$244.25	$216.84	$223.10	$226.64	$199.61	$211.94	$218.29	$216.30	$222.79	$225.82	$199.61	$211.94	$218.29	$215.82	$222.31	$225.34
22	$779.52	$66.90	$67.66	$68.00	$63.46	$65.31	$66.22	$65.90	$66.51	$66.79	$63.46	$65.31	$66.22	$65.90	$66.51	$66.79
28	$64.72	$62.99	$63.06	$63.16	$62.75	$62.89	$63.06	$62.99	$63.16	$63.16	$62.87	$62.89	$63.06	$62.99	$63.16	$63.16
29	$1,032.63	$33.93	$34.67	$34.96	$28.89	$29.58	$29.99	$29.31	$30.20	$30.46	$28.89	$29.58	$29.99	$29.31	$30.20	$30.46
31	$1,014.56	$654.35	$683.55	$699.62	$586.35	$638.21	$661.44	$640.88	$674.79	$687.64	$586.35	$638.21	$661.44	$640.88	$674.79	$687.64
32	$851.86	$55.14	$55.79	$55.79	$51.78	$52.27	$52.23	$52.74	$52.94	$53.05	$51.78	$52.27	$53.23	$52.74	$52.94	$53.05
42	$16.66	$12.37	$13.74	$14.42	$12.37	$13.74	$14.42	$12.37	$13.74	$14.42	$12.37	$13.74	$14.42	$12.37	$13.74	$14.42
43	$147.31	$81.18	$85.62	$87.75	$67.25	$74.78	$79.97	$79.17	$83.72	$84.95	$67.25	$74.78	$79.97	$79.17	$83.72	$84.95
44	$1,810.46	$1,467.41	$1,522.95	$1,556.28	$1,361.36	$1,440.86	$1,484.59	$1,430.95	$1,490.96	$1,524.26	$1,359.84	$1,439.35	$1,483.11	$1,429.41	$1,489.44	$1,522.74
45	$965.96	$611.67	$622.53	$627.27	$611.67	$633.53	$627.27	$650.12	$659.94	$664.24	$611.67	$622.53	$627.27	$611.67	$633.53	$627.27
46	$1,354.97	$397.07	$421.92	$433.52	$397.07	$420.55	$433.52	$397.07	$420.55	$433.52	$397.07	$420.55	$433.52	$397.07	$420.55	$433.52
47	$1,429.08	$1,319.68	$1,366.64	$1,391.65	$1,216.55	$1,299.54	$1,344.56	$1,323.76	$1,371.24	$1,393.00	$1,216.55	$1,299.54	$1,344.50	$1,323.76	$1,371.24	$1,393.00
48	$399.82	$368.21	$390.91	$397.98	$314.54	$353.77	$382.05	$383.45	$396.74	$399.47	$314.54	$353.77	$382.05	$383.45	$396.74	$399.47
49	$3,960.29	$1,098.37	$1,099.43	$1,099.76	$1,099.11	$1,099.43	$1,099.76	$1,098.37	$1,099.43	$1,099.76	$1,098.37	$1,099.43	$1,099.76	$1,098.37	$1,099.43	$1,099.76
50	$570.84	$223.75	$227.08	$227.76	$223.75	$227.08	$227.76	$223.75	$227.08	$227.76	$223.75	$227.08	$227.76	$223.75	$227.08	$227.76
51	$438.01	$431.28	$435.95	$437.31	$414.23	$427.83	$433.94	$433.71	$436.88	$437.58	$414.23	$427.83	$433.94	$433.71	$436.88	$437.58
52	$61.36	$60.62	$61.28	$61.33	$55.80	$59.75	$61.10	$60.69	$61.29	$61.35	$55.80	$59.75	$61.10	$60.69	$61.29	$61.35
54	$116.72	$112.43	$115.43	$116.33	$103.76	$110.77	$114.35	$111.33	$114.95	$116.11	$103.76	$110.77	$114.35	$111.33	$114.95	$116.11
55	$6.14	$5.83	$6.12	$6.14	$5.06	$5.65	$6.01	$5.71	$6.09	$6.14	$5.06	$5.65	$6.01	$5.71	$6.09	$6.14
59	$535.79	$352.08	$370.73	$377.71	$352.08	$370.73	$377.71	$352.08	$370.73	$377.71	$352.08	$370.73	$377.71	$352.08	$370.73	$377.71
64	$336.89	$226.30	$233.89	$237.77	$194.88	$208.51	$214.66	$212.85	$221.62	$224.22	$194.88	$208.27	$214.66	$212.85	$221.62	$224.22
TOTAL	$25,257.48	$7,993.38	$8,244.04	$8,368.21	$7,543.69	$7,938.83	$8,129.89	$8,003.27	$8,251.51	$8,362.39	$7,541.55	$7,926.08	$8,129.35	$7,938.35	$8,198.65	$8,298.97
PRICE INDEXED 1.109	$28,010.55	$8,864.66	$9,142.64	$9,280.34	$8,365.95	$8,804.16	$9,016.05	$8,875.63	$9,150.92	$9,273.89	$8,363.58	$8,790.02	$9,015.45	$8,803.63	$9,092.30	$9,203.56
INCLUDING ADD. CATEGORIES 20%	$33,612.65	$10,637.59	$10,971.17	$11,136.41	$10,039.14	$10,564.99	$10,819.26	$10,650.75	$10,981.11	$11,128.67	$10,036.29	$10,548.03	$10,818.54	$10,564.36	$10,910.76	$11,044.27

EVALUATION SUMMARY

Early in the evaluation of the 15 final alternatives it was determined that service spillways were too costly. Six alternatives, 25, 50, and 100 year Wet and Reduced Wet with Service Spillway, were eliminated.

In order to evaluate the overall feasibility of the remaining nine final alternatives annual costs and benefits were calculated in detail for the Full Size Wet and Dry Reservoirs providing 50 year level flood protection. An interest rate of 8.5 percent over a 50 year project life was used to annualize initial project costs. A five year construction period was used to calculate gross investment amounts. Table 66 lists each cost component, annualized costs and benefits for both alternatives. As shown, both the 50 year Full Size Wet and Dry Reservoir alternatives were determined to be infeasible with benefit cost ratios of 0.44 and 0.57, respectively. While costs for relocations, real estate, and environmental mitigation are significant, the high cost of the dam structure itself appears to be the most critical factor affecting overall project feasibility.

From the results of these cost estimates it was concluded that all alternatives with larger dam structures are also infeasible. Also, it was determined that the benefits produced by including recreational improvements in both the wet and dry alternatives exceed the costs directly attributed to these improvements. However, the cost of raising the dam structure height to achieve design flood protection with both a full and reduced size recreational lake offset the net recreation benefits of the Wet and Reduced Wet Alternatives. This is demonstrated in Figure 1 where it can be seen that the increase in dam height for the Wet and Reduced Wet Reservoirs significantly increases the dam structure cost at various levels of design flood protection.

The cost effectiveness of the smallest structure (Dry Reservoir with 25 year design protection) was analyzed. Again, with reference to Figure 1 it can be seen that the cost curve for the dam structure "flattens" at these lower design elevations. While benefits stay relatively high in this range costs do not reduce significantly enough to bring the benefit cost ratio up substantially. The

estimated costs and benefits for a 25 year Dry Reservoir were estimated as follows:

Total Initial Cost	$236,000,000
Total Gross Investment	$291,321,000
Gross Investment Equivalent Annual Cost	$25,187,000
Annual Operation and Maintenance Cost	$800,000
Total Equivalent Annual Cost	$25,987,000
Total Annual Flood Control and Recreation Benefits	$15,638,000
Benefit/Cost Ratio	0.60

Other studies have concluded that secondary net benefits from potential water supply and hydro-electric power sources are not significant due to the fact that adequate future water and power needs currently exist. Inclusion of these factors have little impact on the cost-effectiveness of these plans.

From the above results, it was concluded that all reservoir alternatives are infeasible.

In comparison to the previous studies cited, calculated costs and benefits were generally comparable with the exception of the dam structure itself. With the benefit of having these studies in-hand, a more comprehensive analysis of geotechnical conditions was afforded. The results of the additional soil borings taken dictated that a more extensive dam structure is required. Specifically, weaker soil strengths require a much larger dam base and section. Higher soil porosities (presence of sand and gravel) require the installation of a slurry trench cut-off wall averaging 70-80 feet in depth. The cost for each dam's emergency spillway was also determined to be significantly higher than addressed in previous reports since it was determined that a steel reinforced concrete cap is required to prevent erosion and failure of the spillway structure.

TABLE 66
ESTIMATED RESERVOIR COSTS AND BENEFITS

ITEM	FULL SIZE WET RESERVOIR (50 YEAR DESIGN)	DRY RESERVOIR (50 YEAR DESIGN)
DAM STRUCTURE	$282,000,000	$193,000,000
RELOCATIONS	$32,100,000	$9,209,000
REAL ESTATE	$30,861,000	$21,420,000
ENVIRONMENTAL MITIGATION	$27,449,000	$5,726,000
RECREATION	$41,425,000	$31,375,000
TOTAL INITIAL COST	$413,835,000	$260,730,000
TOTAL GROSS INVESTMENT	$510,843,000	$321,849,000
GROSS INVESTMENT EQUIVALENT ANNUAL COST	$44,168,000	$27,826,000
ANNUAL OPERATION AND MAINTENANCE		
-DAM AND RESERVOIR	$175,000	$85,000
-RECREATION FACILITIES	$900,000	$680,000
-ENV. MITIGATION AREAS	$269,000	$79,000
TOTAL ANNUAL O&M	$1,344,000	$844,000
TOTAL EQUIVALENT ANNUAL COST AND O&M	$45,512,000	$28,670,000
ANNUAL FLOOD CONTROL BENEFITS	$10,911,000	$10,971,000
ANNUAL RECREATION BENEFITS	$8,900,000	$5,404,000
TOTAL ANNUAL BENEFITS	$19,811,000	$16,375,000
BENEFIT/COST RATIO	0.44	0.57

RECOMMENDATION

As the District Engineer, I have found that this report contains a thorough analysis and evaluation of all practicable alternatives regarding the construction of a dam and reservoir on the upper Amite River. From the findings of this report, I have concluded that no plan of action considered produces a net excess benefit over its cost. I have therefore determined that construction of a dam and reservoir on the upper Amite River is not in the public interest.

This recommendation reflects the information available at this time and current departmental policies governing formulation of individual projects. It does not reflect program and budgeting priorities inherent in the formulation of a national civil works construction program nor the prospect of higher review levels within the Executive Branch.

Michael Diffley
Colonel, Corps of Engineers
District Engineer

APPENDIX A

AGREEMENT BETWEEN

THE UNITED STATES OF AMERICA

AND

THE STATE OF LOUISIANA

FOR THE AMITE RIVER AND TRIBUTARIES STUDY

THIS AGREEMENT, entered into this 5th day, of February 1985, by and between the United States of America (hereinafter called the "Government") represented by the Contracting Officer executing this Agreement and the State of Louisiana (Hereinafter called the "Study Sponsor"),

WITNESSETH, that

WHEREAS, the Public Works Committee of the United States Senate adopted a resolution on 14 April 1967 that authorized the Corps of Engineers to conduct a study to determine the feasibility of providing improvements for flood control and other water related land resource needs including water supply, water quality control, hydropower, recreation, and fish and wildlife; and

WHEREAS, the Corps of Engineers has conducted a reconnaissance phase study of flood problems and other water related land resource problems and potential solutions pursuant to this authority, and has determined that further study in the nature of a "Feasibility Phase Study" is required to fulfill the intent of the study authority and to determine the extent of the Federal interest in solution to flood problems and other water related land resources problems; and

WHEREAS, the Study Sponsor considers it in its best interest to have the "Feasibility Phase Study" promptly completed, and is willing to

cost-share up to 50% of the total costs of the "Feasibility Phase Study" as indicated in Article I.

WHEREAS, the Secretary of the Army may determine that it is necessary for cost-sharing by non-Federal interests on the authorized "Feasibiiity Phase Study."

NOW THEREFORE, the parties agree as follows:

ARTICLE I - CONSIDERATION AND PAYMENT

a. Should the Secretary of the Army determine that cost-sharing is necessary, the Study Sponsor shall pay, as further specified in this agreement, 50% of the total costs of the Feasibility Phase Study for any or all of the following items:

Basinwide Protection

Comite River Diversion

East Baton Rouge Parish

Monte Sano Bayou
Jones Creek
Bayou Fountain
Clay Cut Bayou
Hurricane Creek
Nonstructural measures

Livingston Parish

Grays Creek and Tributaries
Effects of roads and bridges on
Restricting Flood Flows
Nonstructural measures

Ascension Parish

New River, Black Bayou, Bayou Narcisse,
and Bayou Francois
Sorrento
Nonstructural measures

The term "total costs of the feasibility phase study" means:

(1) For any Feasibility Phase Study that is in progress at the time the Secretary of the Army decides to proceed with cost-sharing as provided in this agreement, all actual costs of the Feasibility Phase Study that are incurred by the Government or the Study Sponsor pursuant to the terms of this agreement commencing 60 days after the date the decision to proceed with cost-sharing is made, including supervisory and administrative costs.

(2) For any Feasibility Phase Study that is not in progress at the time the Secretary of the Army decides to proceed with cost-sharing as provided in this agreement, all actual costs of the Feasibility Phase Study incurred by the Government or the Study Sponsor pursuant to the terms of this agreement, including supervisory and administrative costs.

b. A Feasibility Phase Study shall be deemed to be complete for cost-sharing purposes with concurrence of the Study Sponsor when the Division Commander issues his notice of completion of the Feasibility Report.

c. The Government, using funds contributed by the Study Sponsor and appropriated by the Congress, shall expeditiously make and complete the Feasibility Phase Study, applying those procedures usually followed or applied pursuant to Federal laws, regulations and policies. The Government will consult with the Study Sponsor in determining the advisability and scope of work to be performed by contract. All bid proposals will be reviewed by the Study Sponsor at their option and comments will be provided to the Government for use in selection of contractors. Award of any contracts with appropriated funds or Study Sponsor contributed funds shall be exclusively within the control of the Government. Concurrent review of work performed by a contractor by the Government and Study Sponsor will be performed in accordance with the contract provisions.

d. The Study Sponsor makes such contributions with the clear understanding that it does not obligate the Government to either recommend authorization of or undertake the construction of a Federal project upon completion of the Feasibility Phase Study. The Study Sponsor shall not have any recourse for payment or reimbursement of any nature whatsoever from the Government for contributions tendered pursuant to the terms of this agreement (except with respect to excess cash contributions as set forth in Article II.e.).

ARTICLE II - METHOD OF PAYMENT

a. To provide for consistent and effective communication between the Study Sponsor and the Government during the term of study, the Study Sponsor and the Government will appoint representatives to coordinate on scheduling, planning, and other matters relating to the Feasibility Phase Study.

b. The Government will notify the Study Sponsor of the decision to proceed with cost-sharing as provided in this agreement within seven

working days of the date the decision to proceed with cost-sharing is made.

(1) In the event the Feasibility Phase Study is in progress at the time this decision is made, the Government will, in addition to notifying the Study Sponsor of that decision, also provide the Study Sponsor with an estimate of the Study Sponsor share of Feasibility Phase Study Costs for the remainder of the Government Fiscal Year. The Study Sponsor will then satisfy its obligation to provide 50% of the total "Feasibility Phase Study" costs for the remainder of the Government Fiscal Year by providing within 60 working days of notice by the Government cash contributions, and/or in-kind services during the course of the remainder of the Government Fiscal Year, equal in amount to the Study Sponsor share of Feasibility Phase Study costs for the remainder of the Government Fiscal Year.

(2) In the event the Feasibility Phase Study is not in progress at the time the decision to proceed with cost-sharing is made, the Government will, at the time it provides notice of the decision to proceed with cost-sharing, also provide the Study Sponsor with a date on which the Government expects to initiate the Feasibility Phase Study and an estimate of the Study Sponsor share of the Feasibility Phase Study costs for the Government Fiscal Year in which the Feasibility Phase Study is to be initiated. The Government will obtain the Study Sponsor Share of the total costs of the Feasibility Phase Study from the Study Sponsor for the Government Fiscal Year in which the study is to be initiated, prior to initiating the Feasibility Phase Study. The estimated Study Sponsor's share of the Feasibility Phase Study costs for the Government Fiscal Year will be limited to 50 percent of the estimated costs for the Feasibility Phase Study for the Government Fiscal Year.

c. For subsequent Government Fiscal Years; the Government will provide the Study Sponsor with a statement of the Study Sponsor's share of the total costs of the Feasibility Phase Study for that Government Fiscal

Year six months before the start of the Government Fiscal Year. The Study Sponsor will provide its share of the total costs of the Feasibility Phase Study for that Government Fiscal Year by providing at the start of the Government Fiscal Year cash contributions, and/or in-kind services during the course of the Government Fiscal Year, equal to the Study Sponsor share. For each subsequent Government Fiscal Year, the estimate of the Study Sponsor's share for the pending Fiscal Year will be limited to 50 percent of the estimated costs of the Study in the pending Government Fiscal Year.

d. The Study Sponsor shall make any cash contributions required under this agreement available to the Government by either cash payments to the Government or by deposits of cash into an escrow account acceptable to the Government. The Government will draw on such contributions as it deems necessary to cover contractual and in-house obligations as they occur.

e. Upon completion of the Feasibility Phase Study and resolution of any contract claims and appeals, the Government will compute the total costs of the Feasibiltiy Phase Study and tender to the Study Sponsor a final accounting of its share of the study costs. In the event the Study Sponsor has paid less than its share of the total Feasibility Phase Study costs at the time of the accounting, the Study Sponsor agrees to pay the Government within 90 calendar days after receipt of written notice, whatever sum is required to meet the Study Sponsor share of study costs, provided the Study Sponsor received prior written notice that additional funds would be necessary to complete the Study and the Study Sponsor concurred. In the event the Study Sponsor paid more than its share of total Feasibility Phase Study costs at the time of the final accounting, the Government will return to the Study Sponsor within 90 calendar days the excess cash contribution.

f. HOWEVER, where the Study Sponsor is the State itself, this agreement does not obligate future legislative appropriations or other funds for such performance and payment when obligating future

appropriations or other funds would be inconsistent with State constitutional limitations.

ARTICLE III - CREDIT FOR IN-KIND SERVICES

When approved by the Government, the Study Sponsor may receive a credit for in-kind services provided in connection with accomplishment of the Feasibility Phase Study. The credit shall be applied against the requirement established in Article Ia to pay 50% of the cost of the Feasibility Phase Study, but shall not exceed 50% of the total Study Sponsor requirement for Feasibility Phase Study costs. The procedures and methods for computing the value of in-kind services shall be agreed to by the Government and the Study Sponsor prior to the provision of any such services.

ARTICLE IV - TERMINATION

This agreement may be terminated by either party within thirty days by providing written notice to that effect. There shall be a final accounting and settlement upon termination, with all costs including costs incurred as a result of the termination to be cost shared on a 50-50 basis as is otherwise provided under the terms of this agreement.

ARTICLE V - DISPUTES

Any dispute arising under this agreement which is not disposed of by mutual consent shall be decided by the Contracting Officer who shall reduce his decision to writing and mail or otherwise furnish a copy to the

Study Sponsor. The decision of the Contracting Officer shall be final and conclusive unless, within 30 days from the receipt of such copy, the Study Sponsor mails or otherwise furnishes to the Contracting Officer a written appeal addressed to the Corps of Engineers Board of Contract Appeals. The decision of the Board shall be final and conclusive. Pending final decision of a dispute hereunder, the Study Sponsor shall proceed diligently with the performance of the agreement in accordance with the Contracting Officer's decision.

ARTICLE VI - MAINTENANCE OF RECORDS

The Government and the Study Sponsor shall keep books, records, documents and other evidence pertaining to costs and expenses incurred pursuant to this agreement to the extent and in such detail as will properly reflect total Feasibility Phase Costs. The Government and the Study Sponsor shall maintain such books, records, documents and other evidence for inspection and audit by authorized representatives of the parties to this agreement.

ARTICLE VII - RELATIONSHIP OF PARTIES

The parties to this agreement act in an independent capacity in the performance of their respective functions under this agreement, and neither party is to be considered the officer, agent, or employee of the other.

ARTICLE VIII - OFFICIALS NOT TO BENEFIT

No member of or delegate to the Congress, or other elected official, shall be admitted to any share or part of this agreement, or to any benefit that may arise therefrom.

ARTICLE IX - COVENANT AGAINST CONTINGENT FEES

The Study Sponsor warrants that no person or selling agency has been employed or retained to solicit or secure this agreement upon agreement or understanding for a commission, percentage, brokerage, or contingent fee, excepting bona fide employees or bona fide established commercial or selling agencies maintained by the Study Sponsor for the purpose of securing business. For breach or violation of this warranty, the Government shall have the right to annul this agreement without liability or in its discretion to add to the agreement or consideration, or otherwise recover, the full amount of such commission, percentage, brokerage, or contingent fee.

IN WITNESS WHEREOF, the parties hereto have executed this agreement as of the day and year first above written.

THE UNITED STATES OF AMERICA

BY _____

Colonel Corps of Engineers
District Engineer
Contracting Officer

WITNESSES:

STATE OF LOUISIANA

_____

_____

BY _____

Secretary
Department of Transportation
and Development

CERTIFICATE OF AUTHORITY

I, Norman L. Sisson do hereby certify that I am the General Counsel of the Department of Transportation and Development, State of Louisiana, that the State of Louisiana is a legally constituted public body with full authority and legal capability to perform the terms of the agreement between the United States of America and the State of Louisiana in connection with the Amite River and Tributaries Study, and with full authority and legal capability to pay judgments for damages through the Legislature of the State of Louisiana, if necessary, in the event of the failure to perform, and that the persons who have executed the agreement on behalf of the State of Louisiana have acted within their statutory authority.

IN WITNESS WHEREOF, I have made and executed this certificate this 5th day of February, A.D., 19 85.

Norman L. Sisson
General Counsel
DOTD, State of Louisiana

FIRST SUPPLEMENT TO THE
AGREEMENT BETWEEN

THE UNITED STATES OF AMERICA
AND
THE STATE OF LOUISIANA

FOR THE AMITE RIVER AND TRIBUTARIES STUDY

THIS SUPPLEMENTAL AGREEMENT is entered into this_____ day of _____ 1990, by and between the United States of America (hereinafter called the "Government") represented by the Contracting Officer executing this Supplemental Agreement and the State of Louisiana through the Louisiana Department of Transportation and Development (hereinafter called the "Study Sponsor"), represented by its Secretary,

WITNESSETH THAT:

WHEREAS, on 5th day of February 1985, the Government and the Study Sponsor entered into a Cost-Sharing Agreement for the Amite River and Tributaries Study (hereinafter called the "Original Agreement"); and

WHEREAS, the Study Sponsor has requested that the Original Agreement be modified to include the Darlington Reservoir; and

WHEREAS, the Study Sponsor is currently cost-sharing the Amite River and Tributaries Study on a 50-50 basis; and

WHEREAS, the Study Sponsor is willing to cost-share on a 50-50 basis the increased cost of this modification to include the Darlington Reservoir in the Amite River and Tributaries Study.

NOW THEREFORE, the parties agree as follows:

The Original Agreement for the Amite River and Tributaries Study is hereby modified to include the Darlington Reservoir.

The Secretary of the Army has determined that cost-sharing is necessary, therefore the Study Sponsor shall pay, as is specified in this Supplemental Agreement, 50% of the total costs of the Feasibility Phase Study for the Darlington Reservoir Study.

All the Articles of the Original Agreement, not specifically changed or modified herein, shall remain in full force and effect, and apply to this Supplemental Agreement as if they were incorporated herein and made a part hereof.

IN WITNESS WHEREOF, the parties hereto have executed this Supplemental Agreement as of the day and year first above written.

UNITED STATES OF AMERICA

BY _____
Colonel Corps of Engineers
District Engineer
Contracting Officer

WITNESS:

_____

_____

STATE OF LOUISIANA

BY _____
Secretary
Louisiana Department of
Transportation and Development

APPENDIX B

ESPEY,
HUSTON &
ASSOCIATES, INC.
Engineering & Environmental Consultants

Document No. 890360
EH&A Job No. 10943

ENGINEERING REPORT
AMITE RIVER FLOOD CONTROL STUDY
STATE PROJECT NUMBER 575-99-30

Prepared for:

Louisiana Department of Transportation and Development
1201 Capital Access Road
Baton Rouge, LA 70802

and

Amite River Basin Drainage
and Water Conservation District

Prepared by:

Espey, Huston & Associates, Inc.
3900 N. Causeway Boulevard, Suite 730
Metairie, Louisiana 70007

September 1989

For this Engineering Report, the level of detail has been increased for the three remaining flood-control alternatives. Each remaining alternative has been subjected to a variety of flood events across the flood-frequency spectrum in order to confirm that the results of the Secondary Screening, Phase 2, which were based on one single flood event (the 50-year), could be extrapolated to other events and that one alternative was not particularly favored over another by the utilization of the 50-year flood event in the Secondary Screening.

4.3.3 Screening of Alternative Plans

4.3.3.1 Initial Screening

Initially, 33 potential alternative solutions, including the Proposed Darlington Reservoir, were identified to meet the project's objectives. These alternatives were developed after reviewing the public meeting documents of the USCE, the early work by consultants under contract to the LDOTD, and requested comments and suggestions from a number of State and Federal agencies. These alternatives, along with the No Action Alternative, all of which are briefly described in tables 4.3-1, 4.3-2 and 4.3-3, were grouped by similar physical features in the Initial Screening Report and evaluated according to the technical, sociological, and environmental disciplines addressed in the Environmental Report.

The Initial Screening process utilized existing data including: previous USCE reports; prior LDOTD studies; environmental and hydrological data from field observations at the proposed Darlington site; baseline environmental and sociological studies throughout the basin; and knowledge gained through the hydrologic and hydraulic model calibration efforts. The purposes of the Initial Screening effort were: 1) to provide a qualitative assessment as to each alternative's ability to meet the project's flood-control and water-based recreational objectives; 2) to identify alternatives which may have substantial environmental impacts that could not be mitigated, 3) to identify alternatives which appear to have unacceptable costs relative to the benefits provided, and 4) to identify which alternatives should be deleted from further consideration.

TABLE 4.3-1

AMITE RIVER ALTERNATIVES
DELETED IN THE
INITIAL SCREENING PROCESS

Description	Recreation Pool Data			Flood Pool Data			Flood Storage (ac-ft)	Downstream Improvements					
	Elev. (ft)	Volume (ac-ft)	Area (ac)	Elev. (ft)	Volume (ac-ft)	Area (ac)		Channel-ization	Ring Levees	Elevate Struct.	Reloc. Struct.	Comite Divers	Bank Levees
R-1B Single Dam & Wet Reservoir	181.0	360,000	17,800	194.2	624,800	25,200	264,800	-	-	-	-	-	-
R-2A Single Dam & Wet Reservoir	140.0	9,400	1,800	164.5	141,800	9,800	132,400	P	-	-	-	-	-
R-2B Single Dam & Wet Reservoir	140.0	9,400	1,800	164.5	141,800	9,800	132,400	P	X	-	-	-	-
R-2D Single Dam & Wet Reservoir	140.0	9,400	1,800	164.5	141,800	9,800	132,400	-	-	-	-	X	-
R-3B Single Dam & Dry Reservoir	-	-	-	163.6	132,400	9,400	132,400	P	-	-	-	-	-
R-3C Single Dam & Dry Reservoir	-	-	-	163.6	132,400	9,400	132,400	P	X	-	-	-	-
R-3E Single Dam & Dry Reservoir	-	-	-	163.6	132,400	9,400	132,400	-	-	-	-	X	-
R-4B Multiple Wet Reservoirs	Var.	Var.	Var.	Var.	132,400	Var.	132,400	P	-	-	-	-	-
R-4C Multiple Wet Reservoirs	Var.	Var.	Var.	Var.	132,400	Var.	132,400	P	X	-	-	-	-
R-4E Multiple Wet Reservoirs	Var.	Var.	Var.	Var.	132,400	Var.	132,400	-	-	-	-	X	-
R-5B Multiple Dry Reservoirs	-	-	-	Var.	132,400*	Var.	132,400	P	-	-	-	-	-
R-5C Multiple Dry Reservoirs	-	-	-	Var.	132,400*	Var.	132,400*	P	X	-	-	-	-
R-5E Multiple Dry Reservoirs	-	-	-	Var.	132,400*	Var.	132,400*	-	-	-	-	X	-
NR-2A Amite Channelization	-	-	-	-	-	-	-	X	-	-	-	-	-
NR-2B Amite Bank Levees	-	-	-	-	-	-	-	-	-	-	-	-	X
NR-2C Amite Channel. & Bank Levees	-	-	-	-	-	-	-	P	-	-	-	-	X
NR-2D Amite Channel. & Ring Levees	-	-	-	-	-	-	-	P	X	-	-	-	-
NR-2E Amite Channel Ring Levees & Comite Diversion	-	-	-	-	-	-	-	P	X	-	-	X	-
NR-4A Channel., Elev. & Reloc. Struct.	-	-	-	-	-	-	-	P	-	X	X	-	-
NR-4B Channel., Ring Levees, Elev. & Reloc. Structures	-	-	-	-	-	-	-	P	X	X	X	-	-
NR-4C Channel., Comite Divers., Elev. & Reloc. Struct.	-	-	-	-	-	-	-	P	-	X	X	X	-

* Equivalent storage to this volume of single reservoir storage.

P = Partial

Note: Elevation and storage data refined in subsequent analyses.

TABLE 4.3-2

AMITE RIVER ALTERNATIVES

DELETED IN THE

SECONDARY SCREENING PROCESS

	Description	Recreation Pool Data			Flood Pool Data			Flood Storage (ac-ft)	Downstream Improvements				Alternative Eliminated In
		Elev. (ft)	Volume (ac-ft)	Area (ac)	Elev. (ft)	Volume (ac-ft)	Area (ac)		Channel-ization	Ring Levees	Elevate Struct.	Reloc. Struct.	
R-1C	Single Dam & Wet Reservoir	140.0	9,400	1,800	175.8	274,200	15,000	264,800	-	-	-	-	Phase 2
R-2C	Single Dam & Wet Reservoir	140.0	9,400	1,800	164.5	141,800	9,800	132,400	-	X	X	X	Phase 2
R-4A	Multiple Wet Reservoirs	Var.	Var.	Var.	Var.	264,800	Var.	264,800	-	-	-	-	Phase 1
R-4D	Multiple Wet Reservoirs	Var.	Var.	Var.	Var.	132,400	Var.	132,400	-	X	X	X	Phase 1
R-5A	Multiple Dry Reservoirs	-	-	-	Var.	264,800*	Var.	264,800*	-	-	-	-	Phase 1
R-5D	Multiple Dry Reservoirs	-	-	-	Var.	132,400*	Var.	132,400*	-	X	X	X	Phase 1
NR-3A	Elevating Structures	-	-	-	-	-	-	-	-	-	X	-	Phase 2
NR-3B	Relocating Structures	-	-	-	-	-	-	-	-	-	-	X	Phase 2
NR-4D	Ring Levees & Stringent Regulations	-	-	-	-	-	-	-	-	X	-	-	Phase 2

* Equivalent storage to this volume of single reservoir storage.

Note: Elevation and storage data refined in subsequent analyses.

TABLE 4.3-3

AMITE RIVER ALTERNATIVES
RECOMMENDED FOR FURTHER ANALYSIS
IN THE
ENGINEERING REPORT

	Description	Recreation Pool Data			Flood Pool Data			Flood Storage (ac-ft)	Downstream Improvements		
		Elev. (ft)	Volume (ac-ft)	Area* (ac)	Elev. (ft)	Volume (ac-ft)	Area* (ac)		Ring Levees	Elevate Struct.	Reloc. Struct.
R-1A	Single Dam & Wet Reservoir	171.0	210,000	11,900	186.5	460,000	20,000	250,000	--	--	--
R-3A	Single Dam & Dry Reservoir	--	--	--	174.5	255,000	13,600	255,000	--	--	--
R-3D	Single Dam & Dry Reservoir	--	--	--	164.3	141,800	9,850	141,800	X	X	X
NR-1	No Action	--	--	--	--	--	--	--	--	--	--

(*) Note: Flooded acreages do not include the 150± ac of area under the dam embankment that are included in environmental impact discussions.

During the Initial Screening process, the project hydrologists estimated that the multiple small reservoir alternatives would be ineffective from a flood-control standpoint and should not be considered further; however, because the multiple-reservoir concept has the interest of some citizens, it was decided that the alternatives containing multiple reservoirs should be carried into the Secondary Screening effort where computer modeling would confirm or refute the hydrologists' recommendations. Based on the results of the Initial Screening, the 21 alternatives identified in Table 4.3-1 were recommended for deletion from further study for various technical and environmental reasons. The rationale for each deletion is detailed in the Initial Screening Report (Appendix A) and will not be repeated here.

4.3.3.2 Secondary Screening, Phase 1

In order to accommodate the need to examine certain initial screening alternatives in greater detail, the study team divided the Secondary Screening effort into two phases. The sole function of Phase 1 of the Secondary Screening was the verification of the technical validity of the multiple-reservoir concept. In Phase 1, potential multiple-reservoir sites were located within the Louisiana portion of the Amite River Basin, their physical storage characteristics were determined, and mathematical modeling of the reservoirs' effects on downstream flood flows was accomplished with the USCE's HEC-1 hydrologic model which was designed to simulate the surface runoff response of a river basin to precipitation. A detailed description of each multiple-reservoir alternative is included in Appendix A of the Phase 1 screening report and will not be repeated here.

Computerized modeling of the multiple reservoirs verified their ineffectiveness. The results indicated that construction of the 22 small reservoirs would reduce peak flows at Denham Springs by only 10%, making the multiple-reservoir concept a non-viable flood-control option. The four multiple-reservoir options which were recommended for deletion from further screening in Phase 1 of the Secondary Screening, along with a summary of the physical characteristics of each, are shown in Table 4.3-2. The Secondary Screening, Phase 1, report is included as Appendix B to this report.

4.3.3.3 Secondary Screening, Phase 2

The remaining nine alternatives recommended for further evaluation in Phase 2 of the Secondary Screening included: the proposed permit activity, Alternative R-1A, which consists of a single dam and reservoir with a normal pool at an elevation of 171.0 ft; Alternatives R-1C and R-2C which have a normal pool at an elevation of 140.0 ft; Alternatives R-3A and R-3D which consist of a single dam and dry reservoir; and four non-reservoir alternatives including the No Action Alternative and alternatives which achieve the project objectives by elevating (Alternative NR-3A) or relocating (Alternative NR-3B) flood-prone structures or protecting them by the use of ring levees (Alternative NR-4D).

For the Secondary Screening, Phase 2, each alternative containing a reservoir was "designed" to achieve the project's flood-control objectives. Through an iterative process of examining the effectiveness and cost of various spillway and service outlet combinations, a preliminary design of each dam and outlet works was derived. From that preliminary design, the structural and geotechnical recommendations and estimated costs were developed. Although the design storm for the reservoirs and all other alternatives was selected to be the 50-year event, safety considerations in the design of the reservoir dictated a conservatism that was not imposed on the non-reservoir alternatives. Therefore, although the entire screening process has been designed to ensure an "equity of detail" between alternatives, the need to design the reservoir options to safely pass the Probable Maximum Flood (PMF) subjects them to a far greater conservatism than the non-reservoir or non-structural alternatives.

While all reservoir alternatives were designed to provide flood-control protection, only the reservoirs providing a permanent conservation pool offered the potential for low-flow augmentation, maintainance of adequate water quality downstream of East Baton Rouge discharges, and the minimization of impacts related to saltwater intrusion during periods of prolonged dry weather, as well as a water source which could eventually be used for domestic water supply and power generation.

During the project formulation phase of the Darlington Reservoir studies, the primary objective in the basin was identified as flood control, with a secondary objective of providing water-based recreation to the area. Of the alternatives remaining in the screening process at the Secondary Screening, Phase 2, only the proposed permit activity, Alternative R-1A, offered meaningful surface area for a recreational reservoir. Alternatives R-1C and R-2C survived in the screening process to the Secondary Screening because of the possibility that a reduced-size recreation pool could provide water-based recreation with less environmental impact. Based on the work performed in the Secondary Screening and the recommendations of State and private recreational consultants, a reduced-size recreational pool was determined not to be viable. Based on a review of contour maps, it would appear that the average depth of the reduced-size recreational pool reservoir alternatives would appear to be approximately 5.2 ft. It was the concerted opinion of the recreational and lake management specialists that a recreation pool this shallow would offer no significant benefit over a dry reservoir and would have a potential for rapid eutrophication.

Of the non-reservoir alternatives evaluated in the Secondary Screening, only the one which incorporated ring levees and stringent future floodplain regulations was considered to have a practical basis for implementation. It was evident from the data that only 1,700 of the potential 8,600 structures in the Amite River 50-year floodplain have been identified by the USCE as technically (not necessarily financially) capable of being protected by ring levees; however, this alternative does offer the possibility of some form of institutional structure capable of bringing such a solution technique into existence. Whether it be a special taxing district or local county government, it was concluded that an institutional mechanism could be devised to bring about the design, construction, maintenance, and payment for the required improvements. This practical fact proves to be a great advantage over other non-reservoir alternatives such as the elevating or relocating of structures.

In the Secondary Screening, it was determined that the stand-alone alternative of relocating flood-prone structures (NR-3B) would be burdened not only by the institutional practicalities of implementing such a concept, but by the fact that the relocated structures would require an infrastructure system (roads, water and sanitary systems, etc.) in the new locations to

support them. Exclusive of land costs and assuming that the displaced residents of the floodplain would resettle into a subdivision with a density of three units to the acre, it was estimated that the new infrastructure cost to support displaced residences and businesses would be approximately $60.2 million. To this development cost, an estimated land cost of approximately $1,000 per lot and a moving cost of $10,000 per structure brought the cost for this alternative to approximately $154.8 million, not including any costs for the compensation of landowners for their current property.

The stand-alone flood-control solution technique of elevating or flood-proofing flood-prone structures in place (NR-3A) which was evaluated in Phase 2 of the Secondary Screening was also determined to be burdened by the need for an institutional system of voluntary and involuntary implementation of the technique to all properties subject to flooding; however, because the elevating or flood-proofing technique is applied to the structures in place, without displacement from their present locations, it was concluded that NR-3A would be met with less public opposition and difficulty than the relocation alternative (NR-3B).

It was concluded in the Secondary Screening, Phase 2, report that the practical limitations to the implementation of either Alternatives NR-3A, NR-3B, or NR-4D as "stand-alone" solution techniques were sufficient to cause these alternatives to be deleted from further study in the screening process. It was concluded, however, that the techniques could be extremely viable when considered as part of a more comprehensive system of flood control. It was for that reason that these solution techniques were recommended to be carried forward into the Final Screening as part of Alternative R-3D, a reduced-size dry reservoir coupled with the elevating, relocating, and leveeing of flood-prone structures, where appropriate.

The five alternatives which were recommended for deletion from further screening in Phase 2 of the Secondary Screening include Alternative R-1C - the Reduced-size Recreational Pool Single Dam and Wet Reservoir, Alternative R-2C - the Reduced-size Recreational and Flood Control Pool Single Dam and Wet Reservoir with downstream improvements, Alternatives NR-3A and NR-3B - the Elevating and Relocating of Flood-Prone Structures, and Alternative NR-4D - the Construction of Ring Levees and the Implementation of Stringent Flood Control

Regulations. These alternatives are shown in Table 4.3-2 along with a summary of their physical characteristics. The alternatives which were recommended for further analysis in the Final Engineering Report are summarized in Table 4.3-3.

The differences between reservoir surface acreages and flood-storage amounts seen in tables 4.3-2 and 4.3-3 are a result of the refinement of data and the finalization of the design details of the reservoir options in the Phase 2 screening effort. In the prior screening reports, data as to the reservoir pool elevations in feet, surface areas in acres, and volumes of storage in acre-feet were taken from previous engineering studies. As the environmental assessment work proceeded for Phase 2, the acreages of various land uses and cover were digitized and totals were developed for each alternative reservoir recreational pool and design surcharge. The Final Screening assumptions are based on the new totals due to the higher degree of accuracy associated with digitization.

A second minor difference between tables 4.3-2 and 4.3-3 lies in the maximum flood pool and flood-storage data. As the final preliminary designs for each reservoir were developed, the optimum combinations of reservoir outlets resulted in more accurate definitions of the flood pool and maximum flood-storage volumes. These refined data are also utilized throughout the remainder of the screening process unless further refinements are warranted. In LDOTD's permit application to the USCE, this type of continued refinement in the engineering definition of the project was envisioned and stated.

The Secondary Screening, Phase 2, report is included as Appendix C to this report.

4.3.4 Refinement of Alternative Plans

In the LDOTD's 1984 Conceptual Stage Engineering and Economic Feasibility Report for the Proposed Darlington Reservoir, a multi-purpose reservoir was evaluated that had a maximum normal pool elevation of 180 ft MSL, two 10-ft by 10-ft gate-controlled box culverts and four 40-ft wide by 25-ft high tainter gates on a concrete ogee weir section with a crest at 155 ft MSL. In the LDOTD's January 1987 Hydrologic and Hydraulic Engineering Report, several

spillway and outlet configurations were evaluated. The plan detailed in that evaluation that most closely resembled the proposed Darlington Reservoir permit application data specified a service outlet consisting of six 11-ft by 11-ft low-flow conduits with four 30-ft wide by 22-ft high radial gates on a concrete ogee section with a crest elevation at 165.0 ft MSL. In addition, an emergency spillway was utilized consisting of a 4,000-ft long section cut in the west abutment of the dam. The crest of the emergency spillway was set one foot higher than the greater elevation of the maximum water surface reached by routing either the 100-year or the one-half PMF through the reservoir.

Review of the geotechnical data in the area where the 4,000-ft-long emergency spillway was proposed for location indicated that the soils present in the abutments of the proposed dam site consist of surficial clays underlain by massive sand deposits. An emergency spillway located in this material would suffer serious erosion in the event of the passage of an extreme-event flood hydrograph through the reservoir. In addition, the topography of the west abutment is such as to direct the outflow of the emergency spillway across the downstream face of the embankment. In the absence of significant training works to direct this outflow away from the embankment, severe erosion and possible loss of the structure could occur. For these practical reasons, the current study began to re-examine the outlet works to develop a design that would overcome these limitations.

During the alternative screening process, the 171.0-ft po elevation alternative (R-1A) was initially designed without a separate emergency spillway. All f od discharges were directed over a gated concrete gravity ogee section which incorporated si 40-ft by 40-ft radial tainter gates and a crest elevation on the ogee section set at 147.5 ft. his crest elevation provides sufficient hydraulic head, with the gates fully open, to discharge the attenuated PMF flood within the design constraints previously specified. When routed through the spillway, the PMF peak inflow of 492,300 cubic feet per second (cfs) is reduced to an outflow of 329,900 cfs, reaching a maximum reservoir water surface elevation of approximately 199.8 ft MSL.

Although the above-described six-gate reservoir design results in the least cost of the many configurations, LDOTD was not comfortable with the degree of safety associated with a

design that did not incorporate an uncontrolled emergency spillway. As a result, the consultant team was directed to optimize a design which 1) allowed at least one-half of the PMF to pass over an uncontrolled spillway, 2) allowed mechanically controlled gates to pass no more than approximately one-half the PMF, and 3) would structurally fail if the gates could not be opened during the PMF event (the "heart attack" scenario) and water had to pass over the closed gates. The resultant compromise design includes three 40-ft by 40-ft tainter gates and 1,000 ft of RCC emergency spillway. The emergency spillway in this design is located near the center of the dam where downstream tailwaters will prevent the potential erosion problems associated with the 4,000-ft overflow spillway proposed for the western abutment in the 1984 study.

The six 11-ft by 11-ft gated low-level conduits evaluated in the 1987 LDOTD feasibility design report were replaced in Alternative R-1A with three 8-ft by 10-ft gated box culverts which are adequate to provide the releases stipulated by the design when additional releases are allowed through the tainter gates.

4.4 PRESENTATION AND EVALUATION OF THE FINAL ARRAY OF PLANS

4.4.1 General

Through a comprehensive evaluation of all reasonable and technically feasible alternatives for flood damage reduction on the Amite River described above, three alternative flood-control plans have been carried forward to this final level of evaluation. All three plans remaining in the screening process propose a dam on the Amite River about 5.5 miles southwest of Darlington, Louisiana. Two of the plans are for single-purpose or dry reservoirs; one with full flood-control capacity (R-3A), and one with a reduced flood-control capacity (R-3D) coupled with downstream structural and non-structural improvements to include constructing partial ring levees, and elevating and relocating flood-prone structures.

The plan most resembling the proposed permit request by the LDOTD consists of a multi-purpose reservoir for flood control and water-based recreation with the potential for

APPENDIX C

TABLE 3.9-2

ESTIMATED MARKET VALUE OF LAND AND STRUCTURES AT RESERVOIR ALTERNATIVE R-1A

Item	Number	Unit Value $/sq ft or $/ac	Value Range ($)
Structures[1]	93	$ 20 - 25	$ 2,418,000 - 3,022,500
Camps[2]	34	10 - 15	272,000 - 408,000
Land (in acres)			
Agriculture	4,345	1,000 - 2,000	4,345,000 - 8,690,000
Timber	15,058	1,300 - 2,000	19,575,400 - 30,116,000
Residential	40	2,000 - 2,500	80,000 - 100,000
Sand & Gravel Pits	559	1,000 - 2,000	559,000 - 1,118,000
Other	148	500 - 600	74,000 - 88,800
Total Value			$ 27,323,400 - 43,543,300

1 Based on average of 1,300 square feet per structure (includes houses and 7 non-dwelling type structures).

2 Based on average of 800 square feet per camp.

Source: Structural count obtained from structural survey of reservoir site (conducted by GECI); land use acreages determined by planimetry; property value data obtained from conversations between GECI personnel and parish assessors and real estate agents.

TABLE 3.9-8

ESTIMATED MARKET VALUE OF LAND AND STRUCTURES AT RESERVOIR ALTERNATIVE R-3A

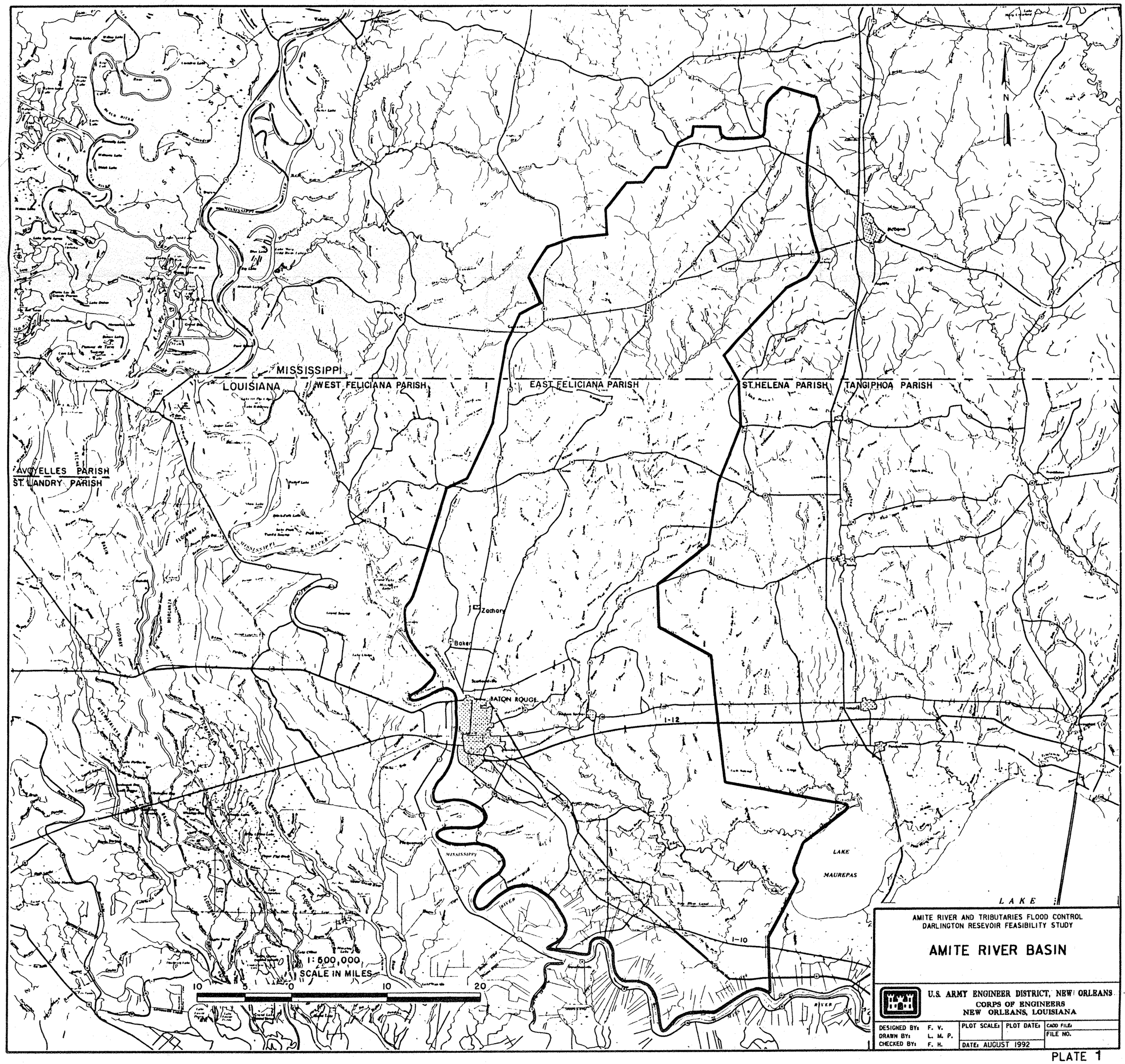
Item	Number	Unit Value $/sq ft or $/ac	Value Range ($)
Structures[1]	60	$ 20 - 25	$ 1,560,000 - 1,950,000
Camps[2]	14	10 - 15	112,000 - 168,000
Land (in acres)			
Agriculture	2,996	1,000 - 2,000	2,996,000 - 5,992,000
Timber	10,149	1,300 - 2,000	13,193,700 - 20,298,000
Residential	36	2,000 - 2,500	72,000 - 90,000
Sand & Gravel Pits	424	1,000 - 2,000	424,000 - 848,000
Other	145	500 - 600	72,500 - 87,000
Total Value			$ 16,758,200 - 27,315,000

[1] Based on average of 1,300 square feet per structure (includes houses and 6 non-dwelling type structures).

[2] Based on average of 800 square feet per camp.

Source: Structural count obtained from structural survey of reservoir site (conducted by GECI); land use acreages determined by planimetry; property value data obtained from conversations between GECI personnel and parish assessors and real estate agents.

PLATES



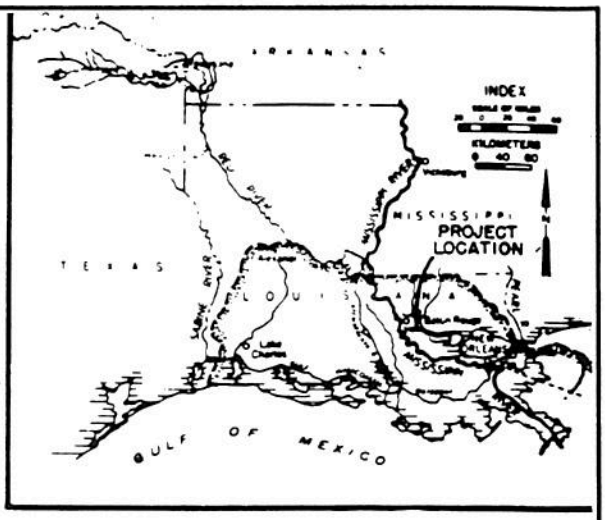
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

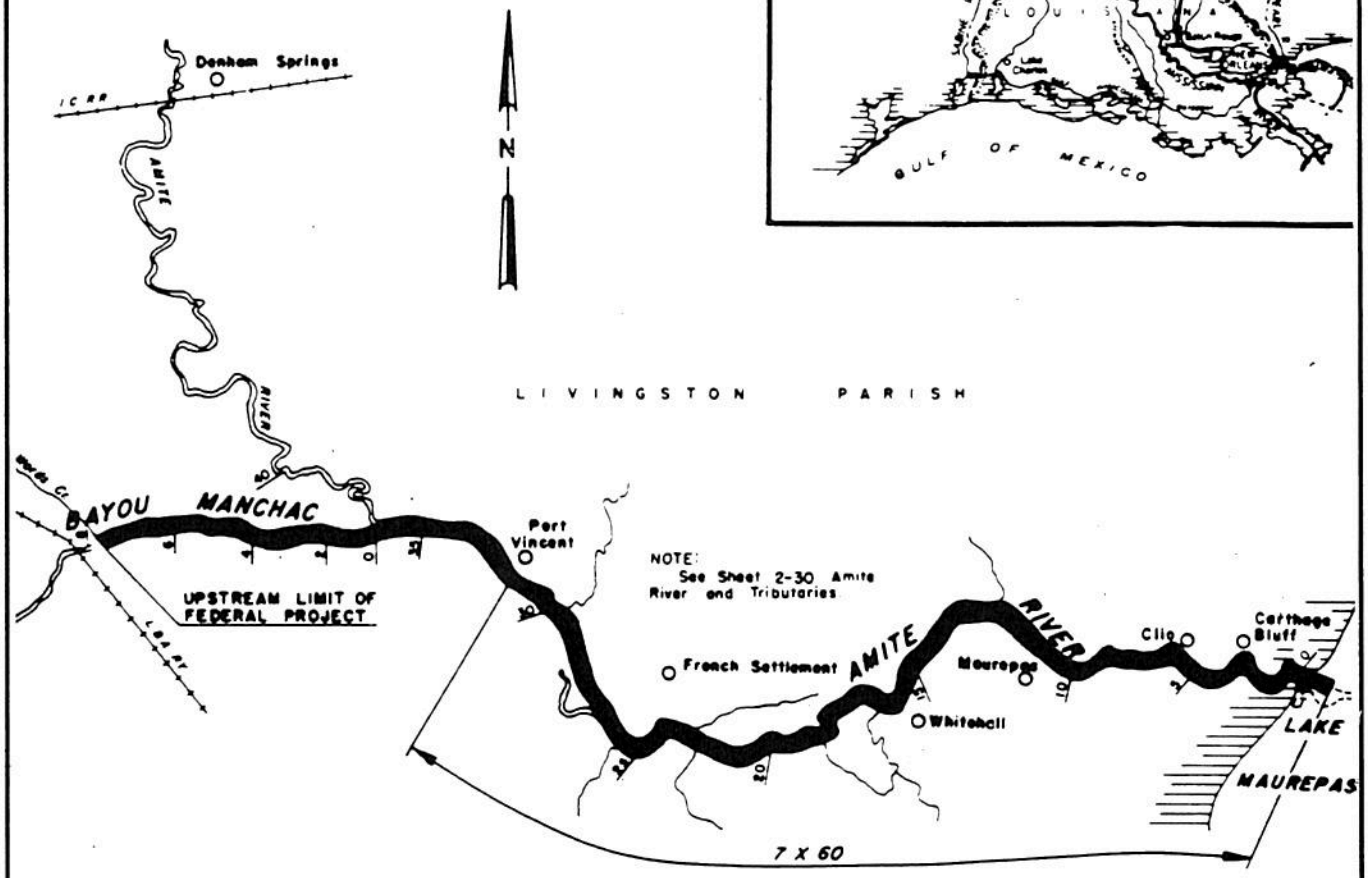
AMITE RIVER BASIN

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

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PLATE 1





PLAN

SCALES

MILES

KILOMETERS

LEGEND

improvements completed

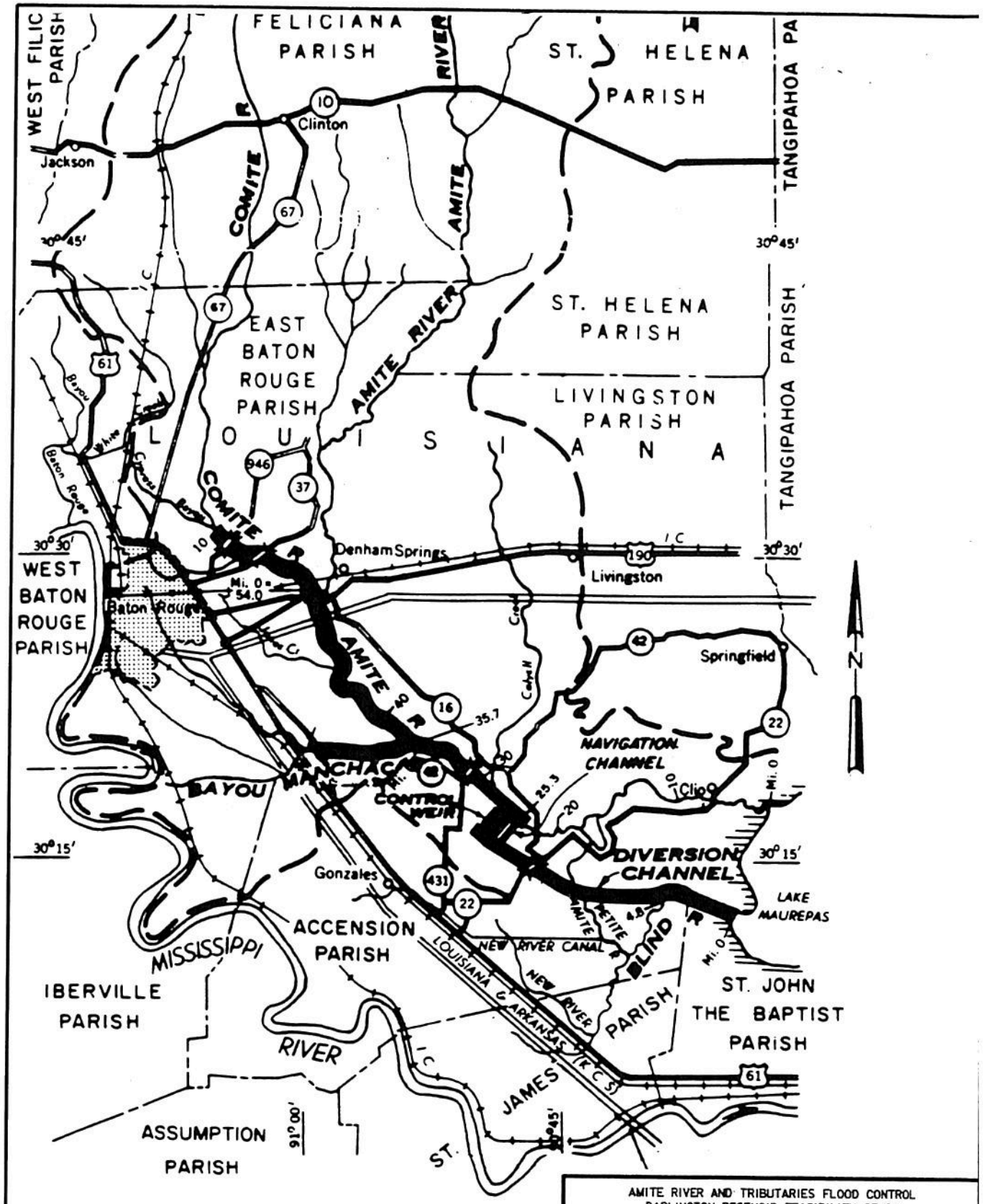
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

AMITE RIVER AND BAYOU MANCHAC NAVIGATION PROJECT

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

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PLATE 2



AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

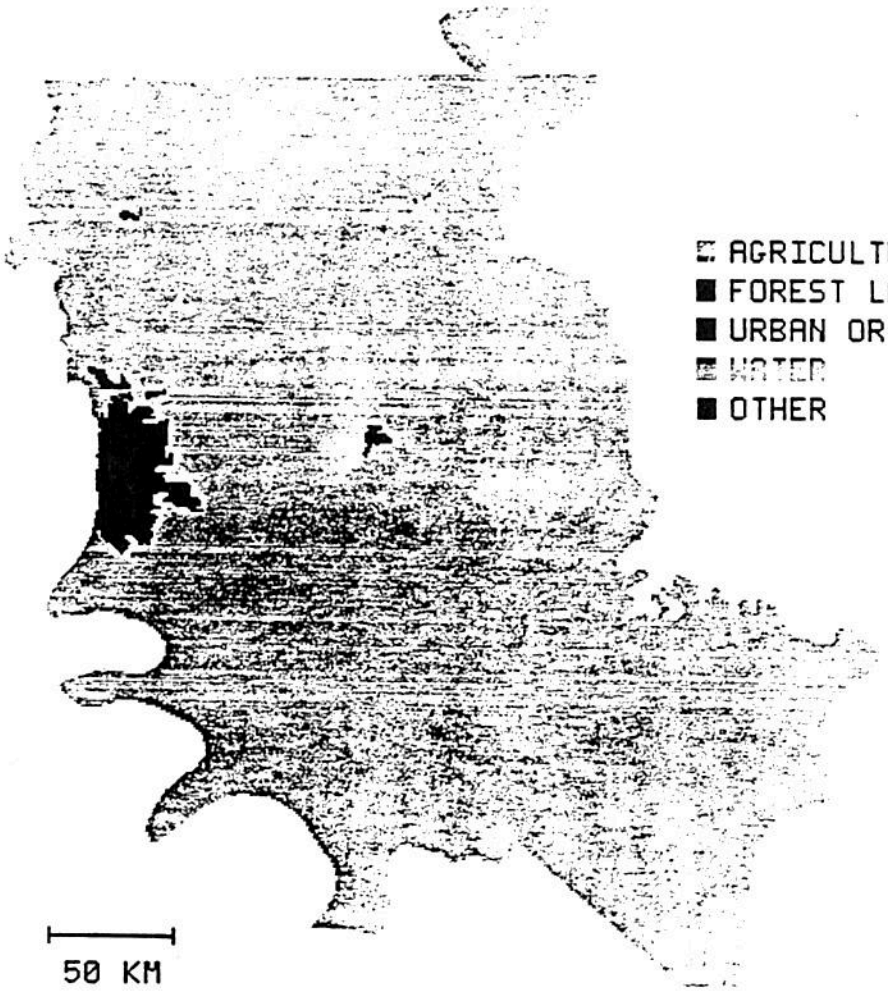
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL PROJECT

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PLATE 3

AMITE RIVER BASIN
1954 LAND USE



AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

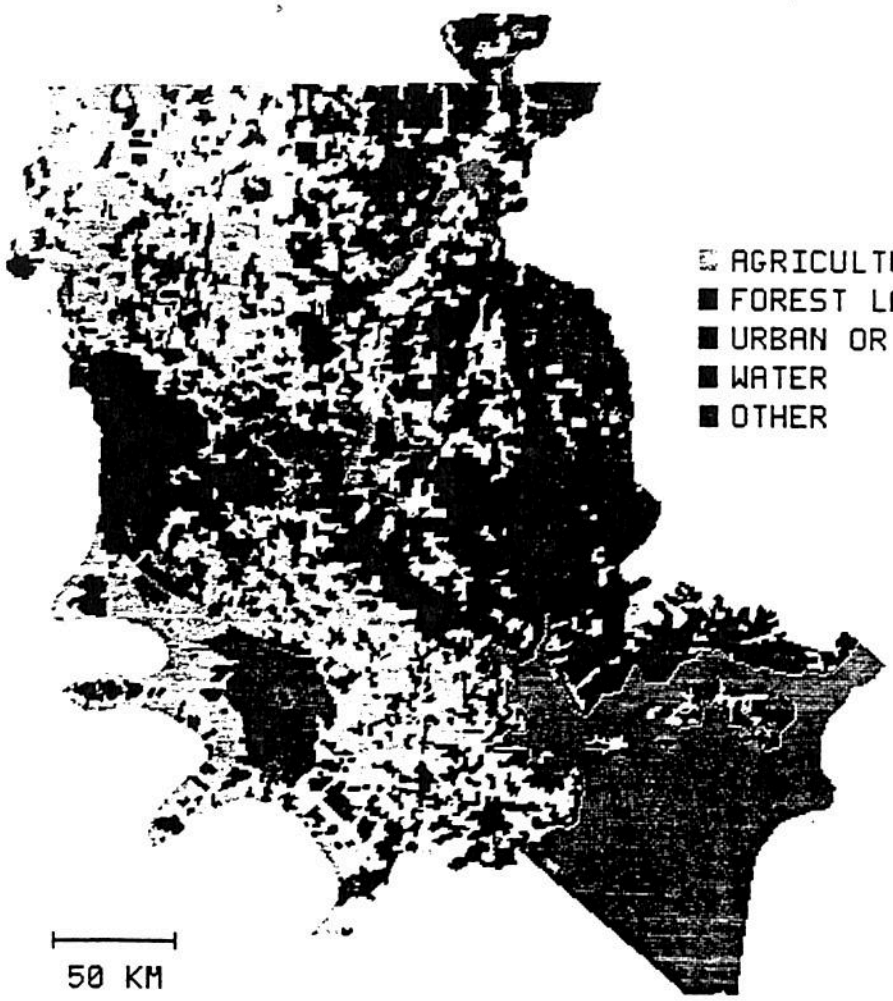
AMITE RIVER BASIN
1954 LAND USE

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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PLATE 4

AMITE RIVER BASIN
1972 LAND USE



AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

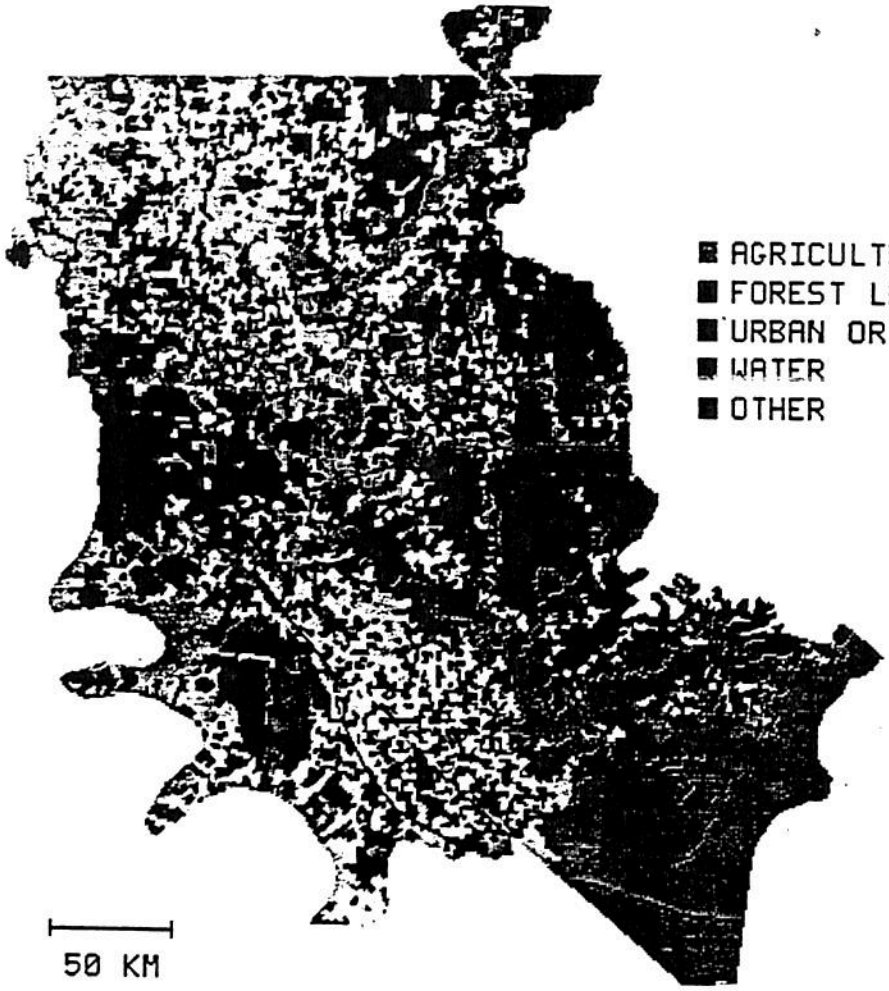
AMITE RIVER BASIN
1972 LAND USE

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

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PLATE 5

AMITE RIVER BASIN
1978 LAND USE



AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

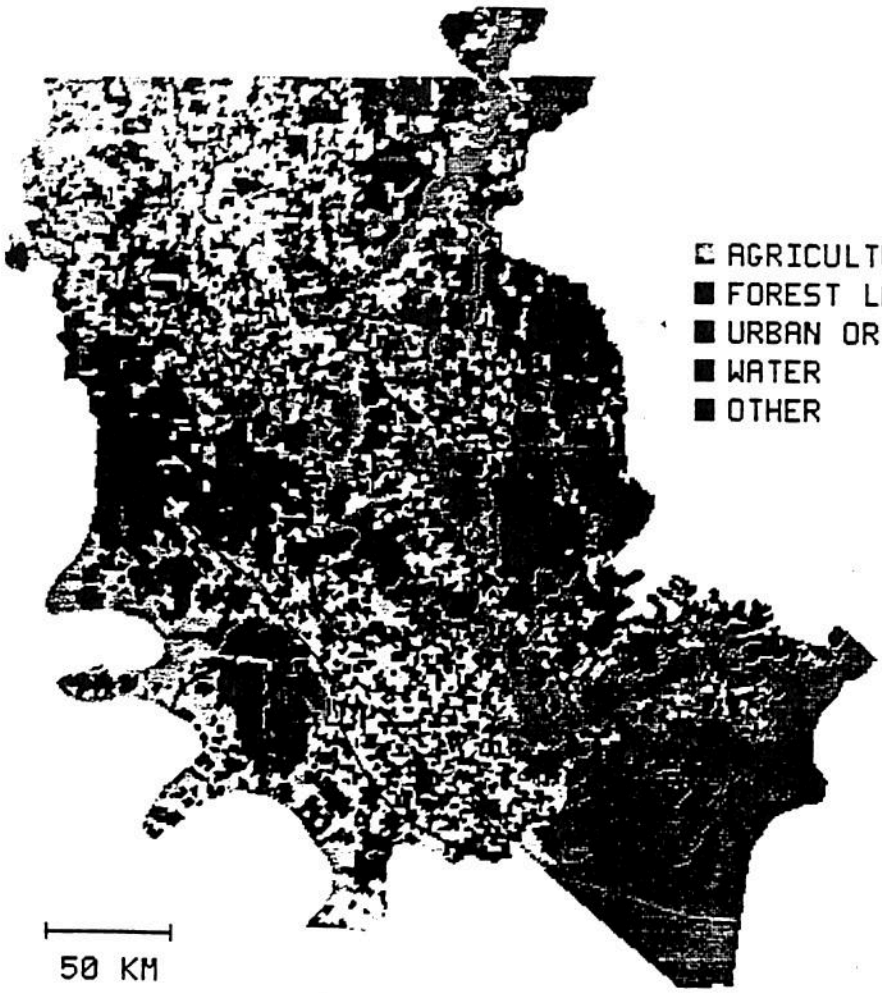
AMITE RIVER BASIN
1978 LAND USE

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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PLATE 6

AMITE RIVER BASIN
1985 LAND USE



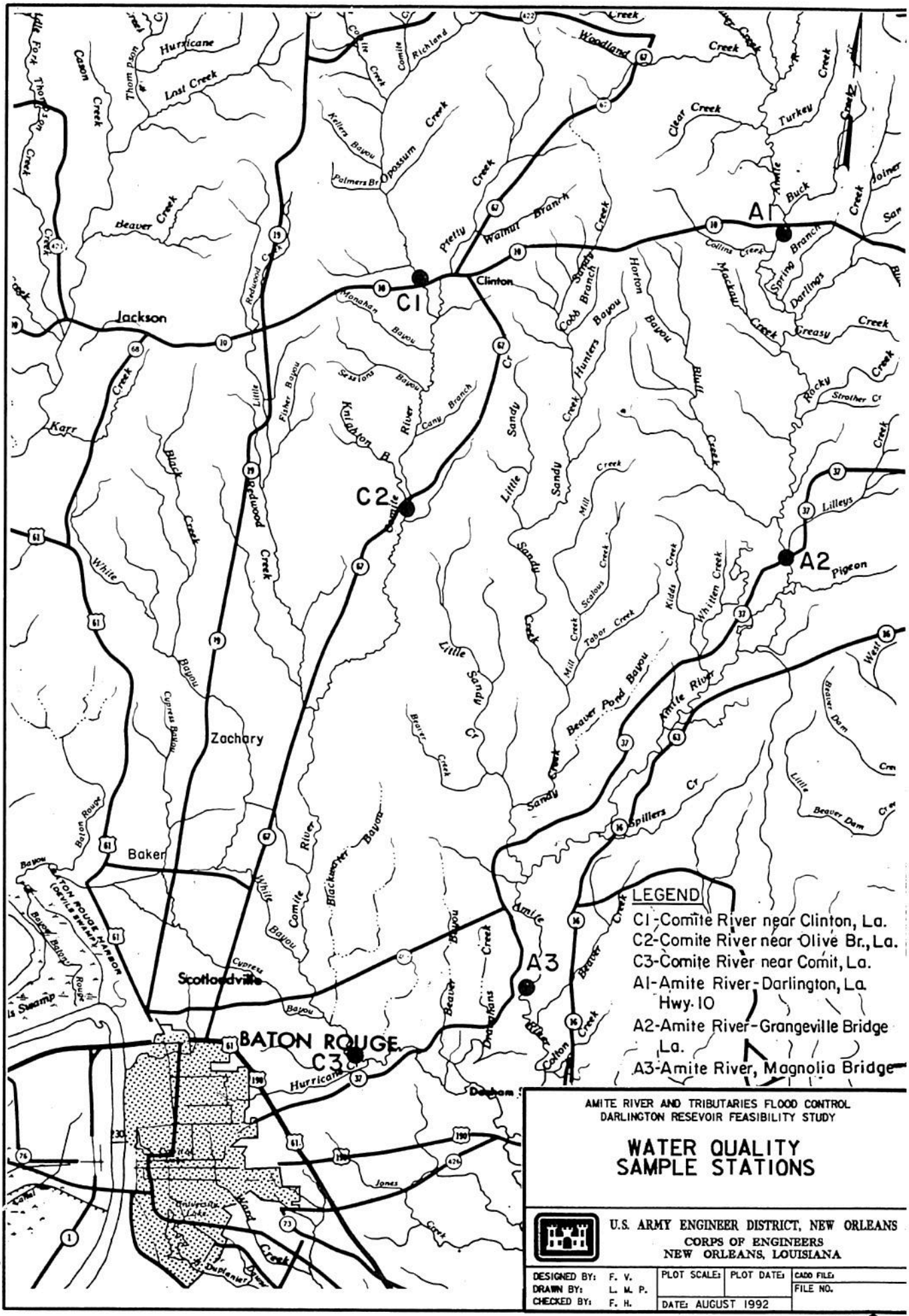
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

AMITE RIVER BASIN
1985 LAND USE

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

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PLATE 7



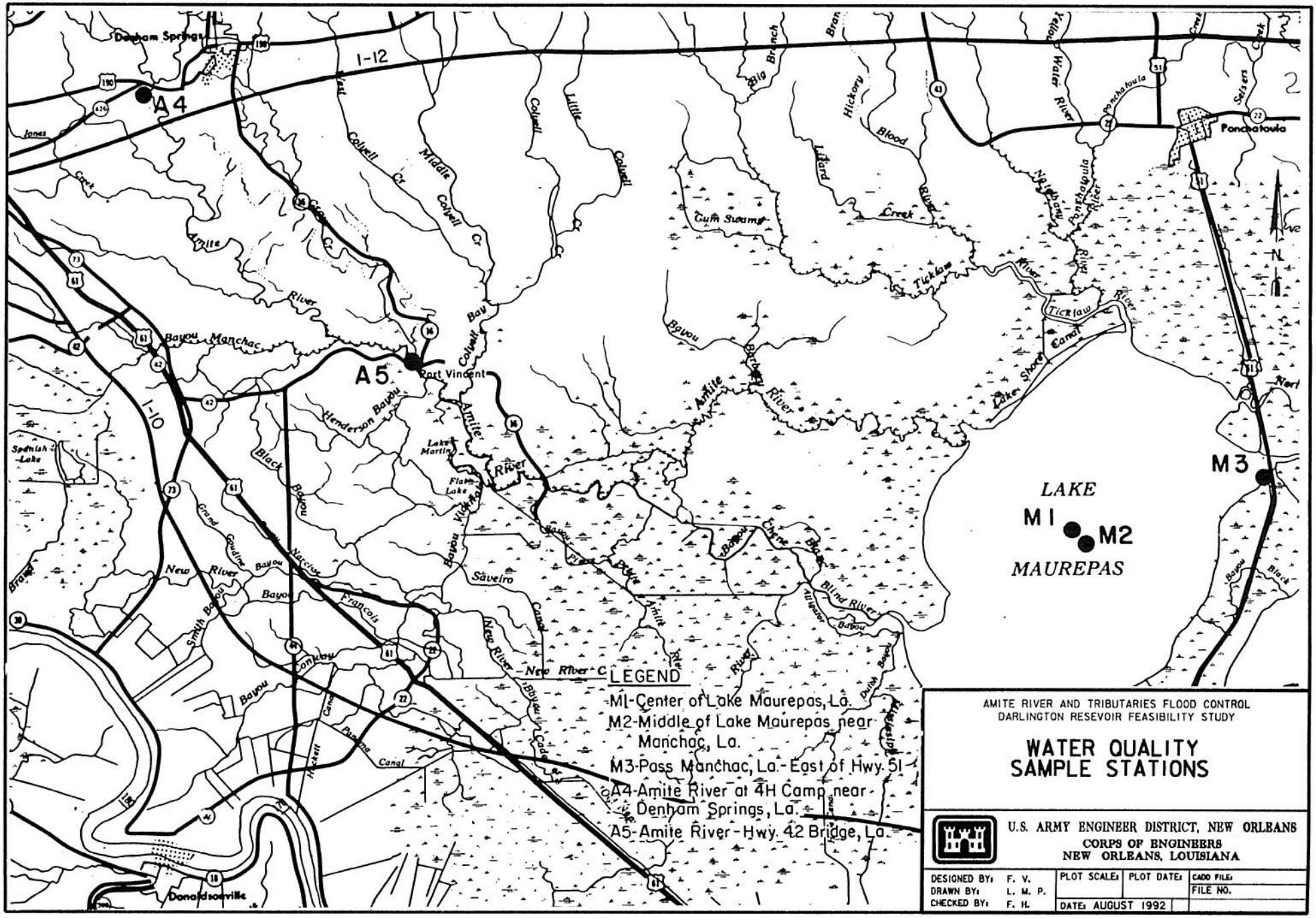
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

WATER QUALITY SAMPLE STATIONS

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

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PLATE 8



AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

WATER QUALITY SAMPLE STATIONS

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

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PLATE 9

EAST FELICIANA PARISH

ST. HELENA PARISH

TANGIPAHOA PARISH LOUISIANA

Clinton

Jackson

Zachary

Baker

Scotlandville

BATON ROUGE

Denham Springs

Amite

Hammond

Ponchatoula

Plaquemine

Donaldsonville

MISSISSIPPI RIVER

LAKE MAUREPAS

1-10

1-12

1:250,000
SCALE IN MILES

0 5 10 15

AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

HYDROLOGIC SUB-BASIN IN THE AMITE RIVER

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

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PLATE 10

EAST FELICIANA PARISH

ST. HELENA PARISH

TANGIPAHOA PARISH LOUISIANA

NORTHWEST

NORTHEAST

SOUTHEAST

URBAN

SOUTHERN

LIVINGSTON

ASCENSION

LOWER BASIN

LAKE MAUREPAS

MISSISSIPPI RIVER

Jackson

Clinton

Zachary

Baker

Plaquemine

Donaldsonville

Amite

Hammond

Ponchatoula

1-10

1-12

1:250,000
SCALE IN MILES

0 5 10 15

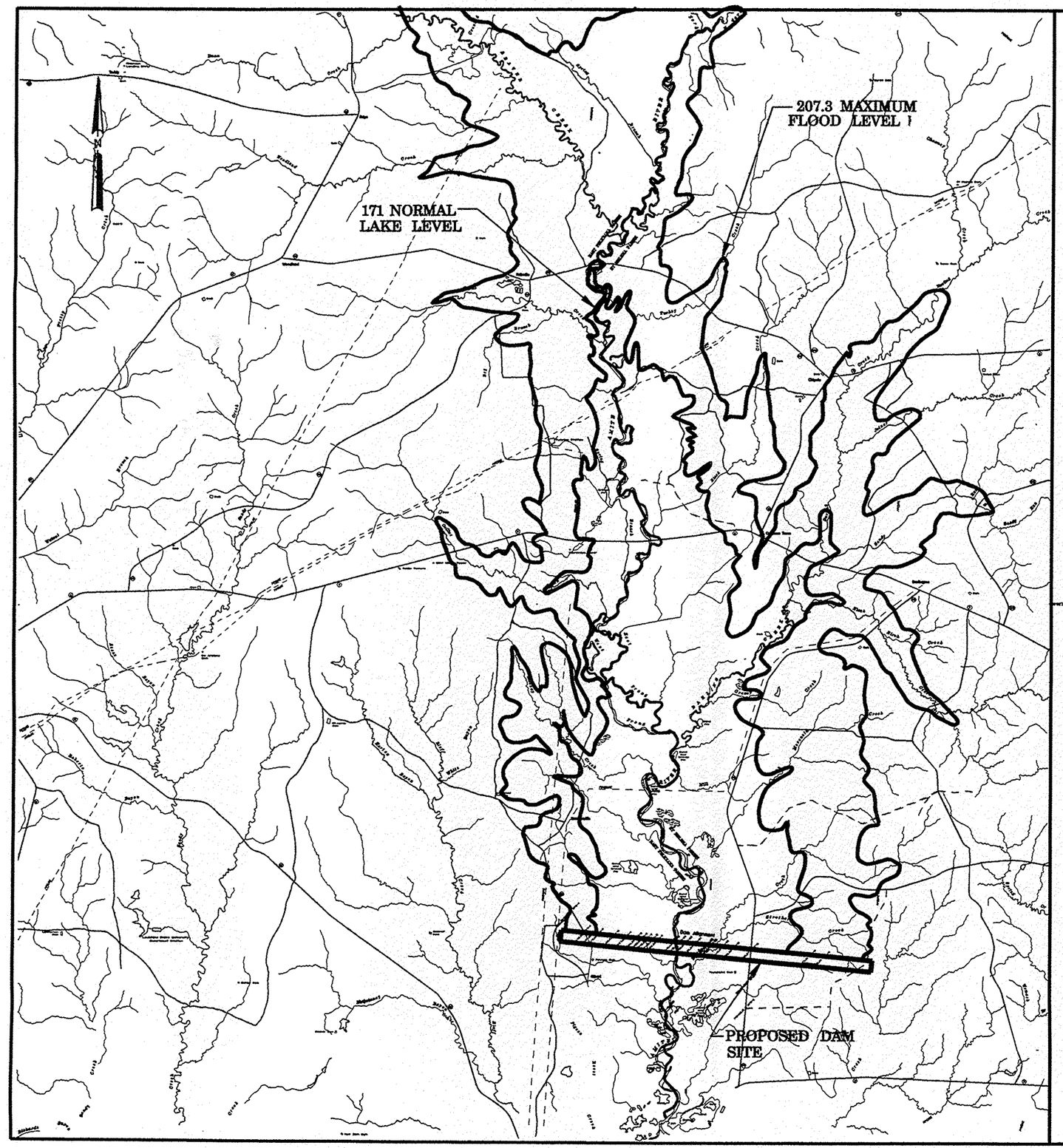
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

LAND USE GROWTH REGIONS

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

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CHECKED BY: F. H.	DATE: AUGUST 1992			

PLATE 11



SCALE IN MILES

1 ½ 0 1 MILE

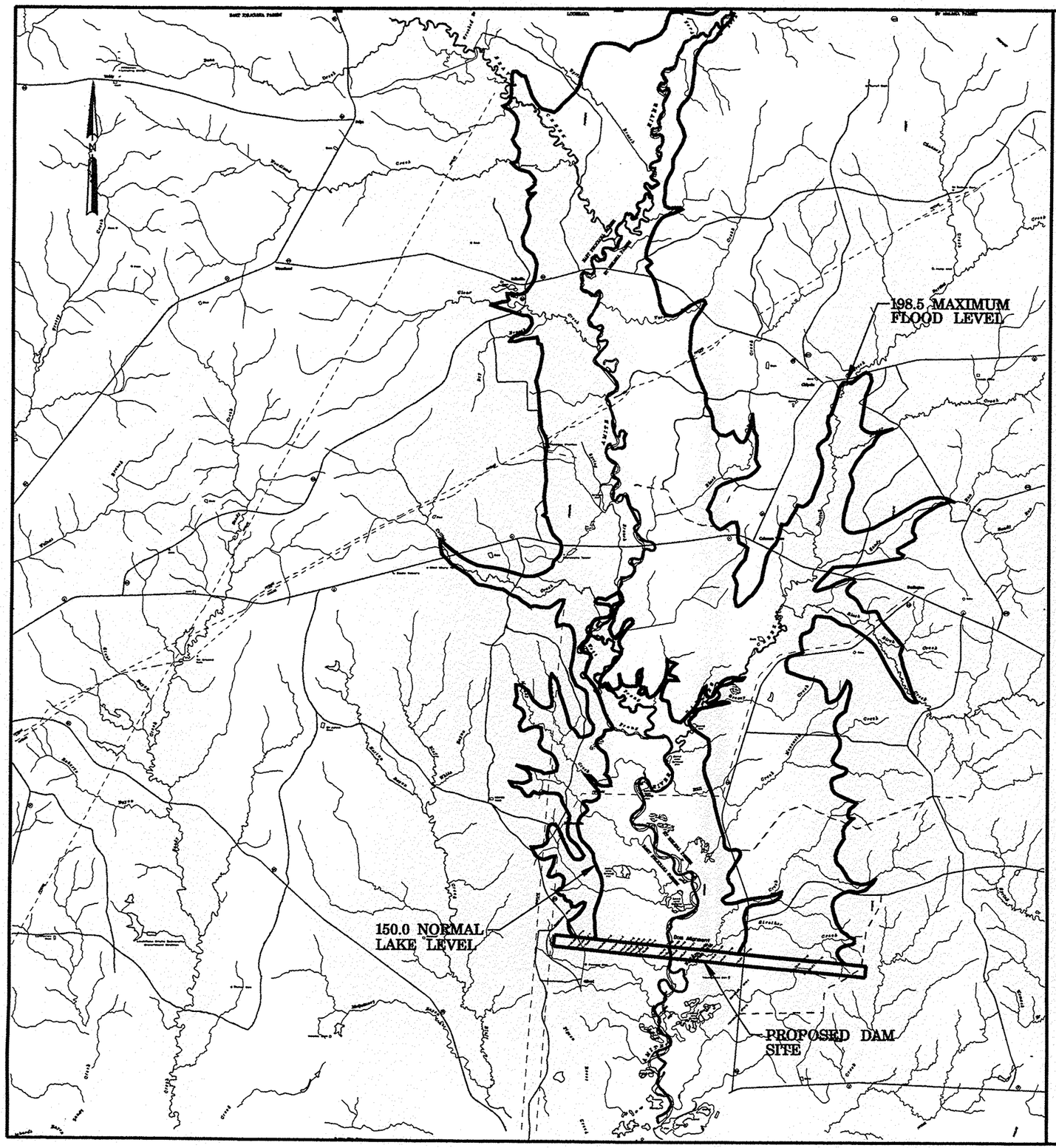
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY

WET RESERVOIR
(50 YEAR DESIGN)

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

DESIGNED BY:	F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE: DARESV9.DGN
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PLATE 12



SCALE IN MILES

1 ½ 0 MILE

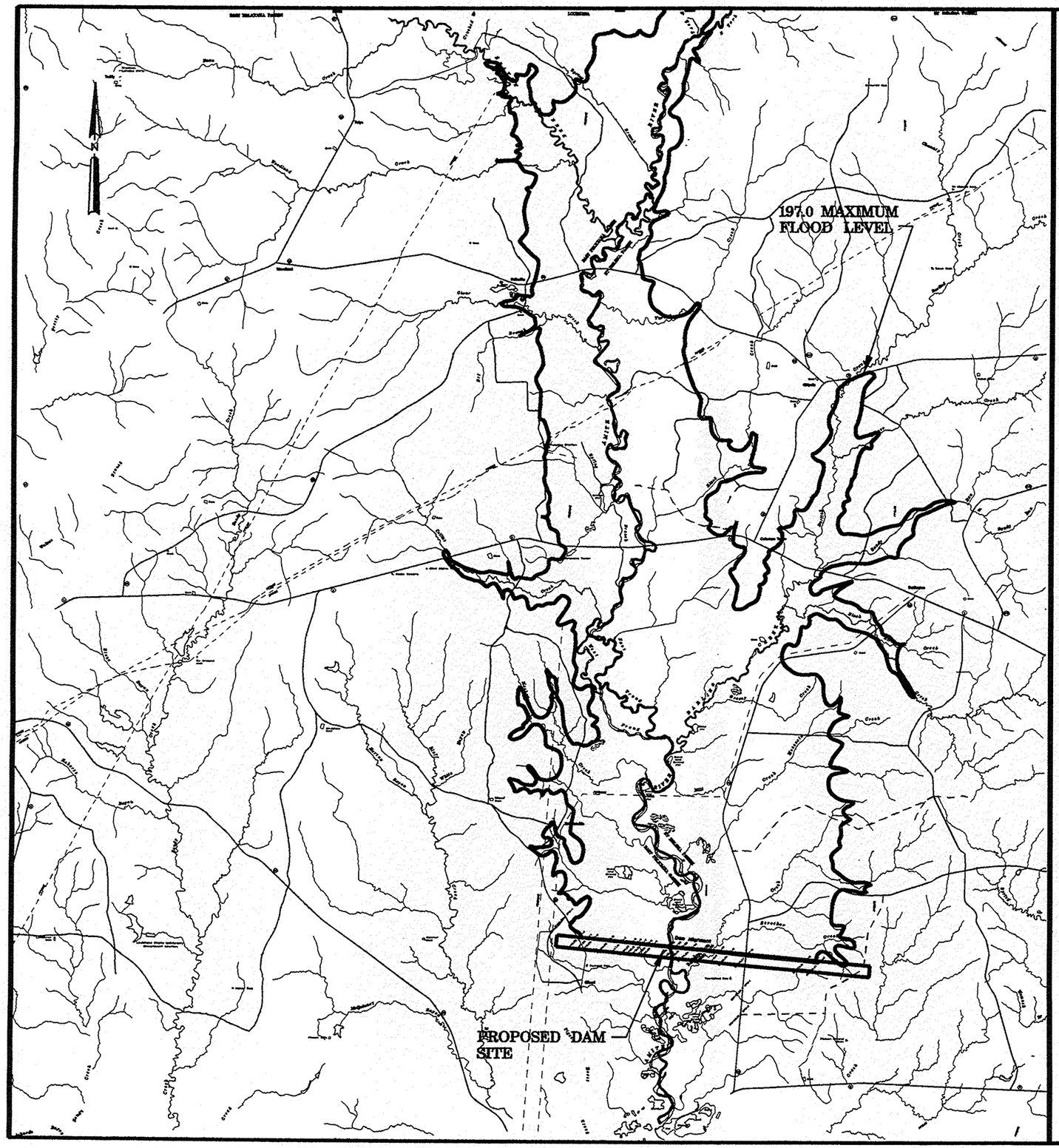
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY

REDUCED WET RESERVOIR
(50 YEAR DESIGN)

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

DESIGNED BY:	F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE:	DARESV10.DGN
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CHECKED BY:	F. H.	DATE: AUGUST 1992			

PLATE 13



SCALE IN MILES

1 ½ 0 1 MILE

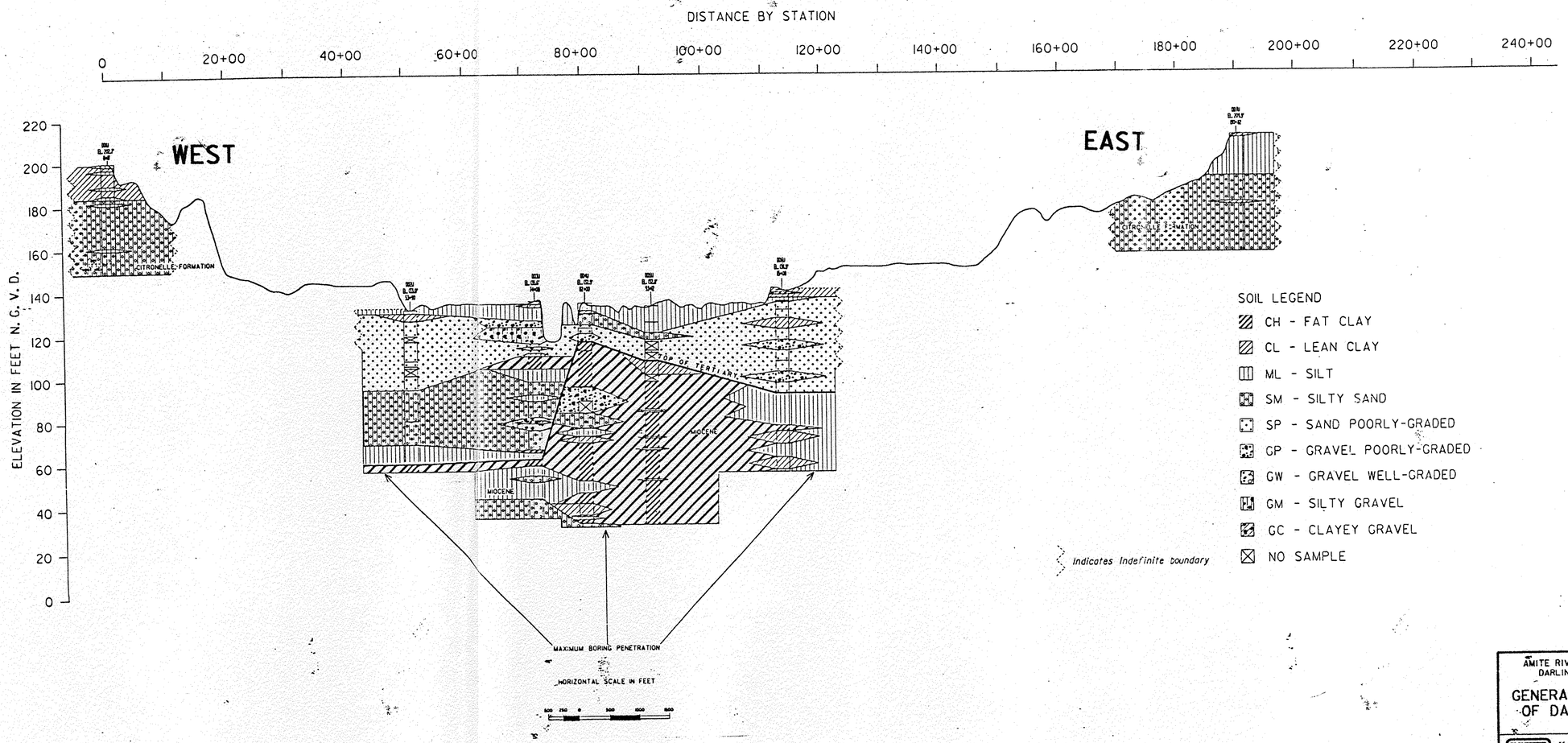
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY

DRY RESERVOIR
(50 YEAR DESIGN)

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

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CHECKED BY:	F. H.	DATE: AUGUST 1992		

PLATE 14



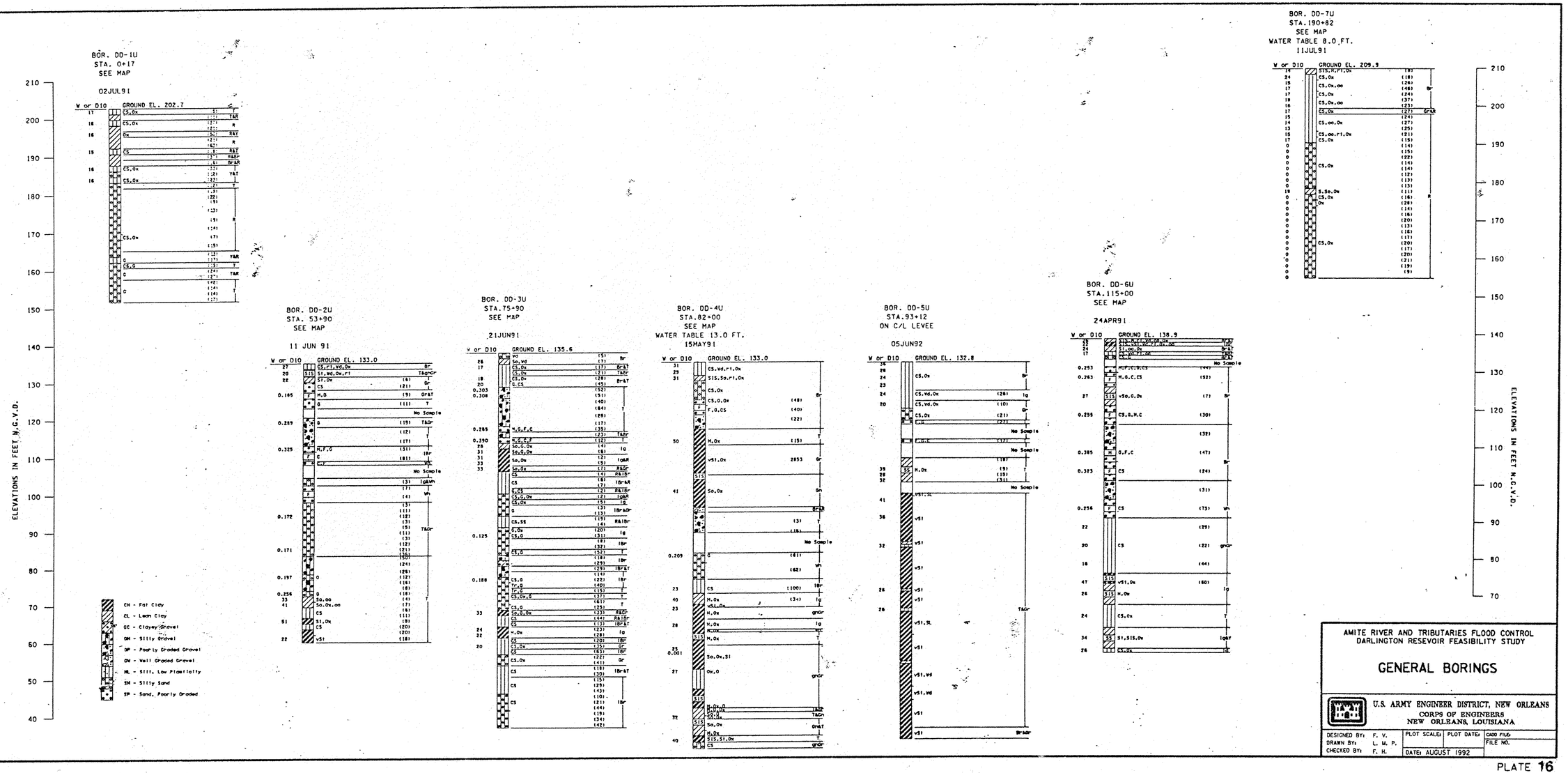
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

GENERALIZED CROSS SECTION OF DARLINGTON RESERVOIR

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

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CHECKED BY: F. H.	DATE: AUGUST 1992		

PLATE 15



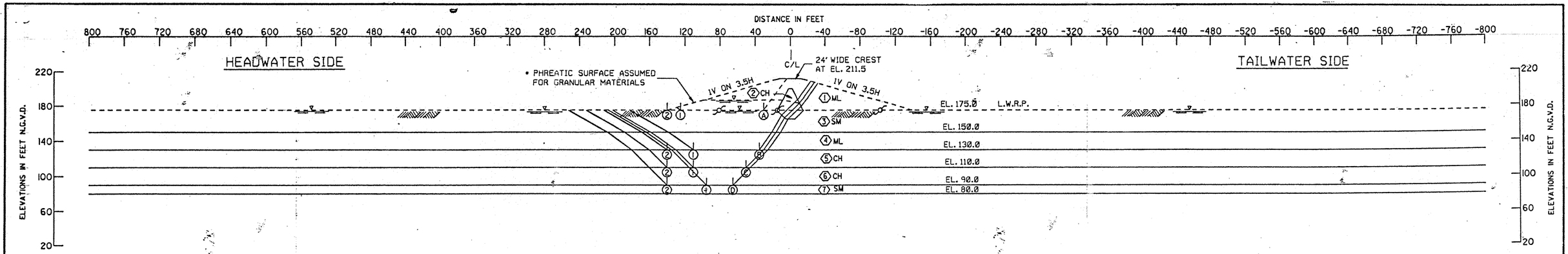
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

GENERAL BORINGS

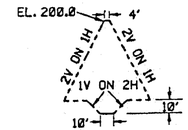
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE:
DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		

PLATE 16



NOTE: EMBANKMENT SYMMETRIC ABOUT C/L



CORE DETAIL

GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES.

CONSTRUCTION CASE

ASSUMED FAILURE SURFACE NO.	ASSUMED FAILURE SURFACE ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓐ ①	175.0	45564	57362	2341	74040	750	105267	73290	1.44
Ⓐ ②	175.0	45564	61551	0	74040	0	107115	74040	1.45
Ⓑ ①	130.0	174794	90000	107390	374088	132906	372184	241182	1.54
Ⓑ ②	130.0	174794	126000	96579	374088	122521	397373	251567	1.58
Ⓒ ①	110.0	219547	72000	145747	571916	267073	437294	304843	1.43
Ⓒ ②	110.0	219547	108000	144579	571916	252718	472126	319198	1.48
Ⓓ ①	90.0	314420	72000	240860	806720	462129	627280	344591	1.82
Ⓓ ②	90.0	314420	180000	240579	806720	428713	734999	378007	1.94

* SUDDEN DRAWDOWN CASE

ASSUMED FAILURE SURFACE NO.	ASSUMED FAILURE SURFACE ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓐ ①	175.0	41378	48429	1982	70474	750	91789	69724	1.32
Ⓐ ②	175.0	41378	51981	0	70474	0	93359	70474	1.32
Ⓑ ①	130.0	157465	90000	101075	374088	132905	348540	241183	1.45
Ⓑ ②	130.0	157465	126000	96579	374088	122521	380044	251567	1.51
Ⓒ ①	110.0	203935	72000	143928	571912	267070	419863	304842	1.38
Ⓒ ②	110.0	203935	108000	144579	571912	252718	456514	319194	1.43
Ⓓ ①	90.0	299806	72000	239970	806716	462123	611776	344593	1.78
Ⓓ ②	90.0	299806	180000	240579	806716	428713	720385	378003	1.91

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.	BOTTOM OF STRATUM VERT.	FRICTION ANGLE DEGREES
①	ML	117	200	200	18
②	CH	112	600	600	0
③	SM	122	0	0	30
④	ML	117	300	300	20
⑤	CH	112	1200	1200	0
⑥	CH	117	2400	2400	0
⑦	SM	122	0	0	30

NOTES

- ○-- STRATUM NUMBER
- ○-- WEDGE NUMBER
- ⌀-- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ▽ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\dfrac{R_A + R_B + R_P}{D_A - D_P}$

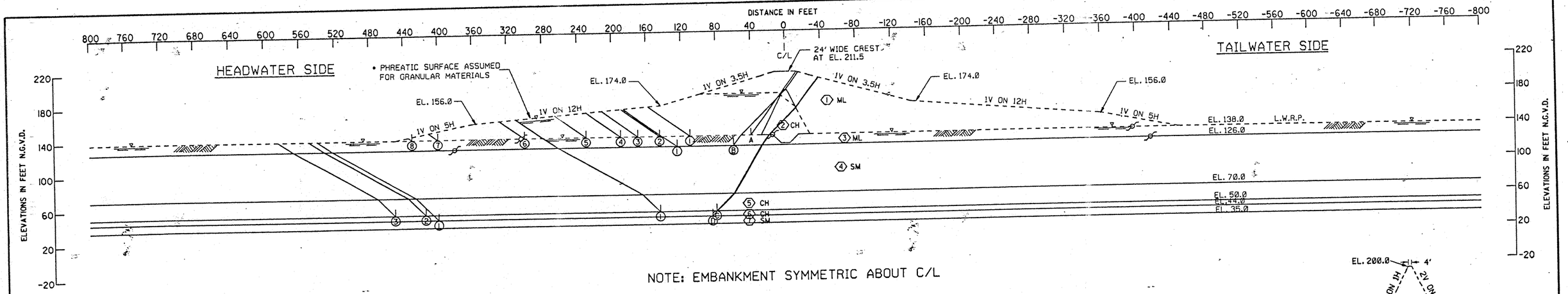
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

WEST ABUTMENT STABILITY ANALYSIS

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE:
DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		

PLATE 17



NOTE: EMBANKMENT SYMMETRIC ABOUT C/L

GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES.

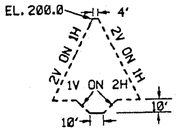
CONSTRUCTION CASE

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓐ ①	138.0	165092	48000	98244	306091	88432	311336	217659	1.43
Ⓐ ②	138.0	165092	69000	78060	306091	67475	312152	238616	1.31
Ⓐ ③	138.0	165092	84000	70222	306091	59862	319314	246229	1.30
Ⓐ ④	138.0	165092	96000	64244	306091	54099	325336	251992	1.29
Ⓐ ⑤	138.0	165092	120000	53072	306091	43448	338164	262643	1.29
Ⓐ ⑥	138.0	165092	162000	36030	306091	27616	363122	278475	1.30
Ⓐ ⑦	138.0	165092	221958	8244	306091	4459	395294	301632	1.31
Ⓐ ⑧	138.0	165092	235781	2278	306091	683	403151	305408	1.32
Ⓑ ①	126.0	131679	174775	149954	386309	131255	456408	255054	1.79
Ⓒ ①	50.0	566585	96000	795074	432685	812552	457659	620133	2.35
Ⓒ ②	50.0	566585	420000	324943	432685	478202	1311528	954483	1.37
Ⓒ ③	50.0	566585	462000	322179	432685	465448	1350764	967237	1.40
Ⓓ ①	44.0	595055	792000	358316	538872	556159	1745371	982713	1.78

* SUDDEN DRAWDOWN CASE

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓐ ①	138.0	121933	42000	53986	297942	88430	217919	209512	1.04
Ⓐ ②	138.0	121933	63000	45821	297942	67475	230754	230467	1.00
Ⓐ ③	138.0	121933	78000	41620	297942	59862	241553	238080	1.01
Ⓐ ④	138.0	121933	90000	38396	297942	54099	250329	243843	1.03
Ⓐ ⑤	138.0	121933	114000	32312	297942	43448	268245	254494	1.05
Ⓐ ⑥	138.0	121933	156000	22834	297942	27616	300767	270326	1.11
Ⓐ ⑦	138.0	121933	207516	6114	297942	4459	335563	293483	1.14
Ⓐ ⑧	138.0	121933	217160	1951	297942	683	341044	297259	1.15
Ⓑ ①	126.0	112986	104804	86837	386309	131255	304627	255054	1.19
Ⓒ ①	50.0	428227	78000	548967	1412125	812552	1055194	599573	1.76
Ⓒ ②	50.0	428227	402000	31494l	1412125	478202	1145168	933923	1.23
Ⓒ ③	50.0	428227	444000	322179	1412125	465448	1194406	946677	1.26
Ⓓ ①	44.0	458849	756000	342014	1516644	556159	1556863	960485	1.62

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.	BOTTOM OF STRATUM VERT.	FRICTION ANGLE DEGREES
①	ML	117	200	200	18
②	CH	112	600	600	0
③	ML	117	300	300	20
④	SM	122	0	0	30
⑤	CH	112	1200	1200	0
⑥	CH	117	2400	2400	0
⑦	SM	122	0	0	30



CORE DETAIL

NOTES

- ○-- STRATUM NUMBER
- ○-- WEDGE NUMBER
- ⌀-- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ▽ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
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- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

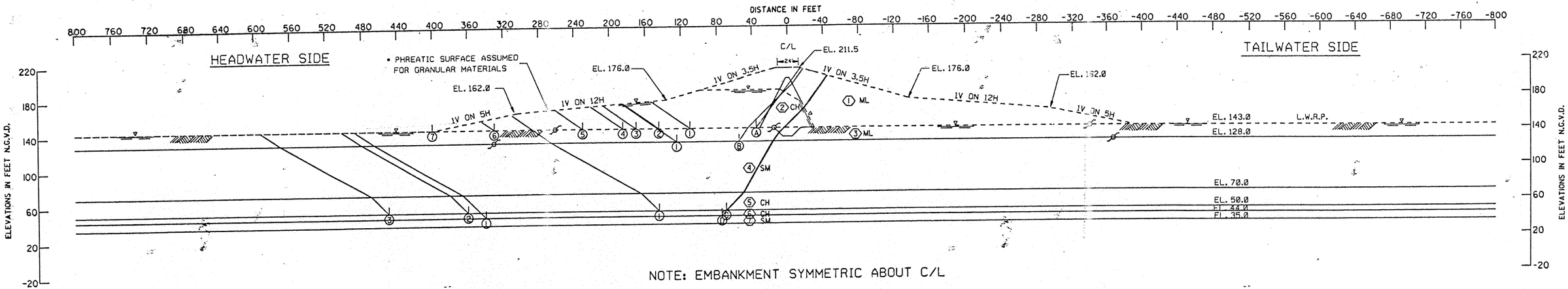
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

WEST RIVER TERRACE STABILITY ANALYSIS

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE:
DRAWN BY: L. M. P.	-		FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		

PLATE 18



NOTE: EMBANKMENT SYMMETRIC ABOUT C/L

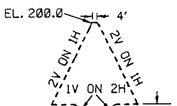
* SUDDEN DRAWDOWN CASE

ASSUMED FAILURE SURFACE NO.	ASSUMED FAILURE SURFACE ELEV.	RESISTING FORCES R_A	RESISTING FORCES R_B	RESISTING FORCES R_P	DRIVING FORCES D_A	DRIVING FORCES $-D_P$	SUMMATION OF FORCES RESISTING	SUMMATION OF FORCES DRIVING	FACTOR OF SAFETY
Ⓐ ①	143.0	110621	45000	45070	262166	70934	200691	191232	1.05
Ⓐ ②	143.0	110621	66000	38713	262166	54662	215334	207504	1.04
Ⓐ ③	143.0	110621	81000	34838	262166	47833	226459	214333	1.06
Ⓐ ④	143.0	110621	90000	32604	262166	43953	233225	218213	1.07
Ⓐ ⑤	143.0	110621	117000	26314	262166	33300	253935	228866	1.11
Ⓐ ⑥	143.0	110621	171244	7581	262166	7120	289446	255046	1.13
Ⓐ ⑦	143.0	110621	191488	0	262166	0	302109	262166	1.15
Ⓑ ①	128.0	113996	112039	85784	370481	126480	311819	244001	1.28
Ⓒ ①	50.0	440759	90000	564460	1426851	829718	1095219	597133	1.83
Ⓒ ②	50.0	440759	348000	350251	1426851	530618	1139010	896233	1.27
Ⓒ ③	50.0	440759	456000	362189	1426851	519158	1258948	907693	1.39
Ⓓ ①	44.0	470133	636000	376202	1532526	616271	1482335	916255	1.62

CONSTRUCTION CASE

ASSUMED FAILURE SURFACE NO.	ASSUMED FAILURE SURFACE ELEV.	RESISTING FORCES R_A	RESISTING FORCES R_B	RESISTING FORCES R_P	DRIVING FORCES D_A	DRIVING FORCES $-D_P$	SUMMATION OF FORCES RESISTING	SUMMATION OF FORCES DRIVING	FACTOR OF SAFETY
Ⓐ ①	143.0	144905	48000	80826	265881	70936	273731	194945	1.40
Ⓐ ②	143.0	144905	69000	64830	265881	54662	278735	211219	1.32
Ⓐ ③	143.0	144905	84000	57693	265881	47833	286598	218048	1.31
Ⓐ ④	143.0	144905	93000	53605	265881	43953	291510	221928	1.31
Ⓐ ⑤	143.0	144905	120000	42225	265881	33300	307130	232581	1.32
Ⓐ ⑥	143.0	144905	180000	11747	265881	7120	336652	258761	1.30
Ⓐ ⑦	143.0	144905	208397	0	265881	0	353302	265881	1.33
Ⓑ ①	128.0	134269	164805	145034	364179	126480	444108	237699	1.87
Ⓒ ①	50.0	558974	96000	786906	1431440	829719	1441880	601721	2.40
Ⓒ ②	50.0	558974	354000	364235	1431440	530617	1277209	900823	1.42
Ⓒ ③	50.0	558974	462000	362188	1431440	519157	1383162	912283	1.52
Ⓓ ①	44.0	587397	648000	400336	1537670	616271	1635733	921399	1.78

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.1	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.1	C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM VERT.1	FRICTION ANGLE DEGREES
①	ML	117	200	200	18
②	CH	112	600	600	0
③	ML	117	300	300	20
④	SM	122	0	0	30
⑤	CH	112	1200	1200	0
⑥	CH	117	2400	2400	0
⑦	SM	122	0	0	30



CORE DETAIL

GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES.

NOTES

- ○ -- STRATUM NUMBER
- ○ -- WEDGE NUMBER
- ∽ -- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
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FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

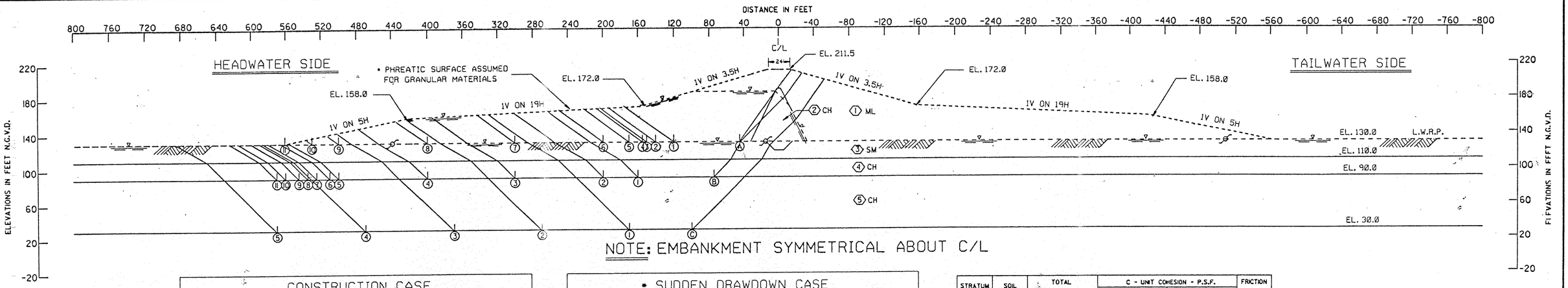
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY.

WEST ABUTMENT TERRACE STABILITY ANALYSIS

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE:
DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		

PLATE 19



NOTE: EMBANKMENT SYMMETRICAL ABOUT C/L

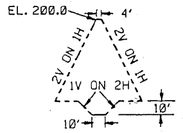
CONSTRUCTION CASE

ASSUMED FAILURE SURFACE NO.	ASSUMED FAILURE SURFACE ELEV.	RESISTING FORCES R_A	RESISTING FORCES R_B	RESISTING FORCES R_P	DRIVING FORCES D_A	DRIVING FORCES $-D_P$	SUMMATION OF FORCES RESISTING	SUMMATION OF FORCES DRIVING	FACTOR OF SAFETY
Ⓐ ①	130.0	199208	51000	123228	373546	112754	373436	260792	1.43
Ⓐ ②	130.0	199208	63000	111073	373546	99769	373281	273777	1.36
Ⓐ ③	130.0	199208	69000	107692	373546	96291	375900	277255	1.36
Ⓐ ④	130.0	199208	72000	106481	373546	95088	377689	278458	1.36
Ⓐ ⑤	130.0	199208	81000	102889	373546	91525	383097	282021	1.36
Ⓐ ⑥	130.0	199208	99000	95889	373546	84605	394097	288941	1.36
Ⓐ ⑦	130.0	199208	159000	74311	373546	63501	432519	310045	1.40
Ⓐ ⑧	130.0	199208	219000	52093	373546	43155	470301	330391	1.42
Ⓐ ⑨	130.0	199208	279000	10062	373546	5815	488270	367731	1.33
Ⓐ ⑩	130.0	199208	294368	3406	373546	1269	496982	372277	1.33
Ⓐ ⑪	130.0	199208	303198	0	373546	0	502406	373546	1.34
Ⓑ ①	90.0	262346	108000	349322	789424	365258	719668	424166	1.70
Ⓑ ②	90.0	262346	156000	330673	789424	346778	749019	442646	1.69
Ⓑ ③	90.0	262346	276000	285939	789424	302692	824285	486732	1.69
Ⓑ ④	90.0	262346	396000	208886	789424	236408	867232	553016	1.57
Ⓑ ⑤	90.0	262346	516000	89710	789424	126152	868056	663272	1.31
Ⓑ ⑥	90.0	262346	528000	81143	789424	117256	871489	672168	1.30
Ⓑ ⑦	90.0	262346	546000	73524	789424	106428	881870	682996	1.29
Ⓑ ⑧	90.0	262346	558000	71821	789424	100897	892167	688527	1.30
Ⓑ ⑨	90.0	262346	570000	71798	789424	97091	904144	692333	1.31
Ⓑ ⑩	90.0	262346	588000	71798	789424	95597	922144	693827	1.33
Ⓑ ⑪	90.0	262346	600000	71797	789424	95596	934143	693828	1.35
Ⓒ ①	29.0	547278	180000	609317	1770125	1116818	1336595	653307	2.05
Ⓒ ②	29.0	547278	420000	564701	1770125	1035821	1531979	734304	2.09
Ⓒ ③	29.0	547278	660000	461234	1770125	906923	1668512	863202	1.93
Ⓒ ④	29.0	547278	900000	364978	1770125	685738	1812256	1084387	1.67
Ⓒ ⑤	29.0	547278	1140000	364595	1770125	598740	2051873	1171385	1.75

* SUDDEN DRAWDOWN CASE

ASSUMED FAILURE SURFACE NO.	ASSUMED FAILURE SURFACE ELEV.	RESISTING FORCES R_A	RESISTING FORCES R_B	RESISTING FORCES R_P	DRIVING FORCES D_A	DRIVING FORCES $-D_P$	SUMMATION OF FORCES RESISTING	SUMMATION OF FORCES DRIVING	FACTOR OF SAFETY
Ⓐ ①	130.0	142406	45000	67681	363406	112753	255087	250653	1.02
Ⓐ ②	130.0	142406	57000	62863	363406	99768	262269	263638	.99
Ⓐ ③	130.0	142406	63000	63049	363406	96291	268455	267115	1.01
Ⓐ ④	130.0	142406	66000	63595	363406	95088	272001	268318	1.01
Ⓐ ⑤	130.0	142406	75000	62009	363406	91525	279415	271881	1.03
Ⓐ ⑥	130.0	142406	93000	57788	363406	84604	293194	278802	1.05
Ⓐ ⑦	130.0	142406	153000	44768	363406	63501	340174	299905	1.13
Ⓐ ⑧	130.0	142406	213000	31563	363406	43155	386969	320251	1.21
Ⓐ ⑨	130.0	142406	266552	6791	363406	5815	415749	357591	1.16
Ⓐ ⑩	130.0	142406	276753	2569	363406	1269	421728	362137	1.16
Ⓐ ⑪	130.0	142406	284126	0	363406	0	426532	363406	1.17
Ⓑ ①	90.0	211403	102000	217097	782163	365259	530500	416904	1.27
Ⓑ ②	90.0	211403	150000	206481	782163	346778	567884	435385	1.30
Ⓑ ③	90.0	211403	270000	181075	782163	302692	662478	479471	1.38
Ⓑ ④	90.0	211403	390000	137202	782163	236407	738605	545756	1.35
Ⓑ ⑤	90.0	211403	510000	77721	782163	126151	799124	656012	1.22
Ⓑ ⑥	90.0	211403	522000	73321	782163	117256	806724	664907	1.21
Ⓑ ⑦	90.0	211403	540000	69451	782163	106428	820854	675735	1.21
Ⓑ ⑧	90.0	211403	552000	70248	782163	100897	833651	681266	1.22
Ⓑ ⑨	90.0	211403	564000	71798	782163	97092	847201	685071	1.24
Ⓑ ⑩	90.0	211403	582000	71797	782163	95597	865200	686566	1.26
Ⓑ ⑪	90.0	211403	594000	71797	782163	95597	877200	686566	1.28
Ⓒ ①	29.0	522540	168000	491233	1766410	1116819	1181773	649591	1.82
Ⓒ ②	29.0	522540	408000	465781	1766410	1035819	1396321	730591	1.91
Ⓒ ③	29.0	522540	648000	409754	1766410	906921	1580294	859489	1.84
Ⓒ ④	29.0	522540	888000	362405	1766410	685736	1772945	1080674	1.64
Ⓒ ⑤	29.0	522540	1128000	364596	1766410	598740	2015136	1167670	1.73

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.↓	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.↓	C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM VERT.↓	FRICTION ANGLE DEGREES
①	ML	117	200	200	18
②	CH	112	600	600	0
③	SM	122	0	0	30
④	CH	112	1200	1200	0
⑤	CH	117	2400	2400	0



CORE DETAIL

GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES.

NOTES

○ -- STRATUM NUMBER
○ -- WEDGE NUMBER
ℴ -- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
▽ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

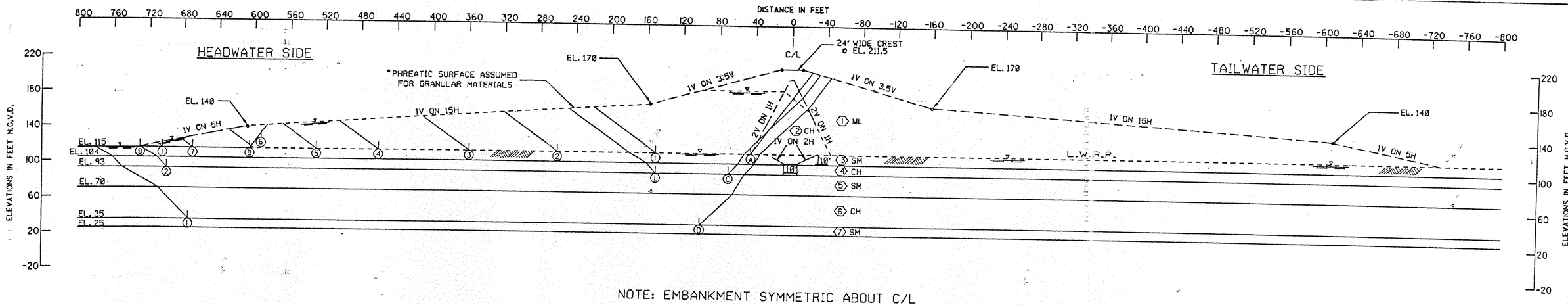
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

EAST RIVER TERRACE STABILITY ANALYSIS

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE:
DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		

PLATE 20



NOTE: EMBANKMENT SYMMETRIC ABOUT C/L

GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES.

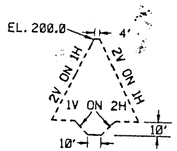
CONSTRUCTION CASE

ASSUMED FAILURE SURFACE NO.	ASSUMED FAILURE SURFACE ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓐ ①	115.0	272544	72000	171739	520498	161100	516283	359398	1.44
Ⓐ ②	115.0	272544	132000	135469	520498	124306	540013	396192	1.36
Ⓐ ③	115.0	272544	192000	103459	520498	92276	568003	428222	1.33
Ⓐ ④	115.0	272544	252000	75708	520498	65008	600252	455490	1.32
Ⓐ ⑤	115.0	272544	294000	58818	520498	48755	625362	471743	1.33
Ⓐ ⑥	115.0	272544	336000	39962	520498	31890	648506	488608	1.33
Ⓐ ⑦	115.0	272544	378000	11806	520498	7166	662350	513332	1.29
Ⓐ Ⓑ	115.0	272544	404454	226	520498	11	677224	520487	1.30
Ⓑ ①	115.0	26066	57603	3615	38930	1387	87284	37543	2.32
Ⓒ ①	93.0	209204	264000	264635	744868	250012	737839	494856	1.49
Ⓒ ②	93.0	209204	755825	39984	744868	38960	1005013	705908	1.42
Ⓓ ①	35.0	566025	1416000	295059	1664140	416518	277084	1247622	1.83

*SUDDEN DRAWDOWN CASE

ASSUMED FAILURE SURFACE NO.	ASSUMED FAILURE SURFACE ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓐ ①	115.0	194226	63000	94881	516460	165564	352107	350896	1.00
Ⓐ ②	115.0	194226	129000	76076	516460	124306	399302	392154	1.02
Ⓐ ③	115.0	194226	189000	59370	516460	92276	442596	424184	1.04
Ⓐ ④	115.0	194226	249000	44646	516460	65008	487872	451452	1.08
Ⓐ ⑤	115.0	194226	291000	35522	516460	48755	520748	467705	1.11
Ⓐ ⑥	115.0	194226	333000	25152	516460	31890	552378	484570	1.14
Ⓐ ⑦	115.0	194226	370492	8382	516460	7166	573100	509294	1.13
Ⓐ Ⓑ	115.0	194226	389397	221	516460	11	583844	516449	1.13
Ⓑ ①	115.0	16246	47069	2952	38930	1387	66267	37543	1.77
Ⓒ ①	93.0	176251	96000	189358	744868	323596	461609	421272	1.10
Ⓒ ②	93.0	176251	748692	36688	744868	38960	961631	705908	1.36
Ⓓ ①	35.0	463114	380000	288594	1642963	416518	2131708	1226445	1.74

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. I	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. I	BOTTOM OF STRATUM VERT. I	FRICTION ANGLE DEGREES
①	ML	117	200	200	18
②	CH	112	600	600	0
③	SM	122	0	0	30
④	CH	112	1200	1200	0
⑤	SM	122	0	0	30
⑥	CH	117	2400	2400	0
⑦	SM	122	0	0	30



CORE DETAIL

NOTES

- ◇-- STRATUM NUMBER
- ○-- WEDGE NUMBER
- ⌀-- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
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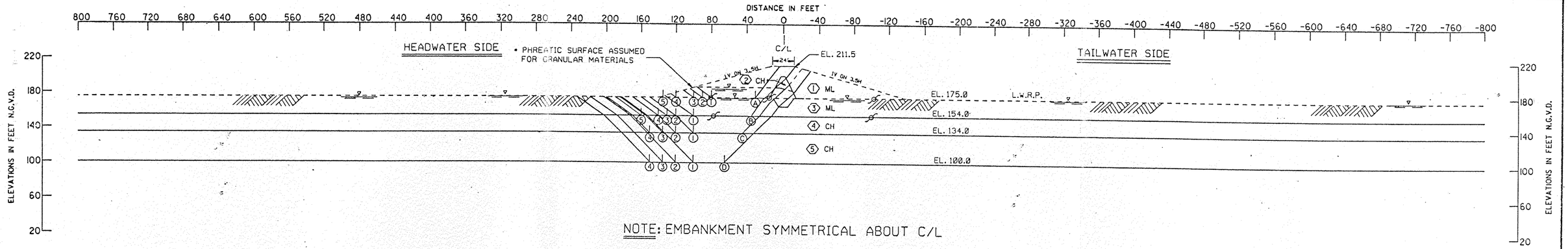
FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

RIVER CLOSURE STABILITY ANALYSIS

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE:
DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.		DATE: AUGUST 1992	



NOTE: EMBANKMENT SYMMETRICAL ABOUT C/L

GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES.

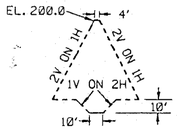
STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. I	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. I	C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM VERT. I	FRICTION ANGLE DEGREES
①	ML	117	200	200	18
②	CH	112	600	600	0
③	ML	117	300	300	20
④	CH	112	1200	1200	0
⑤	CH	117	2400	2400	0

CONSTRUCTION CASE

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓐ ①	175.0	45564	33000	17703	74039	12247	96267	61792	1.56
Ⓐ ②	175.0	45564	39000	13217	74039	8493	97781	65546	1.49
Ⓐ ③	175.0	45564	45000	9344	74039	5425	99908	68614	1.46
Ⓐ ④	175.0	45564	55423	3435	74039	1343	104422	72696	1.44
Ⓐ ⑤	175.0	45564	60426	612	74039	79	106602	73960	1.44
Ⓑ ①	154.0	85823	90000	49982	186017	43531	225805	142486	1.58
Ⓑ ②	154.0	85823	111579	35254	186017	30384	232656	155633	1.49
Ⓑ ③	154.0	85823	120545	31655	186017	26921	238023	159096	1.50
Ⓑ ④	154.0	85823	128295	30487	186017	25798	244605	160219	1.53
Ⓑ ⑤	154.0	85823	142626	30487	186017	25798	258936	160219	1.62
Ⓒ ①	134.0	129846	78000	83255	331750	121836	291101	209914	1.39
Ⓒ ②	134.0	129846	102000	78487	331750	103885	310333	227865	1.36
Ⓒ ③	134.0	129846	120000	78487	331750	97721	328333	234029	1.40
Ⓒ ④	134.0	129846	138000	78487	331750	97336	346333	234414	1.48
Ⓓ ①	101.0	288955	108000	236887	658035	342501	633842	315534	2.01
Ⓓ ②	101.0	288955	156000	236887	658035	322587	681842	335448	2.03
Ⓓ ③	101.0	288955	192000	236886	658035	316422	717841	341613	2.10
Ⓓ ④	101.0	288955	228000	236886	658035	316037	753841	341998	2.20

• SUDDEN DRAWDOWN CASE

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓐ ①	175.0	41073	30000	13742	70473	12244	84815	58229	1.46
Ⓐ ②	175.0	41073	35530	9821	70473	8491	86424	61982	1.39
Ⓐ ③	175.0	41073	39986	6754	70473	5423	87813	65050	1.35
Ⓐ ④	175.0	41073	47002	2794	70473	1342	90869	69131	1.31
Ⓐ ⑤	175.0	41073	50935	575	70473	79	92583	70394	1.32
Ⓑ ①	154.0	81991	75010	40167	181272	43528	197168	137744	1.43
Ⓑ ②	154.0	81991	92713	31401	181272	30383	206111	150889	1.37
Ⓑ ③	154.0	81991	100724	29749	181272	26921	212464	154351	1.38
Ⓑ ④	154.0	81991	108161	30487	181272	25798	220639	155474	1.42
Ⓑ ⑤	154.0	81991	122493	30487	181272	25798	234971	155474	1.51
Ⓒ ①	134.0	127925	66000	79401	327584	121832	273326	205752	1.33
Ⓒ ②	134.0	127925	90000	78487	327584	103884	296412	223700	1.33
Ⓒ ③	134.0	127925	108000	78487	327584	97721	314412	229863	1.37
Ⓒ ④	134.0	127925	126000	78487	327584	97336	332412	230248	1.44
Ⓓ ①	101.0	283015	84000	236887	650896	342497	603902	308399	1.96
Ⓓ ②	101.0	283015	132000	236887	650896	322586	651902	328310	1.99
Ⓓ ③	101.0	283015	168000	236886	650896	316422	687901	334474	2.06
Ⓓ ④	101.0	283015	204000	236886	650896	316037	723901	334859	2.16



CORE DETAIL

NOTES

- ⬡ -- STRATUM NUMBER
- ○ -- WEDGE NUMBER
- ℘ -- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
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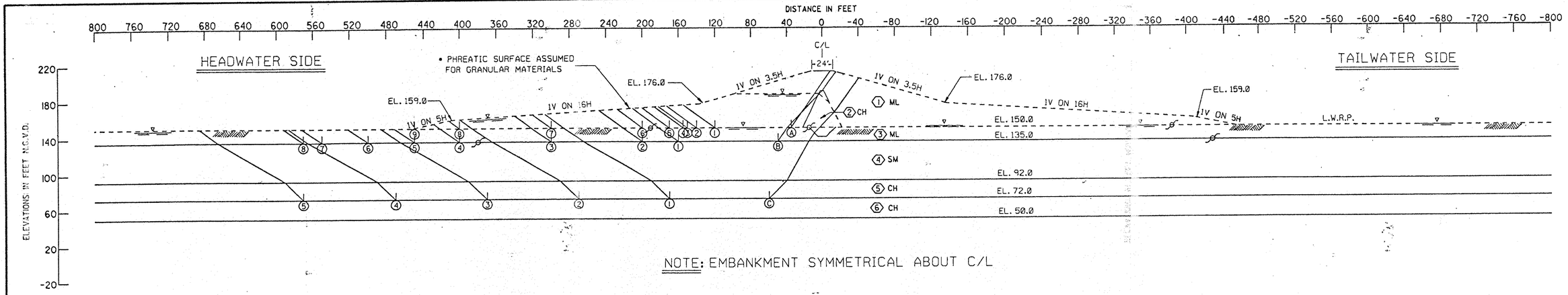
FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

EAST ABUTMENT STABILITY ANALYSIS

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CAD FILE:
DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		



NOTE: EMBANKMENT SYMMETRICAL ABOUT C/L

GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES.

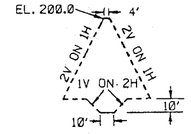
STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.	C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM VERT.	FRICTION ANGLE DEGREES
1	ML	117	200	200	18
2	CH	112	600	600	0
3	ML	117	300	300	20
4	SM	122	0	0	30
5	CH	112	1200	1200	0
6	CH	117	2400	2400	0

CONSTRUCTION CASE

ASSUMED FAILURE SURFACE NO.	ASSUMED FAILURE SURFACE ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
A 1	150.0	119438	57000	51118	216238	41839	227556	174399	1.30
A 2	150.0	119438	69000	45052	216238	35766	233490	180472	1.29
A 3	150.0	119438	75000	43202	216238	34051	237640	182187	1.30
A 4	150.0	119438	78000	42291	216238	33210	239729	183028	1.31
A 5	150.0	119438	87000	39615	216238	30750	246053	185488	1.33
A 6	150.0	119438	105000	34517	216238	26112	258955	190126	1.36
A 7	150.0	119438	165000	19970	216238	13391	304408	202847	1.50
A 8	150.0	119438	224693	8901	216238	4808	353032	211430	1.67
A 9	150.0	119438	245268	303	216238	20	365009	216218	1.69
B 1	135.0	122651	232057	100370	314464	83954	455078	230510	1.97
B 2	135.0	122651	295593	89259	314464	73660	507503	240804	2.11
B 3	135.0	122651	435800	64115	314464	50871	622566	263593	2.36
B 4	135.0	122651	549389	38779	314464	29583	710819	284881	2.50
B 5	135.0	122651	613594	19226	314464	13162	755471	301302	2.51
B 6	135.0	122651	618314	19226	314464	13162	760191	301302	2.52
B 7	135.0	122651	625394	19226	314464	13162	767271	301302	2.55
B 8	135.0	122651	630113	19226	314464	13161	771990	301303	2.56
B 9	135.0	122651	634833	19226	314464	13161	776710	301303	2.58
B 10	135.0	122651	641912	19225	314464	13161	783788	301303	2.60
B 11	135.0	122651	646632	19225	314464	13161	788508	301303	2.62
C 1	72.0	418975	138000	505323	1074322	565797	1062298	508525	2.09
C 2	72.0	418975	258000	419718	1074322	499651	1096693	574671	1.91
C 3	72.0	418975	378000	298545	1074322	414526	1095520	659796	1.66
C 4	72.0	418975	498000	247531	1074322	363824	1164506	710498	1.64
C 5	72.0	418975	618000	247527	1074322	363820	1284502	710502	1.81

• SUDDEN DRAWDOWN CASE

ASSUMED FAILURE SURFACE NO.	ASSUMED FAILURE SURFACE ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
A 1	150.0	91284	51[illegible]	29806	209077	41838	172090	167239	1.03
A 2	150.0	91284	63[illegible]	27963	209077	35766	182247	173311	1.05
A 3	150.0	91284	69[illegible]	26932	209077	34051	187216	175026	1.07
A 4	150.0	91284	72[illegible]	26423	209077	33210	189707	175867	1.08
A 5	150.0	91284	81[illegible]	24923	209077	30750	197207	178327	1.11
A 6	150.0	91284	98[illegible]	22040	209077	26112	212278	182965	1.16
A 7	150.0	91284	152[illegible]	13571	209077	13391	257262	195686	1.31
A 8	150.0	91284	194[illegible]	6678	209077	4808	292752	204269	1.43
A 9	150.0	91284	205[illegible]	293	209077	20	301292	209057	1.44
B 1	135.0	105018	150[illegible]	63300	314463	83954	319049	230509	1.38
B 2	135.0	105018	193[illegible]	57447	314463	73660	355559	240803	1.48
B 3	135.0	105018	290[illegible]	44041	314463	50871	439381	263592	1.67
B 4	135.0	105018	374[illegible]	30145	314463	29583	510160	284880	1.79
B 5	135.0	105018	407[illegible]	18999	314463	13251	531364	301212	1.76
B 6	135.0	105018	430[illegible]	19226	314463	13162	555225	301301	1.84
B 7	135.0	105018	454[illegible]	19226	314463	13161	578823	301302	1.92
B 8	135.0	105018	464[illegible]	19225	314463	13161	588262	301302	1.95
C 1	72.0	344044	132[illegible]	37211[illegible]	1071358	565797	848163	505561	1.68
C 2	72.0	344044	252[illegible]	330547	1071358	499651	926591	571707	1.62
C 3	72.0	344044	372[illegible]	271287	1071358	414526	987331	656832	1.50
C 4	72.0	344044	492[illegible]	247531	1071358	363824	1083575	707534	1.53
C 5	72.0	344044	612[illegible]	247527	1071358	363820	1203571	707538	1.70



CORE DETAIL

NOTES

- ○-- STRATUM NUMBER
- ○-- WEDGE NUMBER
- ∫-- CROSSOVER POINT
- Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
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FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

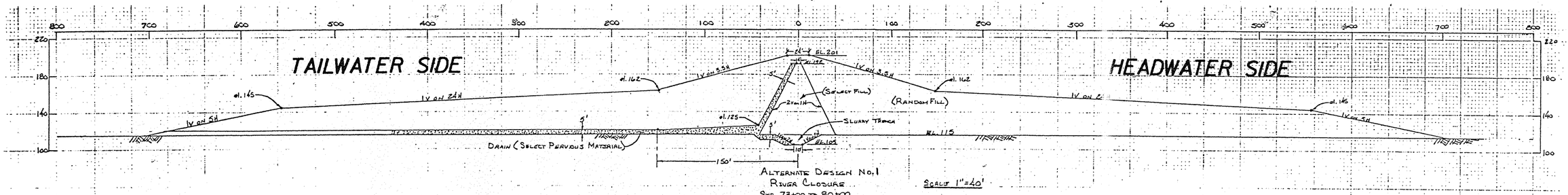
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DARLINGTON RESEVOIR FEASIBILITY STUDY

EAST ABUTMENT TERRACE STABILITY ANALYSIS

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

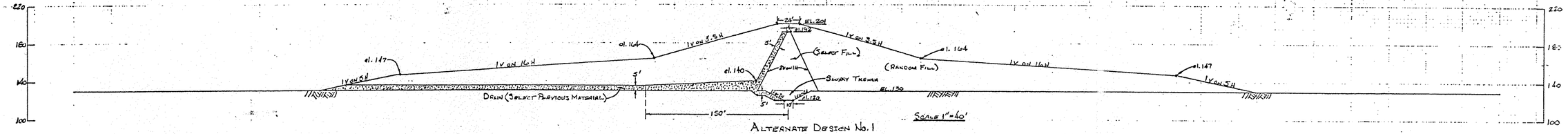
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DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		

PLATE 23



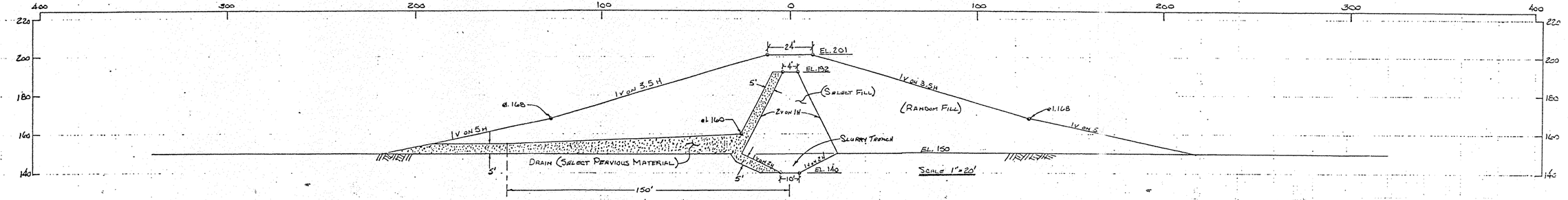
ALTERNATE DESIGN No. 1
RIVER CLOSURE
STA. 73+00 TO 80+00

SCALE 1"=40'



ALTERNATE DESIGN No. 1
EAST RIVER TERRACE
STA. 80+00 TO 120+00

SCALE 1"=40'



ALTERNATE DESIGN No. 1
EAST ABUTMENT TERRACE
STA. 120+00 TO 155+00

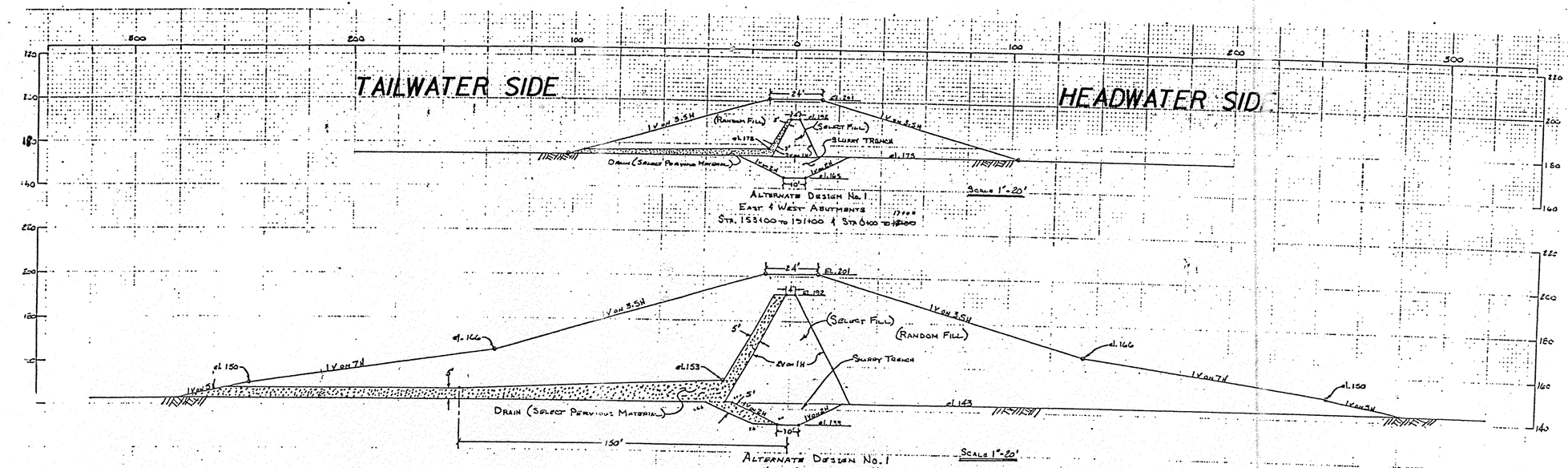
SCALE 1"=20'

AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

TYPICAL CROSS SECTIONS
ALTERNATE DESIGN NO. 1

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

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DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		



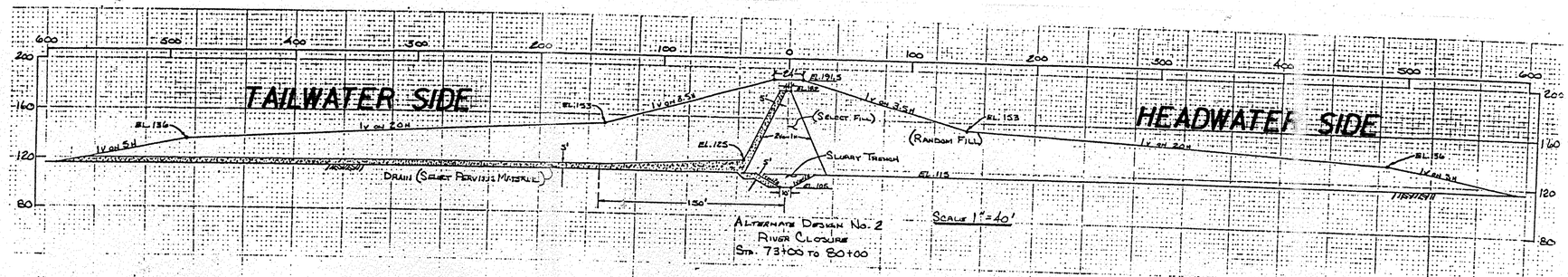
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

TYPICAL CROSS SECTIONS ALTERNATE DESIGN NO. 1

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

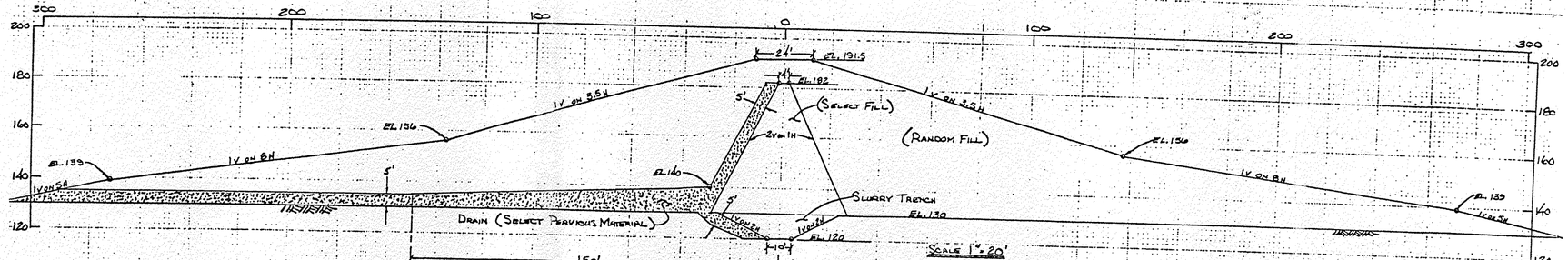
DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE:
DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		

PLATE 25



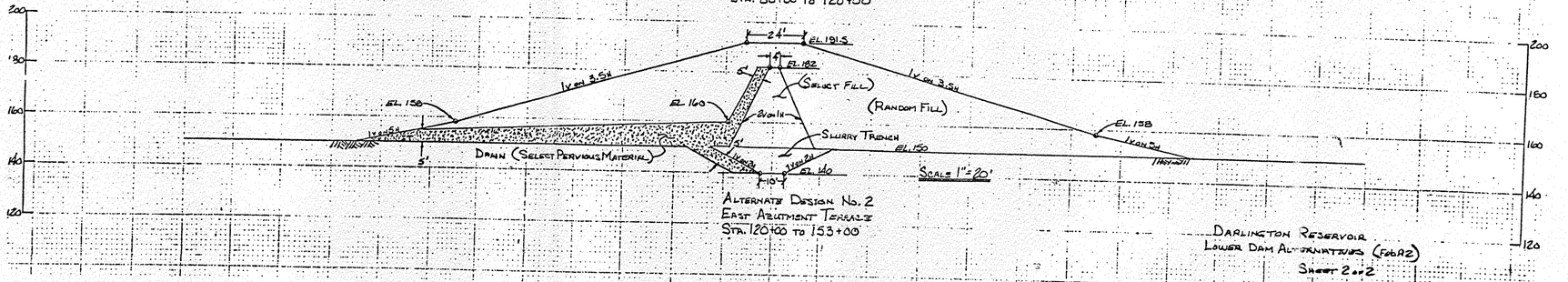
ALTERNATE DESIGN No. 2
RIVER CLOSURE
STA. 73+00 TO 80+00

SCALE 1"=40'



ALTERNATE DESIGN No. 2
EAST RIVER TERRACE
STA. 80+00 TO 120+00

SCALE 1"=20'



ALTERNATE DESIGN No. 2
EAST ABUTMENT TERRACE
STA. 120+00 TO 153+00

SCALE 1"=20'

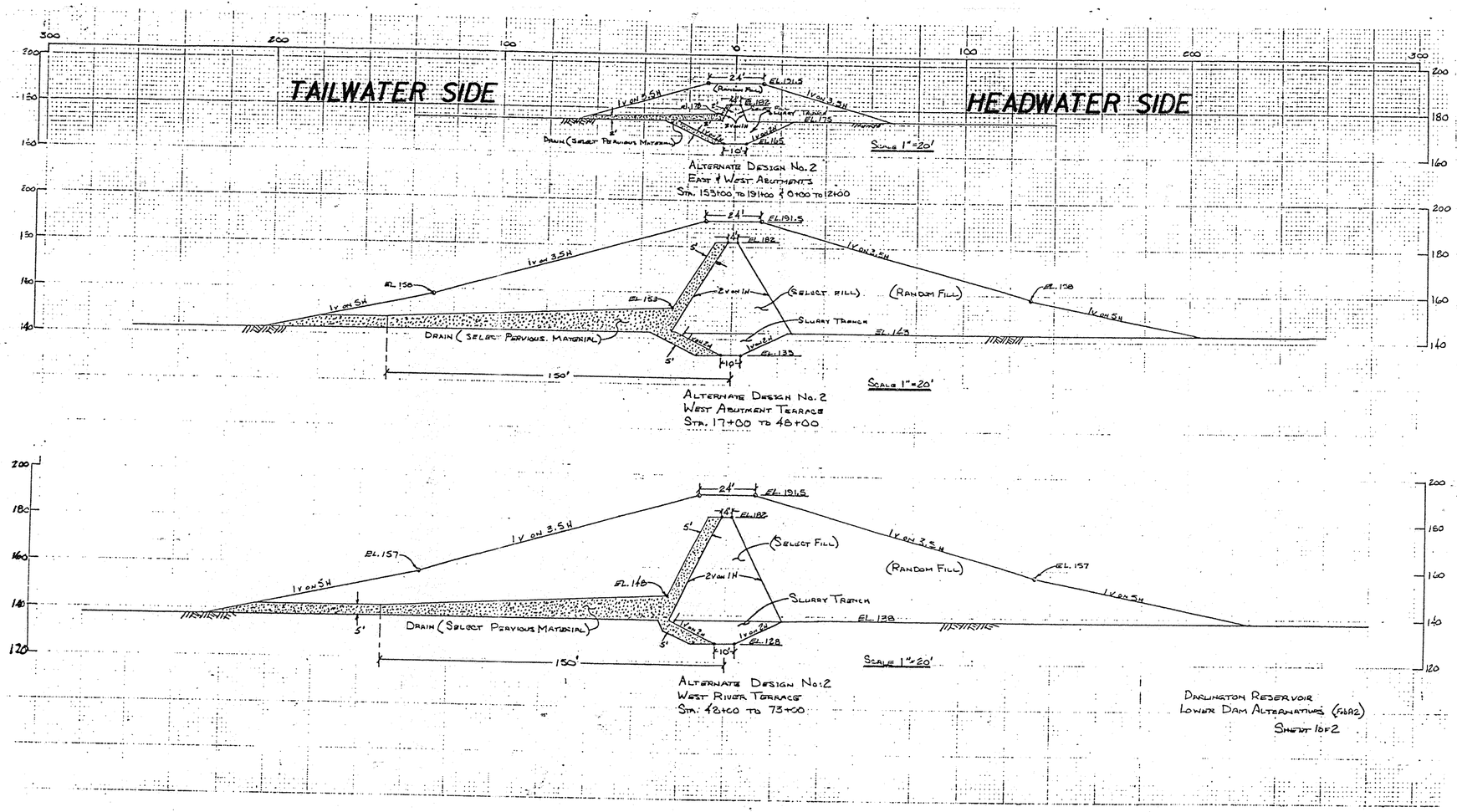
DARLINGTON RESERVOIR
LOWER DAM ALTERNATIVES (FEB82)
SHEET 2 of 2

AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

TYPICAL CROSS SECTIONS

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE:	
DRAWN BY: L. M. P.			FILE NO.	
CHECKED BY: F. H.	DATE: AUGUST 1992			



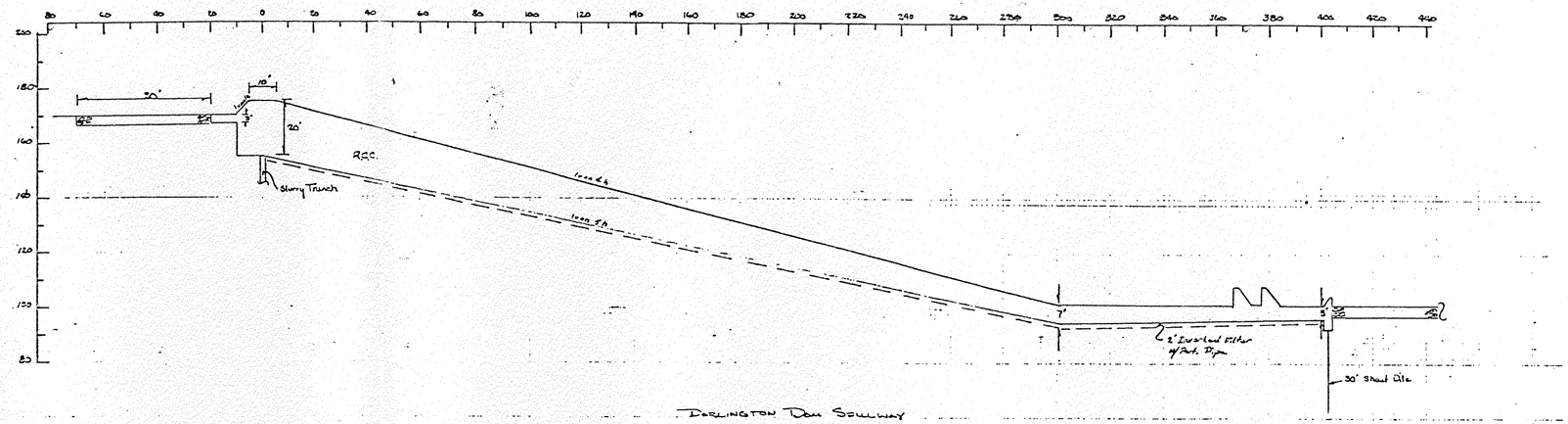
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

TYPICAL CROSS SECTIONS ALTERNATE DESIGN NO. 2

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE:	
DRAWN BY: L. M. P.				FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992			

PLATE 27



DARLINGTON DAM SPILLWAY

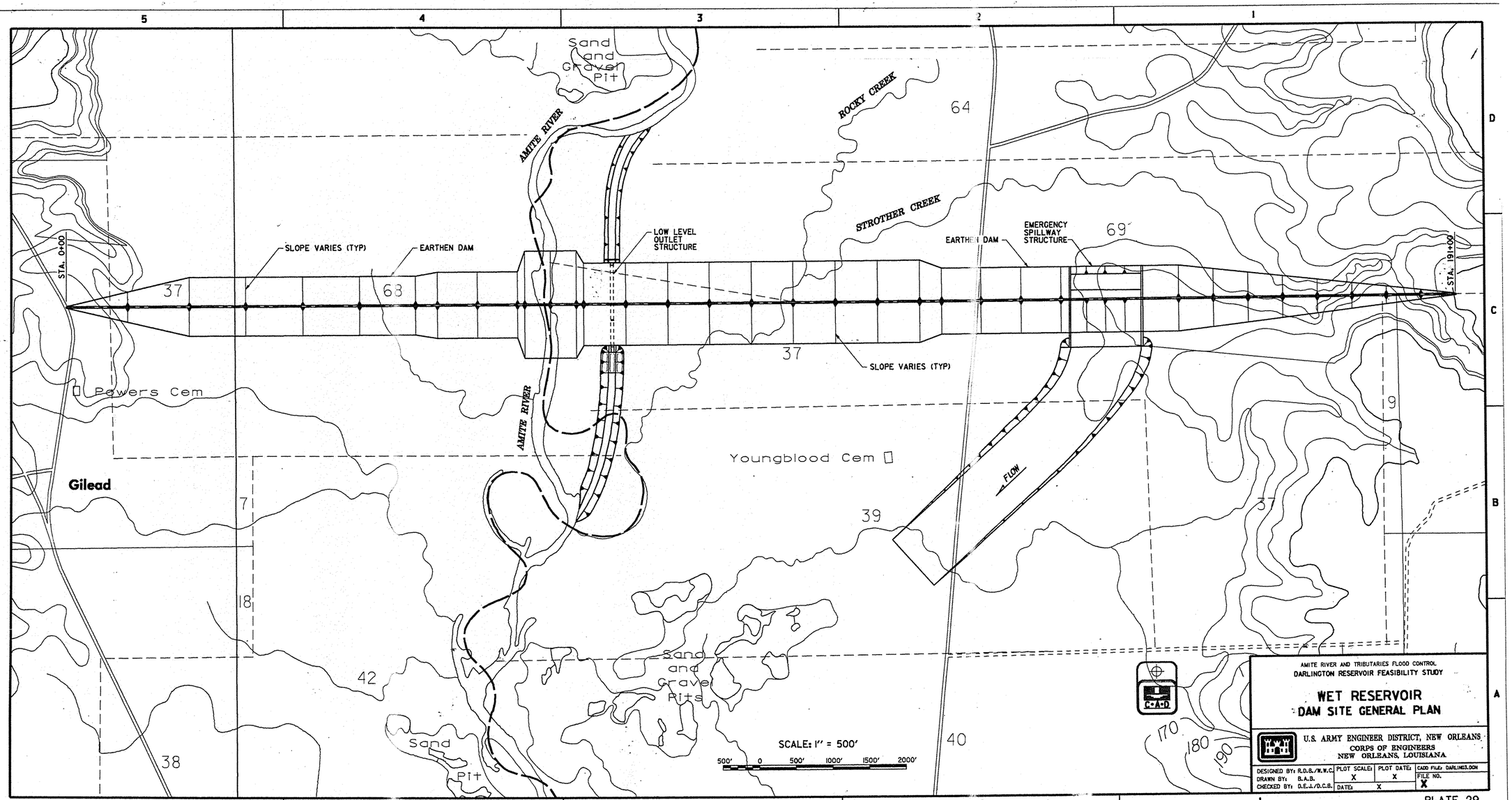
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

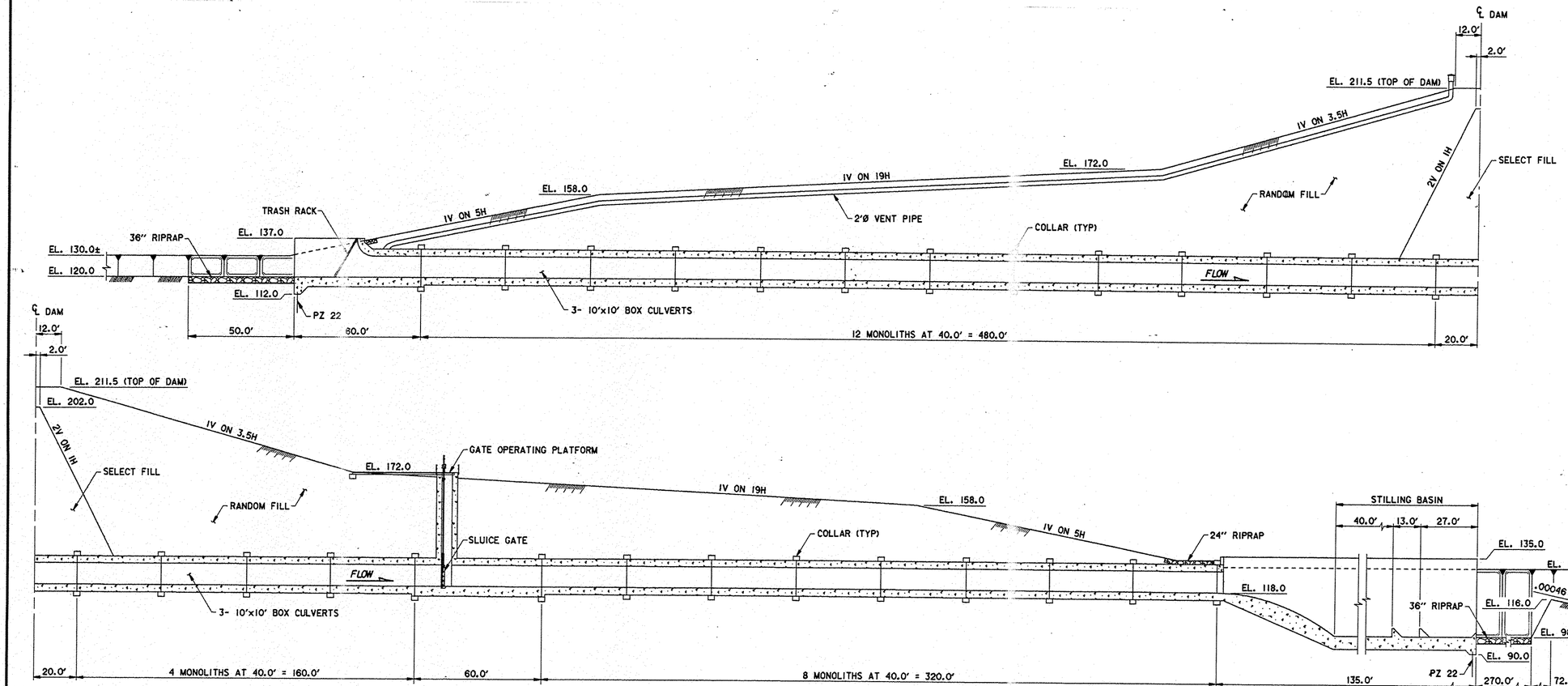
TYPICAL CROSS SECTION DARLINGTON DAM SPILLWAY

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE:
DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		

PLATE 28


Sand and Gravel Pit
AMITE RIVER
ROCKY CREEK
STROTHER CREEK
STA. 0+00
SLOPE VARIES (TYP)
EARTHEN DAM
LOW LEVEL OUTLET STRUCTURE
EARTHEN DAM
EMERGENCY SPILLWAY STRUCTURE
STA. 191+00
SLOPE VARIES (TYP)
Powers Cem
Youngblood Cem
Gilead
FLOW
Sand and Gravel Pits
Sand Pit
SCALE: 1" = 500'
500' 0 500' 1000' 1500' 2000'
C-A-D
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY
WET RESERVOIR
DAM SITE GENERAL PLAN
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
DESIGNED BY: R.D.S./W.W.C.
DRAWN BY: B.A.B.
CHECKED BY: D.E.J./D.C.B.
PLOT SCALE: X
PLOT DATE: X
CADD FILE: DARLING3.DGN
DATE: X
FILE NO. X
PLATE 29



SECTIONAL ELEVATION

SCALE: 1" = 20'-0"

SCALE: 1" = 20'

20' 0 20' 40' 60' 80'

C.A.D.

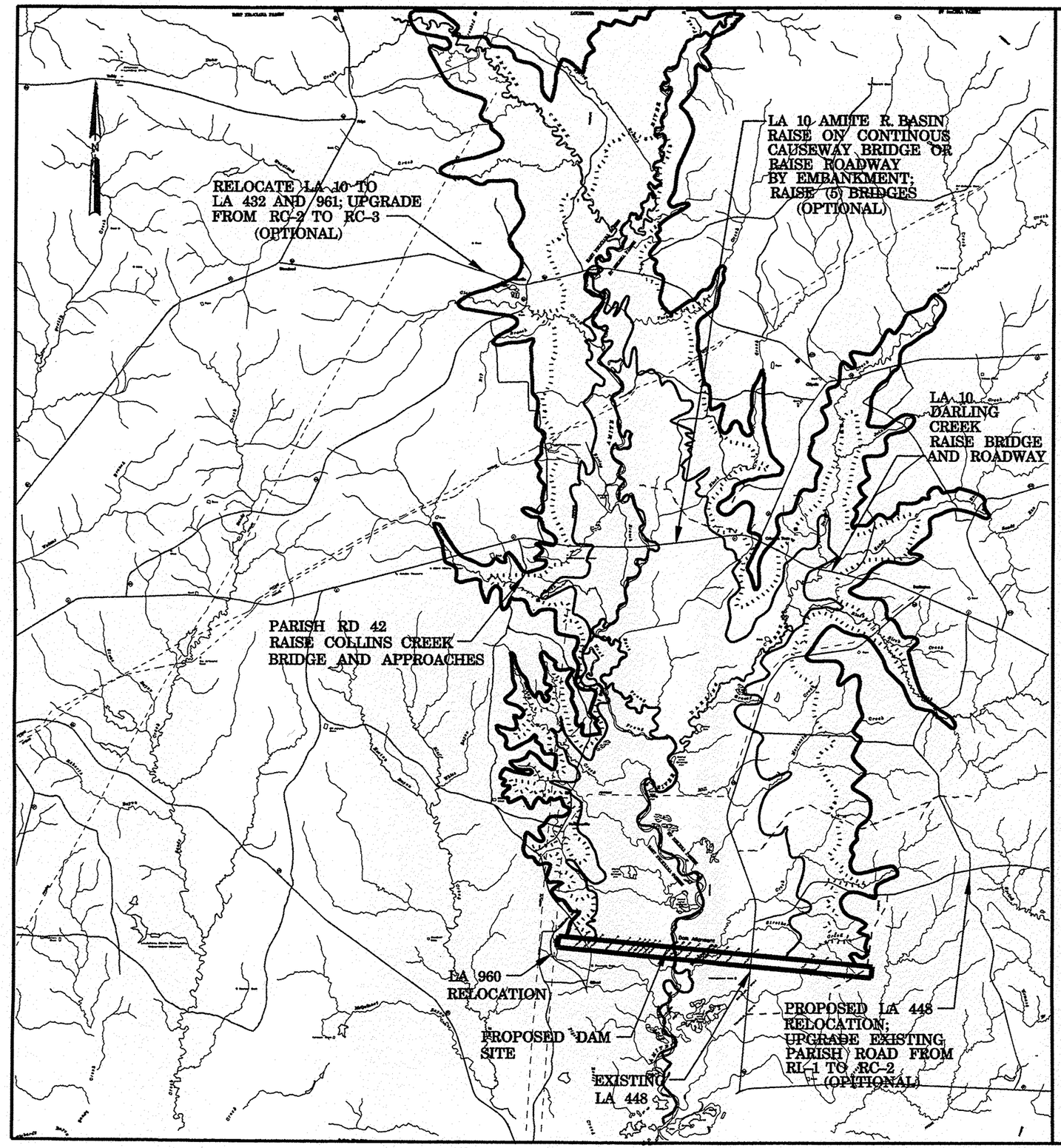
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY

WET RESERVOIR
LOW LEVEL OUTLET STRUCTURE

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

DESIGNED BY: R.O.B./C.G.M.	PLOT SCALE: X	PLOT DATE: X	CADD FILE: DARLING.DGN
DRAWN BY: S.A.B.			FILE NO. X
CHECKED BY: D.E.J./D.C.B.	DATE: X		

PLATE 30



LEGEND

PROPOSED MAXIMUM FLOOD LEVEL (EL. 207.3) ———

50 YEAR FLOOD LEVEL (EL. 186.9) .....

PROPOSED NORMAL LAKE LEVEL (EL. 171.0) ———

SCALE IN MILES

1 ½ 0 MILE

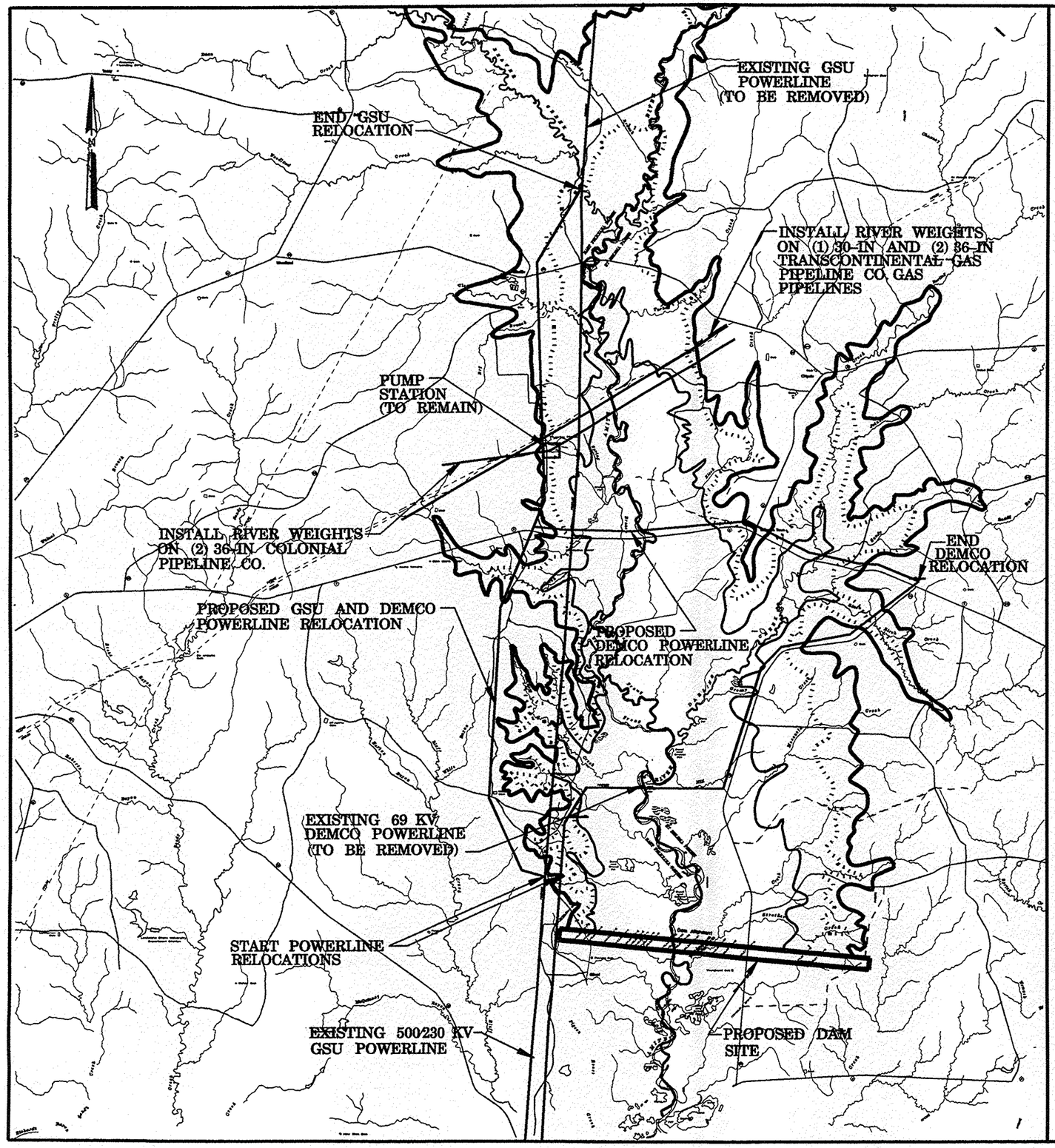
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY

WET RESERVOIR ROADWAY RELOCATIONS

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE: DARESV1.DGN	
DRAWN BY: L. M. P.			FILE NO.	
CHECKED BY: F. H.		DATE: AUGUST 1992		

PLATE 31



LEGEND

PROPOSED MAXIMUM FLOOD LEVEL (EL. 207.3) ———

50 YEAR FLOOD LEVEL (EL. 186.9) .....

PROPOSED NORMAL LAKE LEVEL (EL. 171.0) ———

SCALE IN MILES

1 ½ 0 MILE

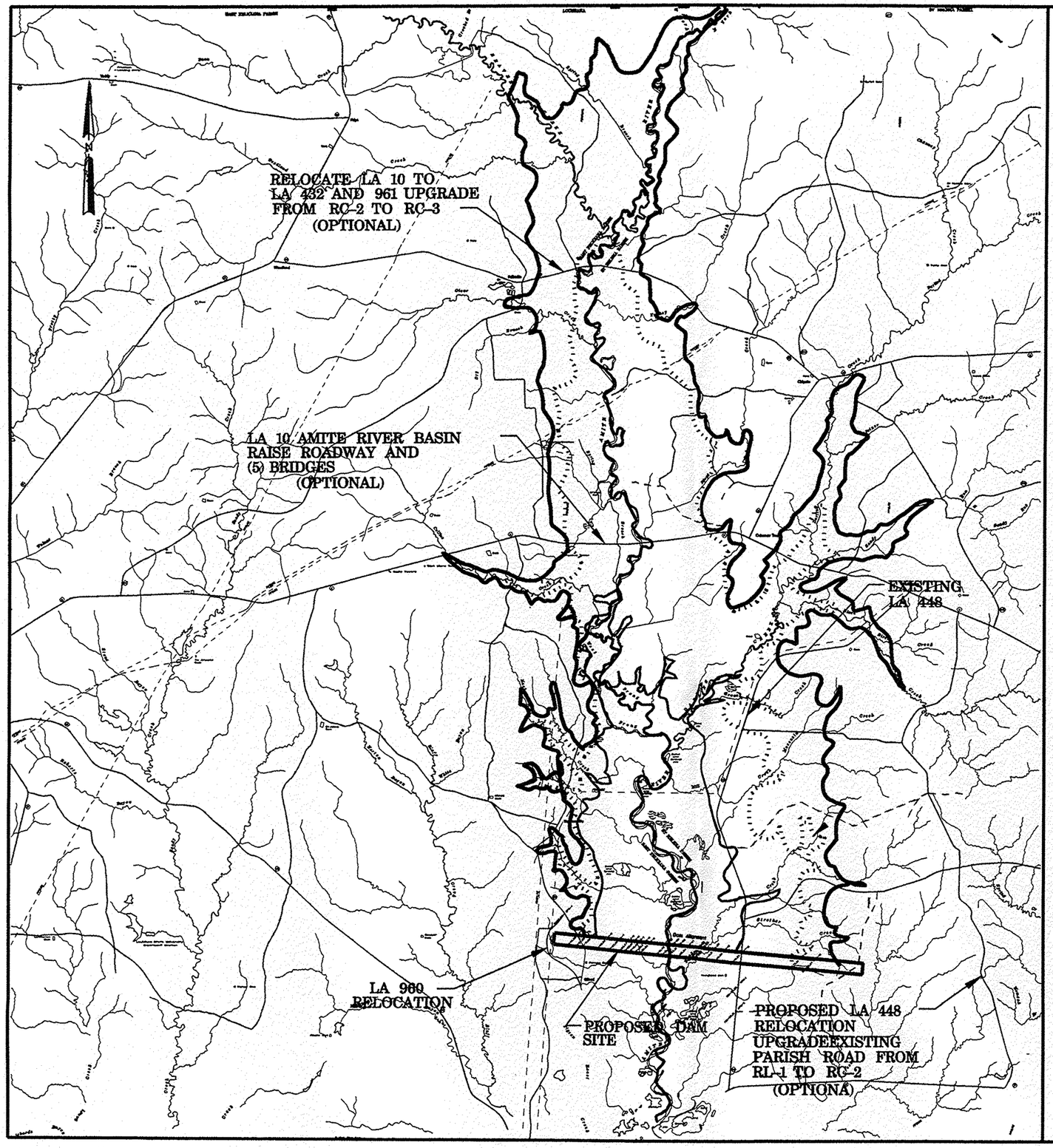
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY

WET RESERVOIR MAJOR UTILITY RELOCATIONS

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE: DARESV2.DGN	
DRAWN BY: L. M. P.			FILE NO.	
CHECKED BY: F. H.	DATE: AUGUST 1992			

PLATE 32



LEGEND

PROPOSED MAXIMUM FLOOD LEVEL (EL. 198.5) ———

50 YEAR FLOOD LEVEL (EL. 176.9) .....

PROPOSED NORMAL LAKE LEVEL (EL. 150.0) ———

SCALE IN MILES

1 ½ 0 1 MILE

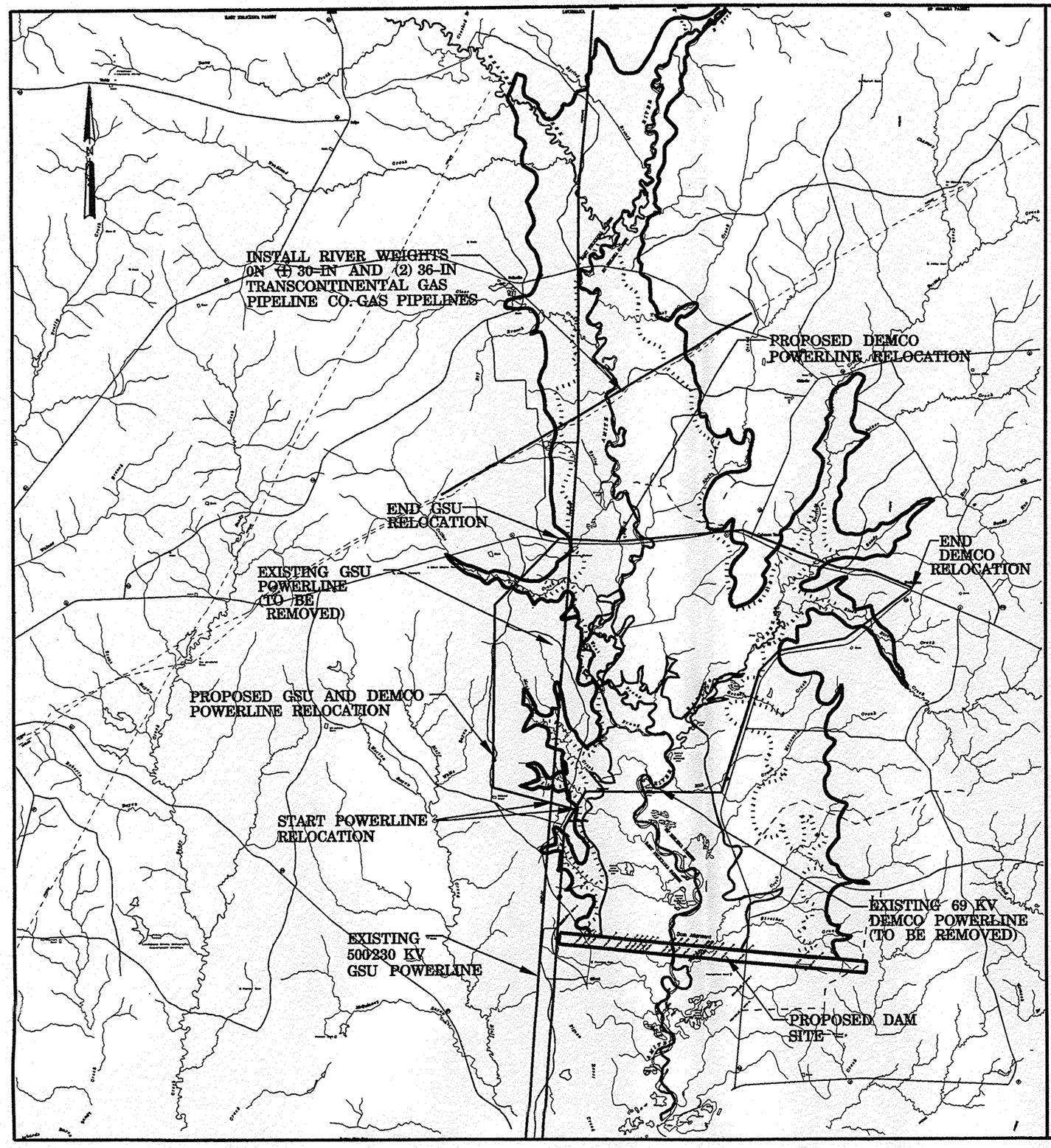
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY

REDUCED WET RESERVOIR ROADWAY RELOCATIONS

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE: DARESV3.DGN	
DRAWN BY: L. M. P.			FILE NO.	
CHECKED BY: F. H.	DATE: AUGUST 1992			

PLATE 33



LEGEND

PROPOSED MAXIMUM FLOOD LEVEL (EL. 198.5) ———

50 YEAR FLOOD LEVEL (EL. 176.9) .....

PROPOSED NORMAL LAKE LEVEL (EL. 150.0) ———

SCALE IN MILES

1 ½ 0 1 MILE

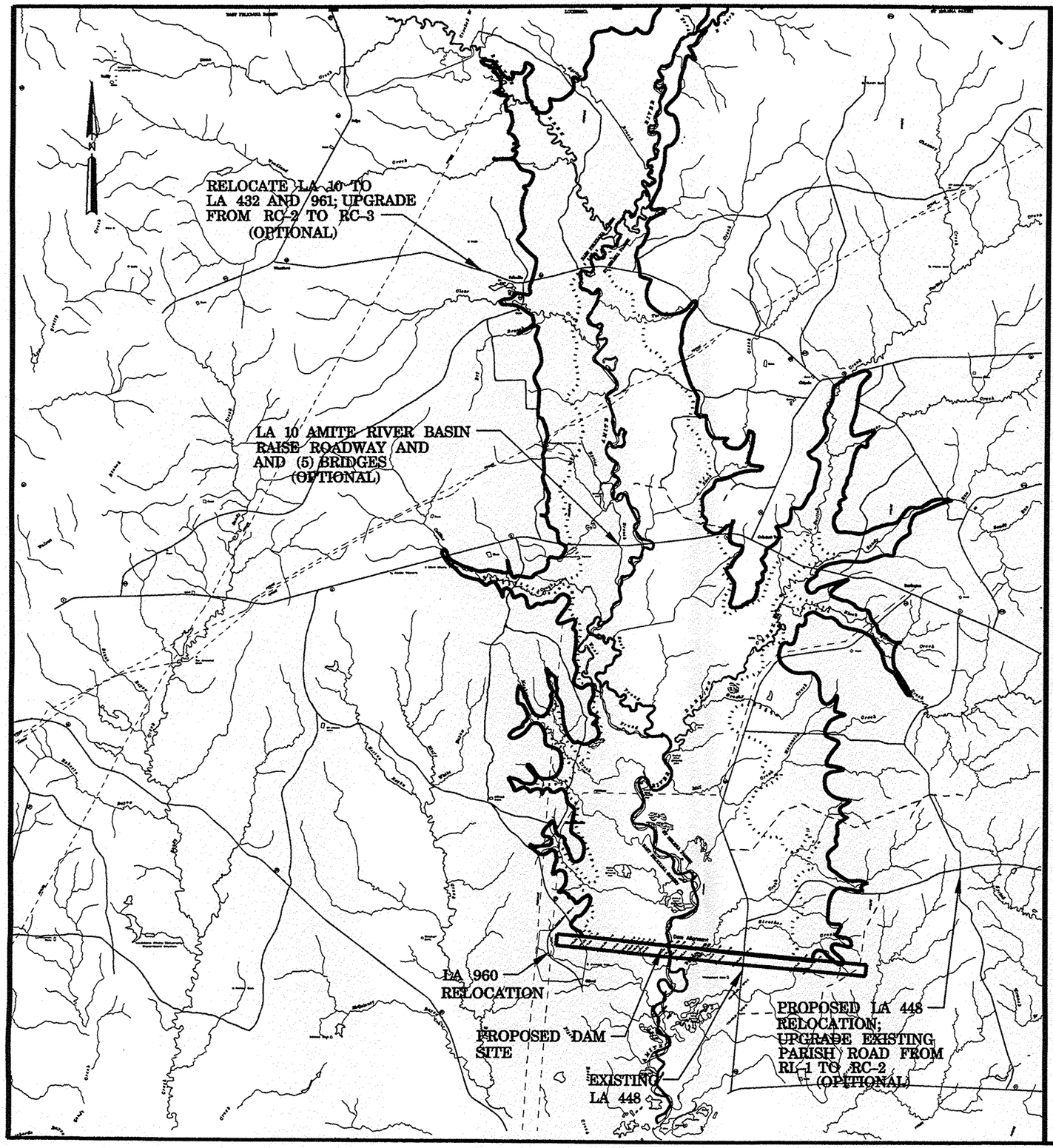
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY

REDUCED WET RESERVOIR MAJOR UTILITY RELOCATIONS

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

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DRAWN BY: L. M. P.			FILE NO.	
CHECKED BY: F. H.	DATE: AUGUST 1992			

PLATE 34



LEGEND

PROPOSED MAXIMUM
FLOOD LEVEL
(EL. 197.0) ———————

50 YEAR
FLOOD LEVEL
(EL. 175.3) .....

SCALE IN MILES

1 ½ 0 MILE

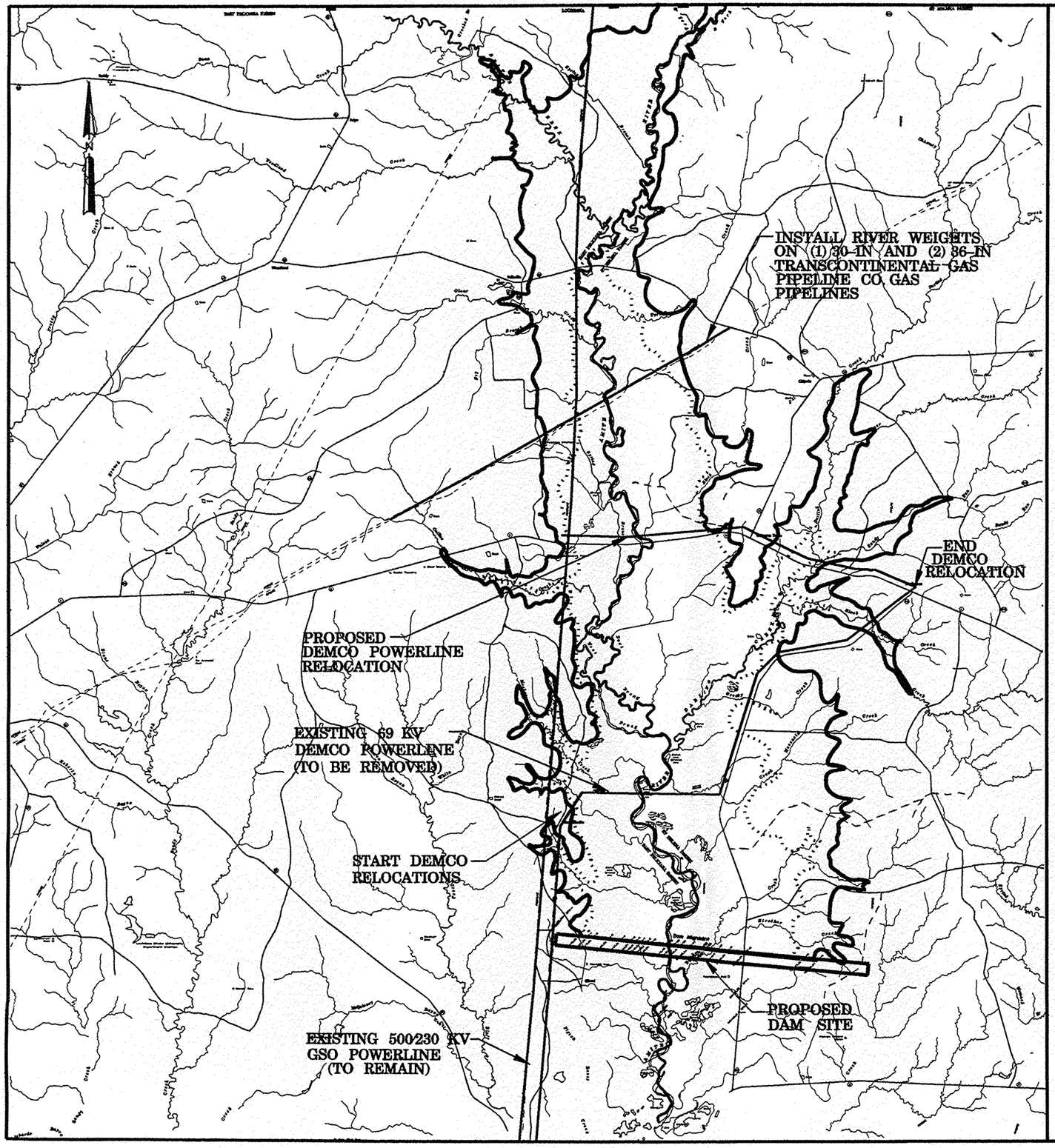
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY

DRY RESERVOIR
ROADWAY RELOCATIONS

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE: DARESVS.DGN
DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		

PLATE 35



LEGEND

PROPOSED MAXIMUM FLOOD LEVEL (EL. 197.0) ———

50 YEAR FLOOD LEVEL (EL. 175.3) .....

SCALE IN MILES

1 ½ 0 1 MILE

AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY

DRY RESERVOIR MAJOR UTILITY RELOCATIONS

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE: DARESV6.DGN	
DRAWN BY: L. M. P.			FILE NO.	
CHECKED BY: F. H.	DATE: AUGUST 1992			

PLATE 36

ENVIRONMENTAL MITIGATION AREA

AREA LOCATION

VICINITY MAP

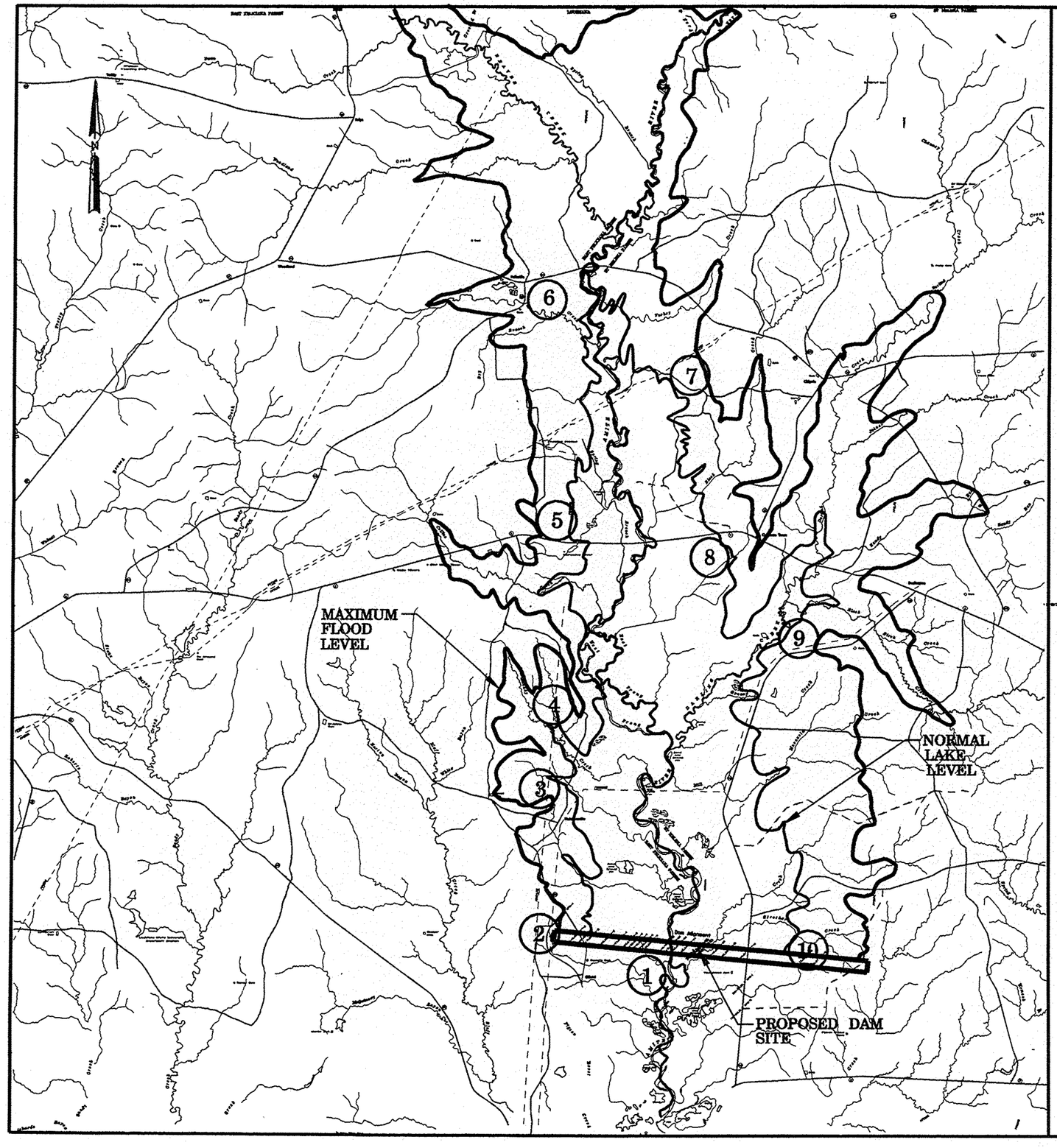
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

ENVIRONMENTAL MITIGATION AREA

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

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DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		

PLATE 37



SCALE IN MILES

1 ½ 0 1 MILE

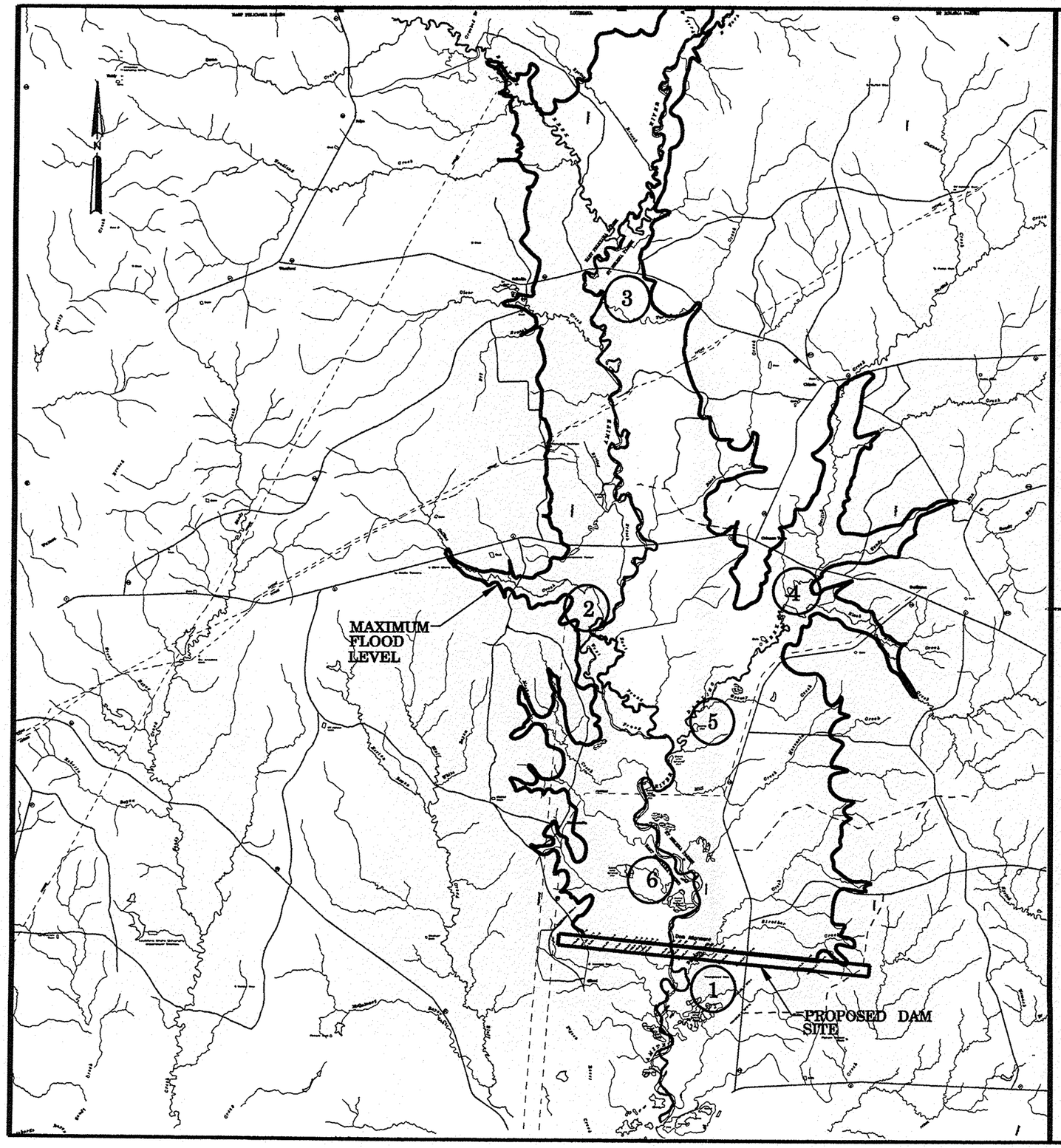
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY

WET RESERVOIR
CONCEPTUAL RECREATION SITES

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE: DARESV7.DGN	
DRAWN BY: L. M. P.			FILE NO.	
CHECKED BY: F. H.	DATE: AUGUST 1992			

PLATE 38



SCALE IN MILES

1 ½ 0 1 MILE

AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR FEASIBILITY STUDY

DRY RESERVOIR
CONCEPTUAL RECREATION SITES

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE: DARESV8.DGN
DRAWN BY: L. M. P.			FILE NO.
CHECKED BY: F. H.	DATE: AUGUST 1992		

PLATE 39

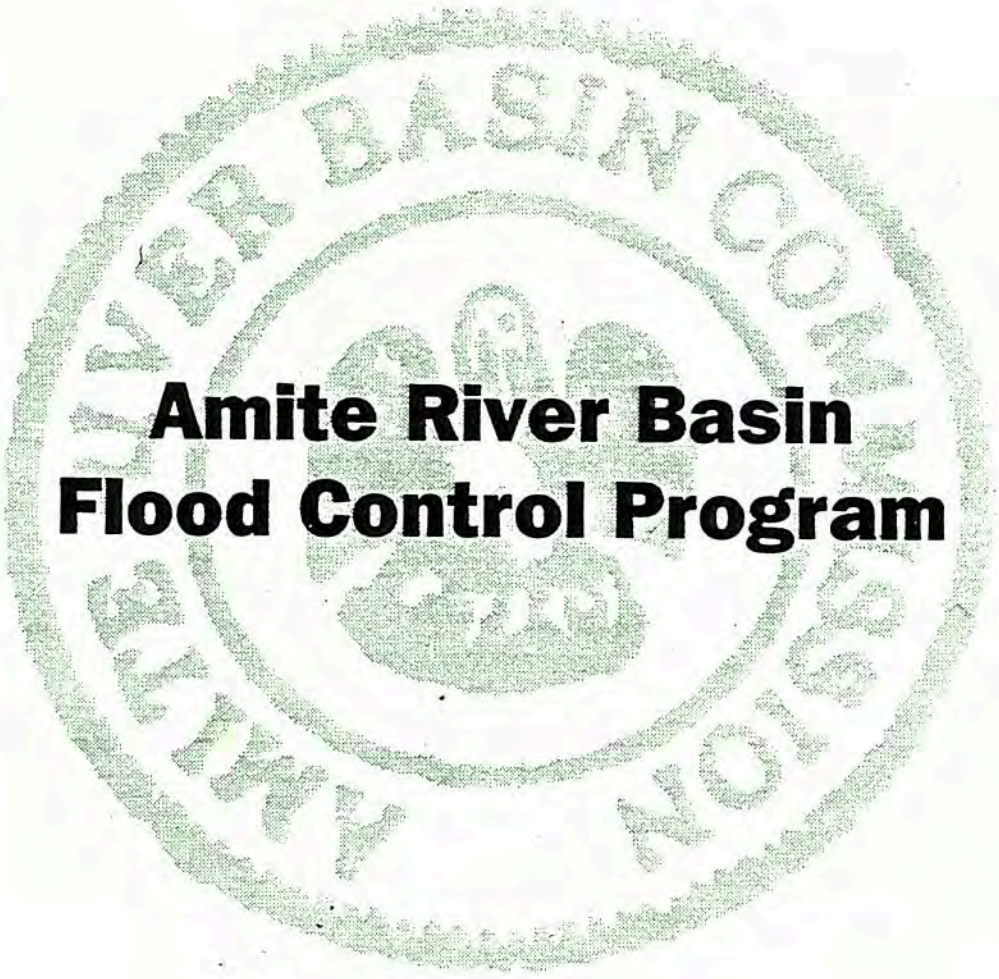
APPENDIX B

HARZA ENGINEERING REPORT, APRIL 1995

Prepared for

Amite River Basin Drainage and Water Conservation District

Baton Rouge, LA



Amite River Basin Flood Control Program

HARZA Engineering Company
Chicago, Illinois
in association with
Brown & Butler, Inc.
Baton Rouge, Louisiana

April 1995

HARZA Consulting Engineers and Scientists

April 18, 1995

Mr. Ross B. Atkins, Executive Director
Amite River Basin Drainage and Water Conservation District
207 Florida Boulevard
Baton Rouge, LA 70801-1719

Subject: Amite River Basin Flood Control Program
Report

Dear Mr. Atkins:

Thank you for the opportunity to present our report on the Amite River basin flood control program. This report presents the results of Harza's review of previous studies and our recommendation for a future course of action. Our analyses were performed under a contract with the Amite River Basin Drainage and Water Conservation District dated December 8, 1994.

Background

In February 1985, the U.S. Government and the State of Louisiana entered into a cost agreement contract for the Amite River and Tributaries Study. The contract identified several major flood control projects:

Basinwide protection:	Comite River Diversion Canal Darlington Reservoir (added to agreement at request of Governor Roemer in 1990)
East Baton Rouge Parish:	Monte Sano Bayou, Jones Creek, Bayou Fountain, Clay Cut Bayou, Hurricane Creek, nonstructural measures
Livingston Parish:	Grays Creek, effects of roads and bridges on restricting flood flows, nonstructural measures
Ascension Parish:	New River, Black Bayou, Bayou Narcisse, and Bayou Francois, Sorrento, nonstructural measures

To date, the Darlington Reservoir study has been suspended pending State response to the Corps of Engineers Draft Feasibility Report. The Comite River Diversion Canal is in the Design Memorandum stage. The East Baton Rouge Parish and Livingston Parish

Sears Tower 233 South Wacker Drive Chicago, Illinois 60606-6392
Tel: (312) 831-3000 Fax: (312) 831-3999 Telex: 25-3540

projects are in the feasibility stage, while the Ascension Parish project has been terminated due to ongoing parish flood control projects.

The projects proposed by the Corps and local entities have been studied on a project-by-project basis when, for efficient management of the entire basin, a basinwide planning approach should be used. This approach will maximize the protection of citizens and property against the damaging floods in the Amite River basin.

Harza's recommendations for a flood control program are based primarily on projects included in the existing contract between the state and federal governments. Our approach included evaluation of the impacts of each project, a judgment as to the efficiency of each project, and the development of a program for basinwide protection.

Impact of the Proposed Developments

The Comite Diversion will provide flood protection for the residents of the Comite basin and, to a lesser extent, the Amite basin. As such, it is an excellent component of any flood control program for the Amite Basin.

The East Baton Rouge Parish (EBRP) tributaries project primarily provides protection to local areas within the parish resulting from headwater flooding. It consists of a series of channel improvements that will contain certain flood flows within the stream channels of five watercourses and reduce the overbank flooding for larger floods. For the most part, the reduction in damages afforded by the EBRP tributaries project can only be realized by this project.

The Ascension Parish project also is more of a localized protection project. It consists of a levee, pumps, and ring dikes that protect the Gonzales/Sorrento area behind the levee from the 100-year flood in the Amite River. Although interior drainage improvements have yet to be completed, the levee is essentially complete.

The Livingston Parish project, consisting of channel improvements in Grays Creek, will provide local relief from headwater flooding. It will function in conjunction with the proposed Miller Diversion Canal.

There are three major conclusions that can be drawn after a consideration of the existing (Ascension) and proposed (Comite, EBRP, Livingston Parish) projects:

1. After construction of the four projects, a portion of the Amite basin subject to damages would remain unprotected.

2. The impact of the Ascension, East Baton Rouge, and Livingston Parish projects on the remainder of the lower basin will be to increase flooding and damages.

3. Additional modeling of the entire Amite River basin would be beneficial to determine the effects of individual flood control projects with and without a basinwide flood control project.

Clearly, additional protection is required. A reasonable solution is to construct a major storage facility on the Amite River to provide main-stem protection in conjunction with the Comite River Diversion Canal. A storage project in the upper basin would provide major benefits:

- It would protect areas not covered by the Comite diversion and the local projects
- It would alleviate negative impacts that might result from the construction of the local projects
- It probably would eliminate the need for the Livingston Parish project or, at a minimum, offer opportunities for reducing the cost of that project.

With this concept in mind, attention was turned to the Corps' evaluation of Darlington Reservoir with the intention of confirming or questioning their conclusion regarding the reservoir's costs and benefits.

Review of the Corps' Darlington Dam Study

The assumptions and design criteria for all of the major cost elements were reviewed and subjected to a cost and value update with the intention of reassessing the economic feasibility of the project. Special emphasis was given to the dam and appurtenant works, mitigation costs, real estate costs, relocations, and recreation costs. It is the conclusion of this review that major cost reductions can be realized for the concept as proposed by the Corps such that the economic justification would be positive. In addition, it is possible that other structural configurations may result in even further cost savings.

Dam and Appurtenant Works

The Corps' layout of the dam section includes all of the features that we consider reasonable and prudent for a design. However, the Corps suggested that the dam section have very flat slopes to provide additional factors of safety under construction and certain operating conditions. The base width of the dam in the Amite channel section was about 1,400 feet. It is Harza's opinion that a section with steeper slopes will provide the requisite safety. For comparison purposes, the equivalent width of dam for the Harza section would be 500 feet. Depending on the height of dam, the reduction in embankment quantity would be about 40 percent.

The length of the outlet works depends on the dam section as the outlet must convey water from upstream of the dam to a location downstream of the dam. With the changed embankment section, the length of the outlet conduit would be reduced by half.

There was not enough information to make any judgement with respect to the layout or cost of the spillway. The crest length was optimized on the basis of a least-cost combination of spillway and surcharge storage constrained by a maximum flow in the river. Based on the revised embankment section, the cost of raising the dam to provide surcharge storage is much less and, therefore, the optimum length of the spillway crest would be less, resulting in a net reduction in cost. We expect that preliminary design studies will result in significant reductions in the cost of the spillway.

Updated Costs

The Corps' cost estimates were based on an evaluation of historical data or estimating procedures at the planning level. As a part of Harza's contract, we developed a preliminary estimate of unit prices for the major cost items in the dam structure, at a 1995 price level.

The original prices for all materials except semi-compacted random and fully-compacted clay were generally accepted and changed to reflect 1995 conditions. However, for the random and clay materials, it is Harza's opinion, based on bids, active projects, and other sources, that these unit prices could be reduced by about 40 percent. In all cases, the contingency percentage used by the Corps, which for most items was 30 percent, was maintained.

To provide an indication of the expected cost reductions for the dam structure, the following tabulation presents a comparison of costs for a dam with a crest elevation at 201 feet:

Corps estimate	$173 million
Revised estimate	$121 million

The difference would be substantially more for a higher dam and slightly less for a lower dam.

Dam Height

As a part of our review, three items contributing to the selection of the height of dam have been identified as being overly conservative. All three conservative assumptions have resulted in an increase in the dam height. First, the antecedent conditions for the routing of the probable maximum flood (PMF) were more severe than our understanding of the Corps' requirements. Second, the PMF may be about 25 percent too high, and third, the wind speed used for calculating freeboard might be reduced slightly. The total effect would be to reduce the height of dam by about 10 feet. This adjustment was not included in our cost analysis because the validity and impact of the revised assumptions could not be established in the scope of this study. We expect that the Corps, in response to State comments, will reevaluate these assumptions.

Real Estate Costs

An attempt was made to confirm or update the real estate costs based on the collection of recent land sales information in the appropriate parishes. It was concluded that the data collected were insufficient to revise the Corps' estimates, and therefore the Corps' estimates were indexed by an inflation factor and used in the analysis.

Roadway and Utility Relocations

In our analysis of the relocation costs, it is concluded that there is approximate agreement in the cost for relocating the roads. For the pipelines, electrical transmission and distribution lines, and phone lines we believe that relocation costs can be reduced by about 33 percent.

Environmental Mitigation

The environmental mitigation costs include a component for the purchase of replacement wetlands and aquatic habitat and for the construction of fences to protect the lands and reforestation.

Our analysis served to reduce the mitigation costs for the full-size wet reservoir by about 50 percent. The costs savings resulted from a reduction in required lands due to recent logging of bottomland hardwoods and establishment by the wet reservoir of new wetlands and hardwoods not considered in the Corps' study. In addition, we have estimated the cost of the replacement wetlands to be about 25 percent lower than originally estimated. The cost of fencing was eliminated to reflect our position that such fencing is overly conservative. It is expected that some other form of management might better serve the purpose of protecting wildlife habitat, and an amount equivalent to a portion of the fencing cost was included.

Recreation

Recreation costs appear to be very high. Unfortunately, no benchmark data are available to us other than estimates for individual recreation facilities. The aggregation of those individual items with estimates for infrastructure led us to reduce the reported recreation costs by 25 percent for the analysis.

In addition, we believe that the reduced wet reservoir condition for recreation, as defined by the Corps, would result in an average depth of water for recreation that is too low. For our evaluation, the average depth for reduced wet reservoirs was increased from five to ten feet.

Hydropower

The region currently is in an energy surplus situation which results in low benefits for additional power development even though retail sales tariffs are moderate to high. As a result, under current conditions, development of a hydro installation with the intention of selling the power and energy to a local utility will not result in significant surplus revenue that could be used to repay project bonds. However, a major power user, such as a city or a large industrial complex, could develop the hydro to offset some or all of their electricity costs. Benefits could accrue to the owner of the project from the purchase, by the user, of the power right. For our analysis, we did not claim any surplus revenue for bond repayment from the potential installation of a hydro project.

Water Supply

There is no question that Darlington Reservoir could provide a major quantity of potable water. In addition, it is becoming increasingly apparent that the Mississippi River is becoming less acceptable as a raw water source for potable supplies and that a new source will be required. Project water could be transported in the Amite River and

withdrawn at any location for domestic and industrial supply. The supply of water from the reservoir could be accomplished with the project as formulated for flood control and recreation. While we recognize this potential, we did not include water supply benefits in our analysis.

Summary

The cost of Darlington dam and its appurtenant structures has been overstated. Based on our technical analysis, the cost of reservoir can be reduced from 30 to 50 percent. Additional reductions are likely for the dam structure. The quantification of those reductions will require revisiting some of the basic assumptions and criteria with the Corps of Engineers.

Recommended Flood Control Program

The program for basinwide flood control considers 1995 conditions and the efficiency with which the proposed elements solve existing problems.

The Ascension Parish project has been essentially completed. It provides local protection, but a basinwide model should be developed to determine how this local project complements the overall flood control program.

The EBRP tributaries project appears to be economically justified by a substantial margin and, in addition, protects areas that cannot be protected by any other identified project. It is recommended that this project be implemented.

The Comite Diversion is far along in its development stage and should not be delayed.

We recommend that Darlington Reservoir be considered next. There are several reasons why we are optimistic that it is a viable federal project:

1. Our preliminary analysis indicates that a 25-year reduced wet reservoir is ecomonically justified.

2. There are more cost savings to be realized for the project, specifically in the spillway, the mitigation costs, and the recreation costs.

3. There will be more damages in the unprotected portions of the basin as a result of the construction of the only-localized projects with the intention of transferring water to another area in the same watershed.

4. There are additional benefits that have not been considered in the Corps study, including reduction in damage to floodplain structures built since 1987, land-use intensification and location benefits, reduced business losses during the flood and clean-up period, and employment of unemployed people during construction and operation.

A second option is to implement the reservoir project on a state level, foregoing federal assistance. Under a regional or state set of criteria, regional benefits that cannot be considered on the federal level can be counted. Under this condition, the reservoir project clearly is justified.

Our basinwide recommendation for construction of flood control projects would result in most of the Amite River basin being protected. However, there will remain small areas that do not receive adequate protection. For those areas, we recommend that local measures such as local ordinances, slab raising, public awareness, and other nonstructural measures be implemented or continued. Careful cost analysis should be made on any localized project.

The Next Steps

Based on our recommended program, the Commission, together with the state, should provide the Corps with the state's comments on the Corps' Darlington Reservoir study. The comments should ask for a review of their assumptions and design criteria.

The Amite River Basin Drainage and Water Conservation District should work with the Corps so that basic input such as benefits, real estate costs, mitigation requirements, relocation requirements, and dam safety requirements can be agreed to as the study proceeds.

As a part of the benefit analysis, a hydraulic model should be developed that shows the effect on the basin of any development so that "with and without" conditions can be assessed and "what if" questions can be answered.

Conclusion

It is Harza's opinion that the opportunity exists to formulate a justified federally-funded reservoir project that will be more effective in controling flooding in the basin than the Comite Diversion and a series of parish projects. In addition, the recreational potential associated with a freshwater lake is very significant and could serve as a major economic driver for the area. There are so many options for improving the viability of the reservoir

project that the state, with the assistance of the Corps, should continue to pursue the project.

We sincerely appreciate working with you toward improving the quality of life in the Amite River watershed. If we can be of further assistance, we are at your service.

Very truly yours,

James E. Trawinski
Regional Manager

rho
Enclosure: As noted

AMITE RIVER BASIN FLOOD CONTROL PROGRAM

Foreword

Authorization

The engineering services associated with the development of a flood control program for the Amite River Basin were provided under a contract with the Amite River Basin Drainage and Water Conservation District dated December 8, 1994.

Purpose and Scope

The purpose of the services was to recommend a comprehensive program for providing flood control in the Amite River Basin. The program is primarily based on the content and results of previous studies.

The scope of the services included:

- discussions with key agencies
- collection and review of previous studies and recommendations
- field reconnaissance
- design review of the proposed wet-reservoir Darlington Dam Project
- investigation of adding water supply and hydropower to the reservoir project
- update of costs and benefits of various projects using indexes
- assessment of significant environmental effects, particularly endangered species
- formulation of results from the above tasks into a basinwide flood control program.

Visits to the basin and discussions with various interested parties, in addition to Amite River Basin Commission and Brown & Butler staff, were performed by:

- Michael Saunders, Harza's Project Manager, on December 12-13, 1994 and Norman Schickedanz, Lead Engineer, on December 12-15, 1994 for meetings with the ARBC public works committee, ARBC President Jeff

David, Mayor Floyd Younger of French Settlement, the East Baton Rouge public works department, and the New Orleans Corps of Engineers

- John Chitty, Environmental Scientist, on January 16-20, 1995 for an inspection of the reservoir area and discussions with the Corps of Engineers and U.S. Fish and Wildlife Service

- James Trawinski, Regional Manager, on January 17-18, 1995 to review the costs of highway and utility relocations.

- Michael Saunders, Project Manager, and James Trawinski, Regional Manager, on March 21, 1995 to discuss preliminary findings and request specific information about flood damage reduction benefit analyses with the New Orleans Corps of Engineers.

Study Participants and Acknowledgments

The Amite River Basin Drainage and Conservation District, known as the Amite River Basin Commission, or ARBC, is the state agency responsible for flood control in the basin. Its President Jeff David, Executive Director Ross Atkins, and Assistant Director Dietmar Rietschier provided special guidance and assistance during the study, as did Charles Costner of Brown & Butler, Inc.

The staff of other agencies also provided data and assistance, particularly:

Jerome Klier, Chief Engineer, and Bryan Harmon, Drainage Engineer - Department of Public Works, Baton Rouge
Domingo Elguezabal, Elizabeth Cottone, Falcolm Hull, Frank Vicidomina, and Charles Shadie - New Orleans District, Corps of Engineers
Floyd Younger - Mayor, French Settlement.

Principal participants of the Harza staff were:

Michael Saunders - Project Manager
Norman Schickedanz - Lead Engineer and Hydrologist
Philippe Martin - Geotechnical Engineer
John Chitty - Environmental Scientist
David Sulkowski - Power and Water Supply Planning Engineer
Rick Suttle - Recreation Planner
David Schuy - Resource Economist
James Trawinski - Regional Manager.

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AMITE RIVER BASIN FLOOD CONTROL PROGRAM

Chapter I

AMITE RIVER BASIN FLOOD PROBLEM

The Amite River drains about 2000 mi^2 of Louisiana and Mississippi in the Baton Rouge area. In recent decades, flooding has caused extensive damage to the thousands of structures in the 100-year floodplain of the basin. Numerous structural and non-structural flood control measures have been studied over the years including a main-stem reservoir project that was evaluated by the Corps of Engineers Some of the measures have been or are being implemented.. Private, local, state, and federal entities have been involved in the previous studies.

The Basin

Location

The Amite River basin drains about 2000 mi^2 of southwest Mississippi and southeast Louisiana. The drainage basin is shown on Exhibit 1. In Louisiana, the basin includes, from north to south and east to west, sizable portions of East Feliciana and St. Helena, East Baton Rouge and Livingston, and Ascension Parishes, and small portions of Iberville, St. James, and St. John the Baptist Parishes. The 170-mi-long Amite River and its major western tributary, the Comite River, flow southerly to their confluence east of Baton Rouge and south of Denham Springs. The Amite continues southeasterly, receiving water from Bayou Manchac on the west side, through flat lowlands below Port Vincent, to Lake Maurepas (Corps, 1955, 1993).

Urban Centers

The major urban center in the basin is Baton Rouge, with a population of about 400,000 people (the city itself had about 220,000 people in 1990). Other towns include Zachary (about 9000 people) and Baker (13,000 people) to the north of Baton Rouge, Denham Springs (8000 people) to the east, and Gonzales (7000 people) to the southeast (Corps, 1993, and atlas census populations). The total population in 1990 in Ascension, East Baton Rouge, East Feliciana, Livingston, and St. Helena Parishes was about 540,000 (Corps, 1992).

Topography

The upper basin consists of gently rolling hills, where elevations vary from about 500 feet to 80 feet (mean sea level). In the lower basin, the elevations vary from about 35 feet

and 40 feet near the Amite-Comite junction to elevations of 5 feet to 10 feet just upstream of Lake Maurepas. The lake, itself, lies at sea level. The meandering Amite River runs relatively swift, clear, and shallow over a sand and gravel bottom above Denham Springs. Downstream of Denham Springs, the river is sluggish, due to the much flatter slope (Mitigation Assistance, 1993 and Corps, 1993) and expanses of wetlands and swamps (Chamber of Commerce, 1991).

Drainage Areas

Significant tributaries to the Amite River are Beaver, Darling, and Bluff Creeks, Wind Bayou, the Comite River, Jones Creek, Clay Cut Bayou, Bayou Manchac, Henderson Bayou, and Colyell Creek.

Drainage areas at various locations in the basin have been reported as follows:

Amite River at U.S. Geological Survey (USGS) streamflow gage at State Highway 10	580 square miles
Amite River at the Darlington damsite	692 square miles
Comite River basin	355 square miles
Amite River at U.S. Geological Survey (USGS) streamflow gage near Denham Springs	1,280 square miles
Amite River at U.S. Geological Survey (USGS) streamflow gage at State Highway 42	1,596 square miles
Amite River at Lake Maurepas	1,820 square miles

Climate

Normal annual precipitation at Baton Rouge airport is 56 inches, although for the period 1980-91 rainfall averaged 66 inches/year. Normal annual precipitation at various stations in the basin ranges from 56 to 67 inches. Prevailing winds are from the south much of the year (Corps, 1992, 1993).

Storms that have caused heavy rainfall show similar characteristics. Typically, major winter storms (generally from December to April) produce heavy rainfall for three to five days as the result of cold fronts from the north nearly stationary over southeast Louisiana displacing warm, humid air from the Gulf of Mexico (Corps, 1992, 1993).

Land Use

Pine forests exist in the northern reaches of the basin and bottomland hardwoods occur in the lower, flatter portions. In 1985, land use in the basin was 72 percent forest and agriculture in the basin. However, in the watersheds of the East Baton Rouge Parish tributaries, urbanization varied from about 30 to 75 percent in 1985 and is projected to reach 40-97 percent by 2040 (Corps, 1993). Urban and built-up lands increased from 1 percent of the basin in 1954 to 12 percent in 1985. Urban land in the 100-year floodplains of East Baton Rouge and Livingston Parishes tripled from 17 mi^2 in 1972 to 52 mi^2 in 1985. Substantial urban growth is expected to occur in East Baton Rouge, Livingston, Ascension, and Iberville Parishes. Water, wetlands, and barren land have been relatively constant making up about 16 percent of the total land use (Corps, 1992).

Environment

The Amite River is used for recreation, the propagation of fish and wildlife, and, to a lesser extent, for water supply, navigation, and waste disposal.

The existing aquatic ecology of the Amite River is primarily a result of the existing flow regime (Brown & Butler, 1987). The river is a meandering, erosional stream which continually changes course, preventing the development of aquatic vegetation, benthos, and productive backwaters (Espey, Huston, 1989). The combination of the river's sand substrate, high flood flows, and sand and gravel mining also contribute to the present low productivity of aquatic resources.

The indicated input of organic matter to the Amite River from out-of-bank flooding (Corps, 1992) does not benefit the upper reaches of the river. Flood flows tend to impoverish the upper reaches of the river by scouring the river bed, banks, and adjacent riparian lands, flushing productive organic matter and sediments downstream. The scouring effect is accentuated by the existing sand and gravel mining, which causes upstream head-cutting. Head-cutting has resulted in erosion of the existing river banks and areas of productive substrates. The continual changing of the Amite River's channel and the ongoing head-cutting eliminate most backwater areas. The head-cutting also affects the foundation and abutments of bridges and other structures along the river bank. Evidence of erosion in the Amite River is seen by the large number of fallen trees and eroding banks.

The organic matter is flushed downstream by the flood flows as suspended sediment and debris. The coarser sediments are the first to settle out downstream, where the stream gradient is less and flow velocities are reduced. Sediment deposition also may have an adverse effect on Lake Maurepas, where river sediments likely form an alluvial plume covering the lake bottom substrates.

Flood Impacts

Causes

The low-lying areas along the Amite and Comite Rivers have commonly experienced flooding during periods of heavy rainfalls. Flooding in the basin is caused by excessive rainfall leading to headwater or overbank flooding of various streams and to backwater overflow in the lower reaches of the Amite and the tributary streams in the vicinity of their confluence with the Amite River. Headwater flooding occurs when the drainage system cannot hold all the runoff from upstream or local rain. Backwater flooding occurs when a high downstream water level forces upstream water to back up behind it (Wilbur Smith, 1991).

Historically, most flooding was confined to swampland and sparsely populated, largely wooded areas. In the past 40 years, though, urban development has expanded into the floodplain. The 100-year floodplain, for example, is determined by the 100-year flood, a statistical event. A structure in the 100-year floodplain has a 1 percent or greater chance of flooding in any given year. In Baton Rouge, 29 of the 33 residential subdivisions built between 1970 and 1984 were built almost entirely within the 100-year flood zone (Chamber of Commerce, 1986). Presently, over 16,000 structures lie within the 100-year floodplain (Corps, 1992). In East Baton Rouge Parish alone, there are about 5,700 houses in the 100-year floodplains of creeks and bayous tributary to the Amite River (Corps, 1993). East Baton Rouge Parish covers 470 mi^2, of which about 42 percent lies in the 100-year floodplain.

Streams in East Baton Rouge, Livingston, and east Ascension Parishes occasionally are unable to drain the local areas due to high stages in the lower Amite River and Bayou Manchac (Chamber of Commerce, 1991). Wind and tidal effects on Lake Maurepas contribute to the backwater flooding; for example, in October 1985, Hurricane Juan produced tides in Lake Maurepas six to eight feet higher than normal (Corps, 1992). Flooding is exacerbated by decreased carrying capacity of the river channel due to mining operations and sedimentation (Mitigation Assistance, 1993).

Due to the travel time of water from the upper Amite and Comite basins, there is time for parish runoff to move out before the upstream flood water arrives. However, if runoff is accompanied by backwater flooding from the Comite and lower Amite, significant damage can occur, according to East Baton Rouge Parish Public Works.

According to the Horizon plan for East Baton Rouge Parish (Wilbur Smith, 1991), the existing drainage system requires better maintenance. According to the Parish Department of Public Works, clearing of canals in old areas is a problem due to lack of "servitude" for access; in newly developed areas, access is available.

Sedimentation in the lower Amite River between River Miles 30 (Colyell Creek) and 40

(4 mi north of Bayou Manchac) has caused increases in flood levels of up to three feet over the original channel improvements of 1961-2. Also, significant sedimentation has occurred in the lower Amite River channel between the diversion weir and Lake Maurepas since construction of the diversion channel (Harza, 1994).

Frequency

Exhibit 2 lists the maximum annual flood peaks at the Darlington and Denham Springs gages for their periods of record. The 1994 values, based on data from the East Baton Rouge Parish Department of Public Works, are preliminary. Table 1 lists the water years of the top ten peaks of record at each gage (a water year starts September 1st of the previous calendar year).

Table 1

OCCURRENCE OF TOP TEN FLOOD PEAKS

USGS gage	Darlington	Denham Springs
Drainage area, mi^2	580	1,280
Period of record	1949-94	1921, 1939-94
Years	46	57
Rank	Year	Year
1	1990	1983
2	1977	1977
3	1994	1990
4	1983	1921
5	1973	1993
6	1955	1979
7	1980	1953
8	1972	1980
9	1964	1973
10	1965	1994

Seven of the top ten flood peaks at Darlington and eight of the top ten at Denham Springs have occurred since 1972. Records for various streams across southeast Louisiana show a similar pattern of higher floods starting with the floods of 1972 and 1973; regional climatic water-budget analysis of annual moisture surplus for the years 1889-1989 shows a distinct pattern of runs of wetter and drier years, with the years 1973-83 being the wettest (Johnston, 1991).

Flood-frequency analyses of the annual peaks on Exhibit 2 indicate that a flood expected to occur, on the average, once in 100 years (100-year flood peak) would have a peak flow

of 118,000 cfs at Darlington and 133,000 cfs at Denham Springs. The 1990 flood of record at Darlington represented a 60-year peak and the 1983 flood of record at Denham Springs about a 45-year peak.

Damages

Table 2 lists some information characterizing historical flood damages (Corps, 1955, 1992, 1993; Mitigation Assistance, 1993). Other floods undoubtedly caused damage, but figures are not readily available for them. For comparison, original-damage dollars were ratioed to current dollars using the all-urban consumer price index, U.S. city average. The results are approximate.

Table 2

HISTORICAL FLOOD DAMAGES

Year	Storm Rainfall, inches.	Acres Flooded	Structures Flooded	Damage, $ million	
				Original	Current Dollars
1953	7 to 17			0.74	4
1962	4 to 7	114,000			
1973	6 to 8	105,000		2	7
1977	6 to 15	164,000	Over 4,000	78[a]	190[a]
1979	9 to 13			9	19
1983	5 to 13	357,000	5,500	172[b]	260[b]
1989	7 to 10	Bayou Fountain 2 ft above '83			
1990	4 to 6		Over 700	50	57
1993	7 to 14				

[a] Without business, agriculture, and infrastructure losses.
[b] Of which originally $48 million, currently $72 million, in Comite basin.

Flood problems by parish are briefly discussed below (Corps, 1992). Most of the damage in the April 1983 flood occurred in the cities of Baton Rouge in East Baton Rouge Parish and Denham Springs, Port Vincent, and French Settlement in Livingston Parish (Chamber of Commerce, 1986).

- East Feliciana and St. Helena Parishes and Mississippi Counties - Flooded lands are mostly wooded or, to a lesser extent, agricultural. Past damages have been minor.

- East Baton Rouge Parish - Flooding is caused by a variety of sources including headwater overflow from the Comite and tributaries, backwater from Hurricane Creek, and the lower Comite, headwater overflow from the

Amite and tributaries, and backwater from Bayou Manchac and the Amite. In 1983, 1,600 residences and businesses were flooded; damages totaled $65 million. About three-fourths of the damage occurred in the Comite basin. About 10,000 ac of agricultural land were inundated, but the fields were fallow. According to East Baton Rouge Parish Public Works, the main damage occurs on Blackwater, Beaver, and Claycut Bayous and Hurricane Creek; most parish damages are residential except along Hurricane Creek where industrial and commercial damages also occur.

- Livingston Parish - Flooding is caused by headwater overflow of the Amite and two tributaries. Backwater overflows occur along several creeks and the Blind River. In 1983, 3,000 residences and businesses were flooded; damages totaled $84 million.

- Ascension Parish - Flooding is caused by headwater flooding of the Amite and tributaries and backwater flooding of their lower reaches. In 1983, 800 residences and businesses were flooded; damages totaled $20 million.

- Iberville, St. James, and St. John the Baptist Parishes - Flooding is caused primarily as a result of backwater overflow from bayous. Much of the areas flooded are wetlands. In 1983, damages totaled $0.2 million.

The public's memory is short. Local real estate agents estimated that the 1983 flood depressed the market value of flooded homes only 10-15 percent and only for the period immediately after the flood. Most of the damaged buildings were repaired and regained their market value (Chamber of Commerce, 1986).

Flood Management Activities

Previous Studies and Reports

The Corps published a flood control study of the Amite River and its tributaries in 1955 (Corps, 1955). As a result, the construction of a diversion channel from the Amite River to the Blind River and channel improvements in the Amite and Comite Rivers were implemented during 1958-63 (Corps, 1963).

The Department of Transportation and Development (DOTD) in 1967 enlarged eight miles of White Bayou north of Zachary. Earlier, in 1956, DOTD had enlarged 2.4 miles of the bayou through Baker. Also, DOTD made channel improvements to Ward Creek during 1953-57 and later in the 1960s to it and Bayou Duplantier. Starting in 1957, the state and Baton Rouge made channel modifications to Clay Cut and Jacks Bayous (Corps, 1993).

The Corps conducted a preliminary evaluation of flood control in the basin in 1972.

They considered four reservoir plans, two diversion plans, and four channel modification plans. All were found to be not economically justified (Corps, 1993).

In the early to mid 1970s, the Corps prepared floodplain information reports for several bayous and creeks in the basin. The reports basically presented flood flow and water surface elevation information about the streams.

From 1973 to 1993, the Corps performed the flood insurance studies for most of the towns and parishes in the basin in Louisiana (Corps, 1992, 1993). Those studies indentified the 100-year floodplains but not regulatory floodways except along the Amite River in East Baton Rouge Parish.

From 1982 to 1990, Baton Rouge city and parish made improvements to Beaver Bayou (Corps, 1993).

A second reconnaisance, "Flood Control, Initial Evaluation Study," was performed by the Corps in 1984 (Corps, 1984). The Corps identified a number of plans including the Darlington Reservoir and the Comite diversion that were economically justified.

In 1984, Brown & Butler, in conjunction with other firms, produced a conceptual study of Darlington Dam for the DOTD. Exhibit 3 shows the location of the recommended facilities. In addition to flood control, the project included recreation and hydropower plus the possibility of water supply. The dam was found to be economically justified (Brown & Butler, 1984).

Also in 1984, Brown & Butler reported on Liberty Lake dam and reservoir upstream of the Darlington site shortly south of the Mississippi border. this project also included recreation and hydropower (Brown & Butler, 1984b).

During 1985-6, Brown & Butler, in conjunction with others, performed a detailed study of the hydrology, hydraulics, reservoir operation, and hydopower potential of Darlington Reservoir for DOTD. The report basically confirmed the findings of the 1984 study (Brown & Butler, 1987).

Espey, Houston & Assoc. studied numerous flood-control schemes for the Amite Basin in 1988-9 and published its report including an EIS for Darlington Dam in 1989. The dam was found to be economically feasible at that time (Espey, Houston, 1989).

DOTD, which had applied to the Corps in 1985 for a Section 404 permit to construct the Darlington Reservoir Project, withdrew the application in 1990 and requested federal participation in the project. That request led to the Corps' 1992 feasibility study of the Darlington Reservoir Project. It was concluded that both dry- and wet-reservoir dams were not economically justified.

The Corps completed a feasibility study and EIS of flood control measures in the Comite basin in 1990. After studying numerous measures, the Corps recommended a 12 mile diversion channel to the Mississippi River. The project is currently undergoing detailed design (Corps, 1993).

In 1990, the Governor's Interagency Task Force on Flood Prevention and Mitigation published a presentation of numerous measures for flood protection and mitigation . Recommendations included nonstructural measures, support of the Comite diversion, and a dry reservoir at Darlington (Johnston, 1990).

In 1991, the Governor's Committee to Study Authority and Powers of the ARBC reported on their analysis, recommending measures to strengthen the commission (Emmer, 1991).

The Horizon land use plan was published in 1991 for East Baton Rouge Parish. Drainage was one of the three elements it addressed (Wilbur Smith, 1991). Recommendations included better maintenance, drainage improvements, public awareness, and securing of drainage rights-of-way.

The Corps completed a draft feasibility study for the East Baton Rouge Parish tributaries in 1993. Numerous structural and non-structural measures were studied. The Corps recommended channel modifications to five watersheds (Corps, 1993).

Existing Improvements and Plans

Structural flood control measures that have been implemented or are in the planning stage include:

- Original Amite Diversion and Channel Improvements. The Amite River diversion channel to the Blind River was constructed and improvements to the Amite and Comite channels were made during the period 1958-63 (Corps, 1963). However, there has been no regular maintenance.

- Beaver Bayou Channel Improvement. The bayou, in East Baton Rouge Parish, was enlarged during 1988-90. About 70 percent of the cost was paid under the Statewide Flood Control Program, and 30 percent was paid by the parish (Corps, 1993).

- Ascension Parish Projects. Based on its 1981 drainage plan, Ascension Parish began a ring-dike and levee system, a pump station, and tributary channel improvements. The project is supported by a 5 mil property tax and a 1/2 cent sales tax.

- Comite Diversion Project. The Corps is performing detailed design of a 12-mile diversion channel to the Mississippi River. The state has sold $0.5 million in bonds for land purchase.

- Slab-House Raising Demonstration Project. Four houses and one office, all on slab, are being raised five to eight feet. Slab sizes range from 1,300 ft^2 to 3,500 ft^2. FEMA has funded about half of the project cost for $277,000. The local share is coming from a variety of sources, including $50,000 from ARBC, $25,000 from the city of Denham Springs, $182,000 from the home owners, $12,000 from the state cooperative extension service, $20,000 from LSU, and $14,000 from engineering companies (Emmer, 1994). The total, $580,000, averages over $100,000 per raising.

Non-structural measures have included the following:

- Flood Insurance. East Baton Rouge Parish entered the flood insurance program in 1979. Also, unincorporated areas of St. Helena, Livingston, Ascension, Iberville, and St. John the Baptist Parishes and about 15 towns, including Zachary, Baker, Denham Springs, Port Vincent, French Settlement, Gonzales, and some towns in East Feliciana and St. James Parishes, have entered the program.

- East Baton Rouge Floodplain Regulations. Since passage of the flood damage prevention ordinance in 1993, the Parish requires that any buildings constructed in the 100-year floodplain to be 2.0 ft above the 100-year flood (base flood) level; buildings outside the floodplain have to be 1.0 ft above the adjacent base flood elevation or record flood level, whichever is higher. In either location, the buildings need to be 1.0 ft above the nearest sewer manhole. Further, the parish requires compensatory storage for any filling in the floodplain.

  Livingston, Ascension, and St. John the Baptist Parish Regulations. These parishes require that any development in the 100-year floodplain have the first floor above the 100-flood elevation (Corps, 1990)

- Amite and Comite Rivers Streamgage Telemetry and Flood Forecasting. The Amite streamgages at Darlington, Grangeville, Magnolia, Denham Springs, and Port Vincent and Comite gages at Olive Branch and Joor Road now have telemetry connections to the National Weather Service River Forecast Center at Slidell. The Center issues flood forecasts to radio and TV stations and newspapers, the state USGS, and the state office of emergency preparedness.

- Public Education and Awareness. Activities have included lectures, publications, videos, and citizen activities:

  Since January 1993, the ARBC executive director, Ross Atkins, has spoken to numerous civic, professional, social, and news organizations about the flood control problems and needs of the basin.

  During 1993-4, ARBC produced and mailed to 100-year floodplain residents in East Baton Rouge Parish, three "Watermarks" newsletters entitled "Can the Flooding Be Stopped?", "Can It Happen Again?", and "Time Is Running Out"--reminding people of the flood potential and solutions.

  A flood tracking chart was produced in 1994, at the instigation of Lawrence Callender, a resident of French Settlement, with the support of the ARBC, USGS, WAFB-TV, and others. Over 10,000 copies have been distributed to the general public. The chart assists residents in determining if they are threatened by a forecasted flood (Emmer, 1994).

  In December 1993, Allan B. Crow and Associates produced a video called "Trouble on the Amite - A Look at Flooding in the Amite River Basin." Lasting about ten minutes, the video examines the impact of the river on the greater Baton Rouge area, the potential for flooding, and possible solutions to the problem. About 75 copies were distributed to legislators, local officials, concerned citizens, and news media. Also, it was shown to numerous civic groups and associations.

  Flood Awareness Week has been an annual event held at the Baton Rouge City Hall since 1992. It comprises a variety of publicity efforts such as displays, panel discussions, brochures, short taped speeches, resolutions, and cable TV talks, involving people from ARBC, NWS, USGS, LSU, and DPW.

Chapter II

POTENTIAL MEASURES

The flood control program will be developed from among projects previously described that are in the partially-complete or planning stages. The structural and non-structural measures are discussed in more detail in the following sections.

Comite Diversion

The purpose of the project is to provide flood protection for the residents of the Comite River basin. It will reduce stages from the diversion point downstream to the Amite River, in the Amite from the junction to Port Vincent, and in Hurricane Creek, Robert Canal, and White's, Cypress, and Baton Rouge Bayous (Corps-LDOTD, 1995). a location map is shown on Exhibit 4. The diversion structure and 12-mile diversion channel will divert about 50-60 percent of flood flow at the diversion point, for example 12,700 cfs during a 10-year flood, to the Mississippi River between Zachary and Baker (Corps, 1990). The project will lower Comite River levels about six feet at White Bayou, four to six feet at Hooper Rd., four to five feet at Comite, four feet at Hurricane Creek, one to one and a half feet at Denham Springs, and 0.6-0.8 feet at Port Vincent (undated fact sheet from ARBC).

Until recently, the diversion project was the only federally-authorized potential project in the basin. The project was authorized for construction by the Water Resources Development Act (WRDA) of 1992. In August 1993, it became apparent that the project cost would exceed the authorized limit, necessitating reauthorization. The federal authorization recently expired. The schedule now is: draft design memorandum (DM) in March 1995, final DM in June, DM approval in December, reauthorization in the 1996 WRDA, project cooperation agreement execution January 1997, real estate acquisition thereafter, start of construction October 1998, and completion September 2000 (Corps-LDOTD, 1995). The final feasibility study and EIS were completed in 1990 (Corps, 1990).

The non-federal cost includes a mandatory 5-percent cash contribution, purchase of all land, easements, rights-of-way, and relocation and disposal areas, and 50 percent of the cost of recreation facilities. Some relocations will be done with federal funds (Corps-LDOTD, 1995).

The state DOTD is the local sponsor. It will finance the state's share of the capital-outlay cost for construction through bond sales. Bonds in the amount of $0.5 million, for the purchase of land for the canal, have already been sold.

According to a recent Corps-LDOTD project fact sheet (1995), the estimated total cost of the Project is $121.8 million ($60.9 million local) in 1994 dollars, resulting in an expected benefit-cost ratio of about 1.05.

The 1990 Environmental Impact Statement does not mention any significant environmental problems. Construction will result in the loss of 435 acres of bottomland hardwood forest and 29 acres of upland hardwood forest. Short-term impact of increased turbidity and temperature and decreased dissolved oxygen will not be significant on aquatic organisms because of the short duration. The project will not affect the existing populations or habitat of the Alabama heelsplitter mussel, Potamilus inflatus, in the Amite River (Hartfield, 1988, 1989).

East Baton Rouge Parish Tributaries Projects

The Corps is presently studying a flood control project involving channel improvements for tributaries of the Amite River in East Baton Rouge Parish (Exhibit 5). Protection would be provided against the 10-year flood in Blackwater and Fountain Bayous, the 25-year flood in Beaver Bayou, and the 50-year flood in Jones Creek and 3 tributaries, and the number of structures in the floodplain of Ward Creek and 2 tributaries would be reduced (Corps-LDOTD, 1995). A draft feasibility report was published in August 1993. A revised draft was released in March 1995.

A sales-tax increase to fund the state's 30-percent share of the Project was defeated in the November 1994 election; the improvments had been packaged as part of the broader land-use Horizon plan. The plan included cleaning or improving channels and gaining servitude for access. Meeting with the Corps after the election, the mayor of Baton Rouge said the parish will pursue the project with funds from elsewhere; various sources are being looked at, e.g. bonds or part of gaming proceeds. The city-parish government has signed a letter of intent to sponsor and cost-share the project.

The feasibility study is scheduled for completion in June. Subsequent activities include authorization in the 1996 Water Resources Development Act, a project cooperation agreement in late 1996, and construction construction in 2005 (Corps-LDOTD, 1995). The project total cost is $143 million ($36 million local), average annual benefit $24.4 million, and the benefit-cost ratio 2.2 (Corps-LDOTD, 1995).

An independent study commissioned by the parish is looking at the feasibility of side-channel overflow reservoirs in sand and gravel pits and at "dry" hydraulic control structures across the river. The initial findings are that these alternatives are not economically justified.

Darlington Reservoir

Darlington reservoir has been identified in several reports as being an appropriate solution to a major part of the flooding problem in the Amite River basin. In 1984, Brown & Butler, in their report titled "Proposed Darlington Reservoir," concluded that the project was economically justified. In 1989, Espey, Huston & Associates, in their report titled "Engineering Report, Amite River Flood Control Study," also concluded that the Darlington reservoir project was economically justified. In 1992, the Corps concluded in their report on Darlington reservoir that the project was not economically justified to the extent that it would return only about $0.44-0.60 for every $1.00 spent.

As a part of Harza's evaluation to identify a comprehensive flood control program, it became apparent that Darlington reservoir could, if economically justified, be an important component of any program. Therefore, the various aspects of the Corps' analysis of Darlington reservoir were reviewed. The review was performed with the intent of identifying areas in which major cost reductions might be implemented. The review consisted of evaluating the assumptions and data presented in the 1992 report, field inspections of the site and vicinity, and re-analysis of the design parameters where appropriate to identify an order of magnitude cost.

Dam Structure. The estimates of the cost of the dam by Brown & Butler, Espey Huston, and the Corps vary dramatically as follows:

Brown & Butler	Crest elevation 200	$50+ million
Espey, Huston	Crest elevation 202	$40+ million
Corps	Crest elevation 204	$190+ million

The Corps reported that their calculated costs and benefits were generally comparable to the earlier studies with the exception of the dam structure itself. They further reported that the dam cost increased mainly because results of additional soil borings indicated a more extensive dam section. The cost of both the spillway and the outlet works also was significantly increased from earlier studies without any detailed explanation.

The configuration of the embankment section, therefore, was the primary focus of Harza's evaluation. The embankment configuration not only impacts the cost of the dam but also the optimization of the spillway size and the length of the outlet works.

Our review of the local geotechnical conditions was based on the reported results of the two drilling programs. During the first program (1983), three 100-ft-deep borings were taken along the dam axis. During the second program (1991), seven more holes were drilled along the dam axis. Both programs provided a consistent rendering of the foundation conditions under the dam axis.

Based on the results of the field investigations, Harza concludes that:

1. The top 100 ft of the foundation consists entirely of alluvial formations, from recently deposited materials to clays of the Upper Miocene period.

2. The prevalence of sands under most of the dam alignment will require measures to reduce underseepage.

3. The water retention capability of the reservoir will be enhanced by the present of a surficial layer of clay and silt found at the top of every hole drilled at the site.

4. The foundation materials offer competent bearing for an earth embankment.

The Corps reported a dam section with very flat slopes, especially across the Amite River channel and near-channel reaches. The section is shown on Exhibit 6. In the river channel, the base width is about 1400 ft. The stability analysis performed by the Corps indicates that the controlling factor for the selection of an embankment configuration is not the foundation, as might be implied by the their statement regarding the need for a more extensive section as a result of the additional soil borings. The embankment configuration was based on the estimated low factors of safety within the section during construction and the rapid drawdown operating condition. As stated in the Corps' report, the construction case governed the selection of the dam cross-section in all cases.

It is Harza's opinion that a dam with an upstream slope of 3 horizontal to 1 vertical and a downstream slope of 2.5 horizontal to 1 vertical would be naturally stable under long-term conditions and provide an acceptable margin of safety required during construction and rapid drawdown. To attain this degree of safety, proper compaction specifications, construction control, and implementation of design details enhancing drainage in the outer portions of the dam would be required. All of these requirements are a part of the normal process in the design and construction of an embankment dam.

The impact on cost of this change in section was evaluated for a dam with its crest at El. 211.5, the highest of the three sections for which the Corps provided estimates. For the revised section, the quantity of random material would be reduced by about one-half, and the cost of the dam materials, using unit costs selected by the Corps, would be reduced by about $55 million.

The prior investigations indicated that the foundation is characterized by a considerable thickness of granular materials. It was also recognized that the reservoir is located in a groundwater recharge area for downstream wells. These factors require that definite seepage control measures be installed in the foundation under the dam. Harza concurs

with the Corps' proposal to install a 70-ft-deep positive slurry trench cut-off under the entire dam alignment.

Outlet Works and Spillway. A change in the section of the dam has a major impact on the length of the outlet works and possibly on the length and configuration of the spillway.

The outlet works, as presented by the Corps, is 1,120 ft long excluding the stilling basin. With the revised section, the length can be reduced by about one-half to about 560 feet. This reduction would result in a savings of about $3 million in the cost of the conduit.

The crest length of the spillway was constrained by the need to prevent downstream flow conditions worse than might occur under PMF flows without a dam. Shorter spillway lengths were rejected because the cost of raising the dam to provide more surcharge storage was more than the savings from shortening the length of the spillway. With the revised section, it is expected that a shorter spillway would result in the optimum condition.

An estimate of cost of the spillway was provided in the Corps report for a dam to El. 201; however, the drawing included with the report was not suitable for review. Both of the prior reports (Brown & Butler and Espey, Houston) included an emergency spillway with a length of 1000 feet, so it is assumed that the Corps accepted the configuration.

As an exercise, a spillway with a surcharge eight feet greater than proposed by the Corps was evaluated. The exercise resulted in a net cost reduction of $15 million after accounting for the shorter spillway crest and the higher dam.

Construction Cost. The Corps' cost estimates were based on an evaluation of historical data or abbreviated estimating procedures. In addition, a single spillway cost and outlet works cost were used for all dam development levels. We assume that if any of the Corps' options were nearly feasible, they would have estimated the costs at the feasibility level. Because of the intense interest in this project, for subsequent analysis, a more rigorous determination of unit prices is recommended.

Harza reviewed the unit prices reported in the Corps' report and estimated unit prices representative of a 1995 price level. For the dam structure, it is Harza's opinion, based on recent experience, that the embankment unit prices are high except for the gravel filter's which is low. For excavation and concrete, the prices were generally appropriate.

Dam structure costs were re-estimated using updated unit prices, revised embankment quantities, and Corps quantity estimates for the remainder of the items. In addition, the cost of the outlet works was reduced by $3 million and the spillway by $15 million. The Corps' contingency, which for most items was 30 percent, was maintained. The engineering cost for design and construction management was reduced from 12 percent

to 10 percent. A curve of dam structure cost versus dam crest elevation is presented on Exhibit 7.

The costs presented on Exhibit 7 are very approximate. No attempt was made to optimize any of the structural facilities. It is expected that the represented costs are high because the spillway cost may be substantially reduced. The costs are sufficiently accurate to substantiate a recommendation to revisit the design of Darlington dam.

Real Estate Costs. An attempt was made to confirm or update the real estate costs identified in the Corp report. Recent land sales information in East Feliciana and St. Helena Parishes was collected and categorized by land classification. The average values estimated using this recent information are considered to be unrepresentative for several reasons: 1) In most cases, the sample size is too small; 2) for residential and woodland, the per-acre cost range was too large, varying from $240 to $6000 for residential and $554 to $1669 for woodland; and 3) a local land assessor suggested that the Corps estimates were high for lands in the reservoir area.

For the present analysis, it was decided to inflate the Corps' numbers at a rate of three percent for two years. For any subsequent studies, it is recommended that a registered land assessor be retained to estimate the value of the specific lands under consideration.

For information, the Corps values, the average values estimated as a result of the recent information, and the values used in the analysis are presented in Table 3.

Table 3

REAL ESTATE VALUES

Land Classification	Corps Estimate 1992	Land Sales Information No. Parcels	Avg. Value	Used
Pasture/agricultural	$1,100	2	$2,650	$1,170
Woodland	$ 900	10	$1,117	$ 960
Residential	$1,800	5	$2,739	$1,910
Sand & gravel pits	$2,000	1	$1,096	$2,120
Ag lands to be used for wetlands mitigation	$910	None	$ 500	$ 700

Relocation Costs for Highways and Utilities. For the full wet reservoir, the Corps included roadway and utility relocations costs for 2 roads, 1 bridge, 5 pipelines, and about 50 miles of various transmission, distribution, and phone lines. To keep those costs to a minimum, the Corps excluded all optional costs.

A review of the scope and cost of the relocations was performed as a part of this study. Distances and elevations were taken from USGS quadrangle maps. Costs were estimated based on discussions with Entergy and DEMCO for transmission line relocations, a materials supplier and contractor for the pipeline weighting, and recent estimates for highways LA 557 and LA 505 performed by Brown & Butler.

A revised estimate was developed for the 50-year development using Corps estimates where there was agreement and revised estimates where there was significant disagreement. The estimates for the 50-year wet condition were prorated to other levels of development. In addition to the changes in direct costs, the allowance for engineering and administration was reduced from 22 percent to 15 percent. The relocation costs used in the analysis and the Corps' 1992 estimates for the 50-year full wet development are shown in Table 4.

Table 4

RELOCATION COST ESTIMATES
($1,000)

Item	Corps (1992)	Revised
LA 960	1,254	800
LA 10	4,148	4,086
E. Feliciana Parish Road 42	593	593
Colonial Pipeline	3,050	3,606
Transcontinental Gas Pipeline	6,469	2,850
Shell Pipeline	-0-	882
EXXON Pipeline	-0-	70
Gulf State Utilities	15,719	4,980
DEMCO	1,728	3,616
Other power and phone	435	435
Total	33,396	21,920

The primary conclusion drawn from this analysis is that there is substantial disagreement in the pricing of these measures and room for substantial reductions in the cost of relocations.

Hydrology. Three items appear to be overly conservative and thus have led to the selection of a a dam height that is somewhat too high. First, the Corps' PMF peak is extremely high and is suspected to be reducible by about one-fourth. Second, the probable maximum flood (PMF), which is the spillway design flood, was routed through

the reservoir starting at the top of the flood-protection pool, e.g. peak 50-year-flood elevation. The starting elevation is overly conservative and not required, according to our understanding, by Corps regulations. Third, the wind speed used for calculating freeboard on top of the PMF maximum surcharge might be reduced slightly. In total, the dam height may be reducible by approximately 10 ft.

The Corps' PMF peak of 556,000 cfs for the 692-mi^2 basin is equivalent to a Creager C of 168, which is typical of the Caribbean with its hurricane rainfall and mountainous slopes. Espey, Houston (1989) calculated a PMF peak of 492,000 cfs. According to the Nuclear Regulatory Commission (1977), nine other PMFs estimated for sites in the region have Creager C's of 52-111, with a median of 79, and the envelope flood peak, which is very conservative in other parts of the country, for the location of the Amite basin is about 215,000 cfs. The peak at Darlington should be untypically high because the Amite and two main tributaries with roughly equal drainage areas enter the reservoir together. Thus, the Creager C is judged to be 130, about 2/3rds above the median, giving a peak of 430,000 cfs. Routing this flood would result in a reduction of the maiximum flood pool elevation of three to four feet.

The Corps started its PMF routing with the reservoir at the top of the flood control pool. However, regulations (Corps, 1991) allow starting at the top of the flood control pool or at the elevation occurring five days after the end of antecedent precipitation equal to about half the main storm. Since the Corps' PMF routing shows the reservoir drops back to its starting elevation four days after the end of probable maximum precipitation (PMP), the reservoir would drop back to the starting level within five days after the end of half the PMP. In other words, even with an antecent flood of half the PMF, the reservoir would not be surcharged at the start of the PMF. Thus, the PMF routing can be started at maximum normal pool, the top of the conservation pool, which is 13-18 ft lower for the large wet reservoir schemes.

The Corps' wind speed concurrent with the maximum PMF pool appears to be a 10-year wind speed. A two-year wind speed should be acceptable (American Nuclear Society, 1981). About 20 percent less, it would reduce the wave run-up and freeboard about 1 ft.

Hardwoods and Other Woodland. Bottomland hardwoods are mentioned by the Corps 1992 feasibility report as a significant resource. The Espey, Huston environmental report (1989) indicates that the full wet reservoir (20,150 acres including flood surcharge area) will inundate about 8,230 ac of bottomland hardwood and that the dry reservoir will inundate about 500 acres. A timber inventory and appraisal of the proposed reservoir area (17,000 acres) by Bennett and Peters, Inc. in 1988 stated the fair market value of timber resources as $5,320,000. These resources can be harvested before inundation by the reservoir and not count as a cost of construction. Analysis of the evaluation procedures in the Corps study indicate that the beneficial effects of the proposed wet reservoir were not included in the analysis.

Analysis of recent 1993 aerial photos indicates that approximately 450 acres of bottomland hardwood forest within the proposed full wet reservoir area have been logged or converted by sand and gravel mining since the environmental report was produced by Espey, Houston in 1989. This will reduce the total area of woodland impacted by the full wet reservoir from 8,230 acres to about 7,780 acres.

The development of new areas of bottomland hardwood and other wetland and woodland types along the shoreline of the wet reservoir would mitigate some of the losses of bottomland hardwood and wetland vegetation types flooded by the reservoir. The establishment of about 1,500 acres of wetland and woodland vegetation types, as indicated below, will reduce the area of impacted woodland to about 6,280 acres.

Many of the areas of bottomland hardwood and other woodland types were observed to have undergone high grading, where the most commercially-valuable trees, primarily oak, have been removed. High grading changes the species mix of the forest, leaving lower value tree species as the dominant stock. Such activity can permanently lower the value of the remaining forest for timber and wildlife habitat. Significant to wildlife is the reduction of mast producing trees, a major food source.

Areas of wetlands will naturally develop along the shoreline of the full wet reservoir. The type and extent of wetland development will vary according to water depth and the frequency of flooding caused by flood retention. Based on a conservation pool elevation of 171 feet (Corps, 1992) and the expected periodic inundation for flood storage, wetlands will most likely devlop in an elevation band between 169 feet and 175 feet. Other factors, such as soils, wave action, slope, and frequency of saturation/flooding will also affect location, extent, and type of wetland that will develop. Forested wetland may develop anywhere within the estimated band. Marsh wetlands will most likely develop from two feet below to one foot above the normal pool, i.e. Els. 169 to 172. Woody brush wetlands will develop along the reservoir up to two feet above the pool, i.e. Els. 171 to 173. Considering that much of this area is presently covered with forest types, roughly 1,500 acres may be available to convert from open lands (grassland, sand and gravel mining, etc.) to new woodland wetland types. As part of mitigation, clearing and planting will accelerate the development of woodland along the wet reservoir shore.

Endangered Species. As indicated by the 1989 Espey, Huston environmental report, no state or federally endangered species of flora or fauna will be seriously impacted by the proposed reservoir. Later studies by state and federal agencies have confirmed that species indicated in the environmental report will not be seriously affected by the reservoir.

Of particular concern has been the Alabama heelsplitter mussel, P. inflatus. Identified habitat and known locations in the Amite River will not be impacted by the proposed reservoir area or by the reservoir's downstream flow regime (USF&W, per. comm., 1995). Although P. inflatus has been known to occur 17 miles downstream from the

proposed dam, its range in the past extended much farther upstream, to Interstate 10. However, over the past 20 to 30 years, land use, sand and gravel mining, head-cutting, and flood flows have removed mussel habitat from the upper reaches of the Amite River (Espey, Huston, 1989, Appendix D.5; USF&W, Hartfield, 1995 per. comm.)

A major impact on P. inflatus is the on-going sand and gravel mining, which directly removes the substrate. Upstream of the actual mining areas, head-cutting continues the removal of potential mussel habitat. Proposed project mitigation actions (acquiring streambank lands below the proposed damsite and the reduction of erosive flood flows by the regulated releases of the dam) may allow P. inflatus to re-establish itself. Areas upstream of existing mussel locations and previously lost to the sand and gravel mining may stabilize through the effects of the proposed dam. Acquisitions of streambank lands below the dam will concentrate in areas where mussel habitat could be restored. Project mitigation may provide the means to introduce P. inflatus to specific areas with the hope that the introductions would spread and reclaim past habitat areas.

Environmental Mitigation. The potential impact and mitigation concerns of the proposed reservoir, expressed in the 1992 Corps report, overlook benefits of the reservoir. Those benefits refer to the reservoir's impact on existing factors adversely affecting the ecological productivity of the Amite River. The present land use and physical characteristics of the river are the major factors adversely affecting the aquatic ecology and productivity.

With the full wet reservoir, about 18 miles of free-flowing river will be flooded (the dry reservoir will only affect the river channel at the proposed damsite). The benefits of the proposed reservoir to the remaining river channel, upstream and downstream of the reservoir, offset much of this impact. Added mitigation to preclude land use in specific areas of critical habitat will also replace lost wildlife habitat.

The full wet reservoir will reduce the effect of detrimental factors on the aquatic ecology by modifying the flow regime. Upstream of the proposed wet reservoir, river erosion, primarily the head-cutting, will be greatly reduced. Organic matter, presently flushed downstream, will deposit in the reservoir, creating productive areas for fish spawning and wildlife nesting. Periodic flooding of the wet reservoir will create wetlands and cause deposition of organic sediments which will enrich the soil.

Downstream of the wet reservoir, scouring due to peak flows will be reduced. Bank erosion and channel movement also will be reduced, allowing development of biologically productive bank vegetation and backwater areas. Organic matter will be contributed to the river more evenly, providing for the build-up of more productive substrates.

Acquisition and fencing of 38 mi of river bank (Corps, 1992) for mitigation of the full wet reservoir is most likely impractical. The probable high cost of maintenance of the

fence and local use pressures (sand and gravel mining and public access) will make it difficult to improve the riverine habitat for the entire reach. A more logical approach may be to concentrate mitigation in the downstream areas of remaining critical habitat and where sand and gravel mining has not had a significant effect. Critical habitats are areas of existing high quality with remnant populations of endangered wildlife and vegetative resources.

Of major importance are the remaining habitat areas and specific locations of known populations of the Alabama heelsplitter mussel, P. inflatus. The 1989 Espey, Houston environmental report and later surveys indicate locations from the vicinity of Watson, upstream of Denham Springs, down to about the state route 42 bridge south of Port Vincent. Some of that river reach has been affected by sand and gravel mining and other activities which have reduced or eliminated mussel habitat. The reduction in flood flows by the proposed reservoir favors restoration of mussel habitat. Land acquisition and protective management of remaining high quality mussel habitat areas as part of mitigation may stop the shrinkage of the area inhabited by P. inflatus and may provide the means for habitat restoration. Recommended mitigation includes acquisition of about 1500 ac of riverbank area between the proposed damsite and the vicinity of Port Vincent.

Based on the above analysis, mitigation costs were developed for the full wet reservoir following the same procedure used in the 1992 Corps feasibility study. Although the habitat suitability index (HSI) of woodland is likely less because of high grading, an HSI of 0.65 was still used to be conservative. Tables 43 and 49 in the Corps report were modified and are presented on Exhibits 8 and 9 to reflect cost adjustments in impacts and needed mitigation.

Recreation. The primary users of recreation resources in the study area live in southern Louisiana, and their primary recreation activities are hunting and fishing. Most fishing occurs by boat. There are 71 boat launching ramps in the study area (Corps, 1992).

In 1980, approximately 2.5 million people lived in Louisiana within 100 mi of the Darlington dam site. For them, there is a distinct shortage of freshwater recreation opportunities. The 3000-ac Catahoula Lake, about 90 mi from Baton Rouge and 180 mi from New Orleans, is the largest freshwater lake available for recreation in the area. There is only one state park, Quin, with a 700-ac reservoir, within 150 mi of the Darlington site which has resort-type facilities; it drew about 430,000 visits in fiscal year 1988-9 (Espey, Houston, 1989).

A brief review of the existing recreational facilities in the Amite River basin shows there is a need for a freshwater recreation facility near the proposed reservoir. The wet reservoir could provide many of those needs. Given the large nearby population and long distance to other facilities, a wet reservoir with a stable water surface and successful fishery would be an attractive recreational resort. Both tangible and intangible benefits

would accrue. For example, savings in travel cost is a valid benefit. Also, facilities used to capacity are not affected by recreation users going elsewhere. Further, some users will go more often to a closer, more desirable facility, causing a net increase in recreation spending.

Hydropower. A review of the basic data and analyses for the evaluation of hydroelectric at the proposed Darlington dam was made. The data and the analyses were given in the January 1987 report by Brown & Butler and the January 1984 report by Brown & Butler and Tudor Engineering Company. The analyses were adequate in addressing the relative merits of adding a hydroelectric facility. The hydro facility was treated as an incidental component, after "optimizing" the flood control, which is the primary function of the reservoir.

The selected powerplant layout appears appropriate for this level of study. The layout included a single unit, surface powerhouse arranged in a tubular configuration. The turbine was a horizontal Kaplan type with adjustable blades and moveable wicket gates rated at 9.0 megawatts (MW). The power conduit consisted of an 11-ft by 11-ft reinforced concrete conduit branching from the most eastern low-level outlet conduit. The draft tubes consisted of two 9-ft by 13-ft concrete conduits exiting at the downstream end of the stilling basin.

The selected generation capacity was 9.0 MW with a rated net head of 57 feet and rated turbine discharge of 2,090 cfs. The rated discharge corresponded to about 13 percent on the flow-duration curve. Therefore the installed capacity may be high and could be reduced to about 7.0 MW or lower without a significant drop in energy production. At 7.0 MW, the rated discharge would be about 1,600 cfs corresponding to 19 percent exceedance on the flow-duration curve. A lower rating will allow operation at a higher efficiency during the low-flow months.

Power and energy analyses were reviewed. The average annual energy was estimated to be 37 million kWh (kilowatt-hours). Independent computations indicated an average annual energy of about 44 million kWh, based on an average annual flow of 1,080 cfs, efficiency of 88 percent, and net head of 62 feet. Therefore, the estimate of 37 million kWh/year looks appropriate considering periods of operation at lower efficiencies and some spill.

Operation of the project was basically assumed as run-of-river at a normal maximum reservoir level of El. 181. Higher reservoir levels, up to 5 feet, were temporarily allowed when inflows exceeded maximum turbine discharge capacity. Lower reservoir levels were allowed to meet the minimum turbine flow of 300 cfs. However, a detailed analysis of reservoir operation is needed to confirm energy estimates based on an established reservoir operation rule curve and other operating constraints.

For this study, the construction cost was updated. The updated cost (December 1994 level) is estimated to be $10,200,000 including a contingency allowance of 25 percent and an allowance of 15 percent for engineering and administration. The updated construction cost was independently checked and confirmed using cost curves developed for small hydropower installations.

Economic analyses were performed to compute the benefit/cost (B/C) ratio and net present value (NPV). The following criteria were used:

- A total investment cost was estimated for an annual interest of 10 percent.
- The annual operation and maintenance (O&M) cost was estimated as 1.5 percent of the total investment.
- An annual interest rate of 10 percent was used for computing B/C and NPV.
- A 30-year period of evaluation was used.
- Benefits were computed as the avoided energy cost. No capacity value was included.
- The O&M and energy costs were escalated at an annual rate of three percent.

Based on the above criteria, the total investment cost was estimated at $11.5 million. A range of avoided energy costs from $0.02 to $0.05 per kWh were used in the computation of annual benefits. It is noted that the benefit is not the rate charged to the customer, e.g. homeowner, but rather the avoided cost. An avoided energy cost of about $0.03 per kWh is required to obtain a B/C ratio of 1.0. For that case, the NPV would be about $430,000. However, current energy costs are lower than $0.03 per kWh, varying as follows: $0.009-0.016 at Cajun Electric Power Cooperative, the wholesaler for DEMCO (Dixie Electric Membership); $0.016-0.022 at Louisiana Power & Light; $0.021-0.026 at Southwestern Electric Power; $0.022 at Central Louisiana Electric; and $0.022-0.024 at Gulf States Utilities Co. Those lower avoided costs mean that a 9.0-MW hydropower addition to Darlington Dam is not economically justified under current conditions. Of course, in the future, if avoided costs increase faster than hydro costs at Darlington, power might become economically justified.

Water Supply. There is no question that Darlington Reservoir could provide a major quantity of potable water. In addition, it appears that the Mississippi River is becoming less acceptable as a raw water source for potable supplies and that a new source will be required. If sufficient demand arises, delivery of water from the reservoir might be

economically justified. The supply of water from the reservoir could be implemented with the project as formulated for flood control and recreation.

In 1984, Brown & Butler estimated an average water demand of 175 mgd for the east bank of Orleans and Jefferson Parishes and a 20-year projection of 220 mgd. The current demand thus would be about 190 mgd. A pipeline from Darlington Dam to New Orleans, about 100 miles long, would be very expensive. Alternatively, the water might be transmitted at least part way in the Amite River channel; at the end point, an intake, pump, and shorter pipeline would be needed. In either case, subsequent costs would be incurred to treat and distribute the water to customers. Even so, the amortized monthly cost per customer might be less than the cost of bottled spring water being delivered to families in New Orleans.

An entity interested now in using the water has not been identified. Future effort should be made to identify a potential user.

Economics

The economic feasibility of Darlington Reservoir was analysed using the revised costs and benefits from an national economic development (NED) standpoint to determine if there is a possibility that the project could be economically justified for federal funding.

Evaluation was limited to wet reservoirs in order to provide water-based recreation.

The economic analysis was based on data in the Corps' planning reports adjusted for reasons explained subsequently. The price level for the economic analysis is December 1994. The period selected for analysis is 50 years after completion of construction. The interest rate used in the analysis is 7.75 percent, the rate currently used for the analysis of federal projects. Interest during construction (IDC) was calculated for the alternative reservoir plans assuming the same rate of annual expenditures for the 50-year-flood wet reservoir plans presented in the Espey, Huston report (1989) as summarized in Table 5.

Table 5

ANNUAL EXPENDITURE FLOW FOR DARLINGTON
WET RESERVOIRS
(percent)

Type	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6
Wet	3.0	7.7	10.6	20.2	26.1	32.4

IDC was reduced for the Comite diversion and East Baton Rouge Parish tributary (EBRP) projects using the current 7.75 percent rate.

National Economic Development Benefits. The nature and value of benefits from a water resource project investment vary depending on the perspective of the beneficiary. In order to obtain funding from the U.S. government, the analysis of benefits and costs must be from a national economic development (NED) perspective in conformity with the rules and procedures developed by the US Water Resources Council (1983). Those guidelines define three categories of flood control benefits:

- Inundation Reduction Benefit - Increased income with no change in floodplain land use. The most significant increase in income is measured by the estimated reduction in average annual flood damages under with-project conditions compared to without-project conditions. Other types of inundation reduction benefits are reduced losses of wages or net business profits caused by disruption of economic activity, savings in emergency flood relief costs, and reduced costs for administering floodplain insurance.

- Intensification Benefit - Increased income resulting from more intensive use of the floodplain, without a change in the type of land use. Examples are building additions to homes as a result of project flood protection or using basements as living space.

- Location Benefit - Increased incomes from new activities (change in land use) developed in the floodplain as a result of project flood protection. An example would be the higher value of land (expressed as an annual income flow) resulting from a land use change from agriculture to residential.

An increase in land value observed under post-project conditions is a result of the market expectation of higher income from the land as measured in the above categories. For example, if the flood control project causes a reduced annual average flood damage to a building and its contents of $200, that is the same as saying the land is producing $200 more in annual income, which is capitalized into the market value of the land. If the owner of the land expected a return of 10 percent on land investment, the increase in land value would be $200/0.10 = $2000. Therefore, it is not proper to claim the increase in market value of land and the annual reduced flood damage as project benefits because that would count the same benefit twice. Conceptually, either measure of flood benefit could be used, but not both. As a practical matter, it is more accurate to estimate reductions in flood damages than to predict changes in market land values (Institute, 1991).

If the economic analysis of the project is from another perspective such as state or local, the benefits and costs may be different depending on where they occur. For instance, if recreation development at Darlington reservoir attracts out-of-state visitors, their expenditures will increase local profits and wages, including income multiplier effects.

That is a benefit from the state perspective, but not from the national perspective to the extent increased recreation expenditures in Louisiana are offset by reductions in other states. Analysis of costs and benefits from the state or local perspective under the Water Resources Guidelines is done in a regional economic development (RED) account as discussed subsequently.

The NED flood control benefits in all of the Corps of Engineers studies are based primarily on the estimated reduction in annual flood damages to existing property. The reasons given in support of this position were:

> "...future development would not affect the benefit computations significantly as all future structures must be built at or above the 100-year flood elevation and all project alternative plans would offer protection equal to or less than the 100-year flood" (Corps, 1984).
>
> "Inundation reduction is the only primary category of national economic development (NED) benefits considered, since no significant differences in land or structure value in the study area can be readily observed strictly on the basis of flood risk. Lands and other properties with significantly different theoretical flood risks, but similar quality and size, command the same market values; thus, past price differentiation has been on the basis of factors other than flood risks.
>
> Inundation reduction was considered only on existing structures. The calculation of inundation reduction benefits requires knowledge of the values and types of structures which would be inundated, and while an estimate is available of the number of acres expected to be developed in the floodplain during the project period, any projection of the values and types of structures to be built would be highly speculative at best" (Corps, 1990).

The preceding two paragraphs from the Comite River Basin report also appear in the Corps' Darlington report (1992). The flood control benefits given in the Darlington reservoir study consist of the estimated annual savings in flood damages to existing real property, plus an additional 20 percent to account for reductions in 1) vehicle inundation damages, 2) emergency flood relief costs, 3) federal insurance administration costs, and 4) fill reduction benefits. The 20-percent factor for these benefits in relation to structural benefits was determined based on detailed quantified estimates in the Comite River basin study (Corps, 1990).

The rationale for the fill reduction benefits was (Corps, 1990):

> "When new areas are developed in the Amite River Basin, much of the land will have to be filled in order to bring slab elevations up to the level of the 100-year flood event. This is necessary to comply with floodplain regulations. Benefits will accrue to future homeowners to the extent that a flood reduction project lowers

> the elevation of the 100-year flood event, since for every one-foot decrease in the 100-year event, the homeowner may use one foot less of fill material under the slab of the house."

This statement also supports the position that real property damage reduction benefits should be based on existing structures, if new buildings must be elevated above the 100-year floodplain.

Adjustments to NED Benefits. Benefits from reduced structural damages in the Darlington reservoir and Comite diversion reports are based on an April 1987 survey of structures by Gulf States Research Institute of Baton Rouge. The damages are expressed in both reports at the 1989 price level. Two adjustments were made to these benefits:

> The price level of benefits for Comite and Darlington were indexed from 1989 to the end of December 1994 by a factor of 1.207 based on the change in the Consumer Price Index (CPI) during that period. Benefits for the EBRP projects were indexed from 1993 to December 1994 by a factor of 1.036 based on the CPI.
>
> Minor changes in benefits for subbasins 48-51, the west Denham Springs area, were based on corrected estimates of structural damages provided by the Corps (Corps, 1995).

An incorrect analysis of house slab elevations was discovered in Denham Springs (Denham Springs, 1993). The cause was the procedure used to produce the two-foot contours on orthophoto maps for the Corps (Aero-Data, 1993). According to the Corps, the main surveying problem was in Denham Springs north of Florida Avenue. The Corps resurveyed elevations and also checked other areas around Denham Springs. Minor errors found at Port Vincent were corrected also. The Corps produced corrected contour maps, then revised house elevations, and finally reran damages. Without-project average annual damages for the basin increased by only 2.2 percent, so other cases were not rerun. In 1989 dollars, Honey Cut Bayou's subbasin annual damages increased by $1300, Miller's Canal's increased by $919,000, Upper Gray's Creek's decreased by $338,000, and Upper Amite River Influence's decreased by $17,000. Therefore, no other adjustments were made to the NED benefits. However, a more rigorous review of the slab elevation problem is warranted.

The benefits from subbasin 44, Amite River in Ascension Parish, were not deleted because there is little overlap with the parish flood control program including construction of levees and pumping stations (Parish, 1995).

The revised benefits are listed by subbasin in Table 6. Since some subbasins included in the Darlington-reservoir-benefited area may be also benefited by the Comite diversion and the EBRP Tributaries projects, the total flood control benefits of each plan depend

on the sequence of construction. The benefits are summarized in Table 6 according to construction sequence. According to the Corps, there is practically no overlap in structures benefitted by the EBRP Tributaries project and by Comite or Darlington, as can be seen by the small difference between numbers in the last two columns of the table; thus, EBRP's position in the sequence does not significantly affect benefits. The benefits of Darlington without Comite have not been computed and thus are not shown.

Table 6

EXPECTED ANNUAL FLOOD CONTROL BENEFITS

	Comite Diversion 12,700 cfs	Darlington (with Comite in place)[a] Reduced 25-Yr Wet	Reduced 50-Yr Wet	Full 25-Yr Wet	Full 50-Yr Wet	EBRP (w/o Dar.)[b] Without Comite	With Comite
Construction sequence: Comite, Darlington, EBRP							
Total annual benefits, $1000[c]	9,158	8,109	8,358	8,045	8,306	21,258	21,223
x multiplier[d]	1.4484	1.4484	1.4484	1.4484	1.4484	1.036	1.036
= adjusted annual benefits, $1000	13,264	11,745	12,106	11,652	12,030	22,023	21,987

[a] Benefits for subbasins 48-51 (Honey Cut, Millers Canal, Upper Grays, Upper Amite) adjusted by percentage change in annual flood damages reported by Corps Jan. 4, 1995.

[b] Corps, East Baton Rouge Parish Watershed Flood Control Project, 1993, App. H, pp. 65-66.

[c] Benefits for Amite River subbasin 44 included since little overlap with Ascension Parish levy project.

[d] Multipliers for Comite and Darlington are 1.2 for other flood control benefits and 1.207 for inflation from 1989 to Dec. 1994 based on the CPI. Inflation multiplier for EBRP is 1.036 from 1993 to Dec. 1994.

Recreation Benefits. The recreation benefits used in this analysis are the same as reported by the Corps, $8.9 million annually for the full size wet reservoir plan, increased by 6.02 percent based on the change in the CPI from August 1992 to December 1994. Reduced wet reservoir scheme were prepared for the economic analysis with the conservation pools raised 5 ft, resulting in raised dam crest elevations. The reason for those reduced-wet alternatives is to increase the depth of the reservoir pool for better quality recreation development than would be possible with the Corps' reduced wet reservoir alternatives. Recreation benefits for the reduced wet reservoirs were estimated at 1/2 the difference between a dry reservoir and full wet reservoir benefits - $7.6 million. Recreation benefits were increased by 20.7 percent for Comite diversion and 3.6 percent for the EBRP projects to bring them all to a December 1994 price level.

Hydropower and Water Supply Benefits. According to the Corps' report, there are no water supply or hydropower benefits from the proposed Darlington reservoir because adequate supplies are available for future needs without the reservoir (Corps, 1992).

The value of power generated at the project is less than the specific annual cost of constructing power generation facilities.

Preliminary analysis by Brown and Butler indicates investment in a $300 million pipeline to provide municipal and industrial (M&I) water from Darlington reservoir to the city of New Orleans is justified. The justification for M&I benefits in that analysis is based on a monthly payment to pay off the pipeline over 30 years at 10 percent interest of $2.6 million representing the savings if one-fourth of the families in the service area convert from bottled water to Darlington reservoir water. There may be such potential for M&I use from the reservoir, but more information is necessary. The benefit of an M&I supply from Darlington reservoir is the cost of the most likely alternative source of M&I water that would be developed in the absence of the reservoir. Some of the questions are:

1. What alternatives are available to the utilities now serving the potential customers in the New Orleans service area?

2. Would consumers switch from bottled water to Darlington reservoir water? In other words, is bottled water considered a proper basis for pricing the value of reservoir water?

3. If the water is valued in the home, what are the costs in addition to the pipeline to deliver it?

4. Will Mississippi River water be commingled in the delivery system with Darlington reservoir water? Can Darlington reservoir replace all of the water currently supplied from other sources?

5. After subtracting all of the costs to a point of comparable valuation, will there be any net benefits remaining to help justify Darlington reservoir?

Results of Economic Analysis. The results of the economic analysis are presented in Table 7, showing the effect of construction sequence on the benefit/cost ratios.

The economic analysis indicates that a Darlington reservoir project may be economically justified from the national perspective with the revised costs and flood control benefits derived from the Corps analysis, and the economic attractiveness improves with estimated additional flood control benefits as discussed subsequently.

The most likely scenario is that construction of the Comite diversion and EBRP projects will precede construction of Darlington reservoir (Table 7). Under that assumption, the 25-year reduced-wet Darlington reservoir has a benefit/cost ratio of 1.00 with the Corps flood control benefits and 1.08 with estimated additional flood control benefits. The Comite diversion is justified because the benefit/cost ratio is 1.07 to 1.19. The EBRP projects also are economically justified regardless of the construction sequence.

The 25-year redcuced wet reservoir, with conservation pool at Elevation 155, would cover about 6250 acres (Espey, Houston, 1989), have a shoreline about 35 miles long, and rise about 22 feet during the 25-year design flood. It would reduce the river stage at Denham Springs 6.6 feet during a 50-year flood (Corps, 1992), about the magnitude of the 1983 flood. Annual benefits would average about $19 million including recreation, or about $21 million with estimated additional flood control benefits.

The estimated additional flood control benefits shown in Table 7 that are assumed to result from the Darlington reservoir and Comite diversion projects are $1.5 million annually for the reservoirs with 25-year flood design and $2.5 million for the reservoirs designed to contain the 50-year flood. An additional benefit of $1.5 million was also assumed for the Comite diversion. We were not able to prepare quantified estimates of these benefits within the funding and time limitations of this report. However, we believe there is a basis for some increase in benefits of the magnitude indicated for the following reasons:

The Ascension Parish levees take away about 12 mi^2 of floodplain-area storage, so the flood crest and damages to other areas may be somewhat higher as a result of the new levees.

There have probably been substantial increases in the number of structures on the floodplain in some of the subbasins since 1987. That would increase the benefits from avoiding flood damages to existing structures.

There will be intensification benefits from increased investment without change in land use, and location benefits from a change in land use under with-project

Table 7

COMPARISON OF COSTS AND BENEFITS OF ALTERNATIVE PLANS
($1000, December 1994 price level) (a)

Construction sequence:
1. Comite Diversion
2. Darlington Reservoir
3. EBRP Tributaries

	Darlington Reservoir				Comite	EBRP
	Reduced Wet Reser		Full Wet Reservoir		Diversion	Tribs
	25-Year	50-Year	25-Year	50-Year	12,700cfs	w/Comite
Crest elevation, ft msl	207	210	212.5	214.8		
Cost item						
- Dam (or other) structure	131500	137000	142000	147400	121800	179000
- Relocations	9000	12000	18000	21000		
- Real estate	25000	28000	31000	32500		
- Mitigation	9500	11500	13000	14000		
- Recreation	25000	27500	29500	31100		
Subtotal	200000	216000	233500	246000	121800	179000
- Interest during construction	30000	32400	35120	37010	34480	34490
Total investment	230000	248400	268620	283010	156280	213490
Annual investment (b)	18262	19723	21329	22471	12409	16951
Annual operation, maintenance, and replacement						
- Dam and reservoir	144	158	175	175	253	249
- Recreation facilities	790	869	900	900		
- Mitigation areas	91	100	141	141		
Total OM&R	1025	1128	1216	1216	253	249
Total annual cost	19287	20851	22545	23687	12662	17200
Annual benefits						
- flood control	11745	12106	11652	12030	13264	21987
- recreation	7595	7595	9452	9452	243	272
Total	19340	19701	21104	21482	13507	22259
Benefit/cost ratio	1.00	0.94	0.94	0.91	1.07	1.29
Est. additional flood con. ben	1500	2500	1500	2500	1500	1238 (c)
Total benefits	20840	22201	22604	23982	15007	23497
Benefit/cost ratio	1.08	1.06	1.00	1.01	1.19	1.37

(a) Price indexes based on Consumer Price Index: Darlington reservoir, August 1992 base, index 1.062. Structural damage benefits, 1989 base, index 1.207.
(b) 50 yrs, 7.75 percent.
(c) EBRP benefits during construction and erosion control indexed by 1.036.

conditions. Those benefits were not estimated in the Corps' study because they are more difficult to quantify. Nonetheless, they do occur, and some attempt should be made to estimate their magnitude.

There are benefits from avoiding losses of business profits and employment during the period of economic disruption caused by the flood and clean-up.

There are benefits from the employment of unemployed or underemployed people during construction and operation of project facilities and in the private businesses stimulated as a result of project recreation development.

Additionally, an improvement to the viability of the project could be provided by a further reduction in costs. Significant reductions are expected to occur as a result of the re-analysis of the spillway and recreation facilities. In addition, if the Comite diversion is not reauthorized, the economic picture for Darlington improves markedly.

Regional Economic Development Benefits. Governor Edwards was recently reported as saying the Darlington reservoir project would probably not qualify for federal funding and that it would have to be financed by the state (Baton Rouge *Advocate*, Mar. 14, 1995). The Water Resources Council's Principles and Guidelines provide for a regional economic development account for presenting the costs and benefits of a project from the regional perspective. That is the proper accounting stance for an analysis of project benefits and costs from the state or local perspective.

The flood control benefits and recreation benefits from the NED account are also applicable to the state account because they occur in Louisiana. Other categories of benefits to the state from increased income and employment as a result of transfers of economic activity which could not be included in the NED account are:

1. Income from implementation outlays that increase state income. That only applies to project implementation costs financed from outside the state.

2. Income from transfers of basic economic activity. Those are wages and profits of new businesses that locate in Louisiana as a result of improved flood protection or recreation development.

3. Indirect effect income resulting from expansion in the production of inputs to industries supplying increased final products or exports from the state.

4. Induced effects income resulting from changes in consumption expenditures generated by increases in personal expenditure (US Water Resources Council, 1983).

Similarly, on the cost side, the relevant project costs from a state viewpoint are those paid by residents of Louisiana. These would be the O,M & R costs, and the state- or locally-funded share of project investment costs. Regional economic development (RED) benefits were considered in the Corps' East Baton Rouge Parish study (1993):

> "Reducing flood damage frequency and cost will improve economic growth, employment, property valuation, and tax revenue in the region. Conversely, allowing flooding to continue to occur could likely result in decreasing same. Direct economic benefit to existing property is included in the NED estimates above. Induced economic benefits are speculative to a large degree and are not calculated directly into the benefit-cost analysis."

The display of RED effects in the Corps' report (1993) is summarized in Table 8.

Table 8

REGIONAL ECONOMIC DEVELOPMENT EFFECTS OF FLOOD CONTROL IN EAST BATON ROUGE PARISH

Item	With Project	Without Project
A. Regional income and employment	Improved value will likely facilitate urban growth	Income and business may be reduced due to recurring flooding
B. Regional growth and business activity	Increased employment for construction, better economic climate due to reduction in flood threat	Same as above
C. Tax revenue	Improved flood protection will stabilize tax base	Same as above
D. Property value	Improved flood protection will likely stabilize or raise property values	Property values may decline due to recurring flooding

The table describes what likely will happen under with-project conditions. Comments follow.

If the benefits were quantified, items A and B describe the same thing--increased regional income consisting of business profits and employment payrolls.

Although increase in tax revenues, item C above, is commonly thought of as a benefit, it actually is just a transfer payment within the state. Higher tax revenues to state and

local governments, sometimes considered as a benefit, are really not a benefit from the state viewpoint because they are just a transfer of income (a cost) from taxpayers to the tax recipients.

Although property values would probably be higher with flood protection as indicated in item D, it is improper to consider the amount of increase as a monetary benefit because it is just capitalization of the reduced flood damages as explained earlier.

A careful RED analysis could quantify basic income benefits and multiplier effects induced in the state economy from both flood protection and recreation development. The economic benefits from the state perspective would probably exceed project costs.

Financial Analysis

The financial analysis was based on a review of the financial proposals for EBRP and the Comite diversion. Based on these analyses, a financing schedule for a combination Comite/Darlington implementation was developed.

The Corps has prepared a detailed financial plan for the East Baton Rouge Parish flood projects. The Parish has agreed to be the local sponsor. At the 1993 price level, the total project cost is $128.7 million, of which the federal share is $96.2 million (74.7%) and the parish share $32.5 million (25.3%). Based on the estimated schedule of construction expenditures and an annual inflation rate of 3.5 percent, the estimated actual outlay for the project in current dollars for the years of expenditure is $179.5 million, of which the parish share is $45.35 million. The average annual O,M&R cost in 1993 dollars is $240,000. Allowing for inflation, the annual O,M&R cost in 2006 is estimated at $374,000 (Corps, 1993). Financing was evaluated in terms of bonded ad valorem or sales tax revenues for 25 years at 8 percent interest. Funding on the basis of 25-year bonded sales tax revenues results in a buildup of annual debt service cost to the parish, reaching a stable payment of $4.8 million when construction is completed in the year 2005. It was concluded that the local share of the EBRP projects cost was well within the financial capability of the parish.

The Louisiana Department of Transportation and Development was identified as the local sponsor for the Comite diversion project, and it would likely be the sponsor for the nonfederal costs of Darlington reservoir. A financing plan for the Comite diversion project and the Darlington reservoir project is presented in Table 9. The total cost of the Comite diversion at the 1995 price level is $121.8 million, of which the nonfederal share is 50 percent, or $60.9 million. After inflation at 3.5 percent during the construction period, the total state share of the project cost in current dollars as of the years of scheduled expenditures is $65.5 million.

Table 9 shows the annual cost of debt service if the state construction expenditures are financed by 30-year general obligation bonds at 8 percent interest. The actual interest

rate may vary depending on when the bonds are issued. Assuming that construction is completed in 1999, the cost of debt service from the project will be $5.8 million annually through the year 2025. Thereafter, the bonds are retired until debt service reaches zero in 2029. The total cost of repayment plus interest is $174.5 million.

The cost to the state for construction of the Darlington reservoir project is shown under two situations, with federal funding and without federal funding, using the 25-year reduced wet reservoir with dam crest at 207 ft. Without federal funding, the cost of debt service to the state of Louisiana will be $25.5 million annually over the period 2003 to 2027, with buildup preceding this period and decreasing amounts after 2027 as bonds mature. The total cost of debt service and annual O,M&R of both the reservoir and diversion projects builds to $30.8 million in 2004, the first year after construction of Darlington reservoir is completed. Thereafter, annual costs continue to increase because of the effect of inflation on O,M&R costs until the peak of $32.6 million in 2025. Thereafter, costs decline to $4.6 million by 2033 for O,M&R. The last bond for the reservoir is paid off in 2032.

If Darlington reservoir qualifies as a federal project, the estimated state share of project construction cost is $64.7 million (28.13 percent), consisting of the costs for land acquisition, relocation, mitigation, 50 percent of recreation, and 5 percent of other costs during construction. The cost of debt service builds to $6.9 million in 2003 and remains at that level again until 2027, when it declines in succeeding years as earlier-issue bonds are paid off. Total state costs for both projects are $14.4 million in 2004 instead of $33.0 million as in the previous case. Federal participation in the Darlington reservoir project means an annual debt service cost savings to the state of about $18 million.

Table 9. Financing Plan 1996–2033

Schedule of State of Louisiana Expenditures and Debt Financing
($000, current prices)

Feature	Total Cost	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
Comite Diversion																
Total Construction Cost	121,800															
State Share 50% a/	60,900	18,909	20,876	24,308	1,398											
Debt Service 8%, 30 Yrs.		1,680	3,534	5,693	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818
Annual O,M&R	253	-	-	-	-	300	311	322	333	345	357	369	382	396	410	424
Subtotal State Cost		1,680	3,534	5,693	5,818	6,118	6,129	6,139	6,151	6,162	6,174	6,187	6,200	6,213	6,227	6,241
Darlington Reservoir																
Total Construction Cost	230,000	-	-	7,141	18,971	28,956	57,111	76,375	98,129							
Without Federal Funding																
State Share 100%	230,000	-	-	7,141	18,971	28,956	57,111	76,375	98,129							
Debt Service 8%, 30 Yrs.		-	-	634	2,320	4,892	9,965	16,749	25,466	25,466	25,466	25,466	25,466	25,466	25,466	25,466
Annual O,M&R	1,025	-	-	-	-	-	-	-	-	1,350	1,397	1,446	1,496	1,549	1,603	1,659
Subtotal State Cost		-	-	634	2,320	4,892	9,965	16,749	25,466	26,816	26,863	26,912	26,963	27,015	27,069	27,125
With Federal Funding																
Land Acquisition	25,000															
Relocations	9,000															
Mitigation	9,500															
Recreation 50%	12,500															
Dam, reservoir, other 5%	8,700															
State Share 28.13% b/	64,700	-	-	4,688	11,782	18,442	13,521	13,171	16,188							
Debt Service 8%, 30 Yrs.		-	-	416	1,463	3,101	4,302	5,472	6,910	6,910	6,910	6,910	6,910	6,910	6,910	6,910
Annual O,M&R	1,025	-	-	-	-	-	-	-	-	1,350	1,397	1,446	1,496	1,549	1,603	1,659
Subtotal State Cost		-	-	416	1,463	3,101	4,302	5,472	6,910	8,260	8,307	8,356	8,407	8,459	8,513	8,569
State Cost Summary																
Without Federal Darlington		1,680	3,534	6,328	8,137	11,010	16,093	22,889	31,617	32,978	33,037	33,099	33,162	33,228	33,296	33,367
With Federal Darlington		1,680	3,534	6,110	7,281	9,219	10,431	11,612	13,061	14,422	14,482	14,543	14,606	14,672	14,740	14,811

a/ Schedule of construction expenditures based on USACE Comite Diversion Report, Appendix A, Table 58.
b/ Schedule of construction expenditures based on Espey, Huston & Associates Report, Table 5.7-7.

Table 9. Financing Plan 1996–2033 (continued)

Schedule of State of Louisiana Expenditures and Debt Financing
($000, current prices)

Feature	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026
Comite Diversion																
Total Construction Cost																
State Share 50% a/																
Debt Service 8%, 30 Yrs.	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818	5,818	4,138
Annual O,M&R	439	454	470	486	503	521	539	558	578	598	619	640	663	686	710	735
Subtotal State Cost	6,256	6,272	6,287	6,304	6,321	6,339	6,357	6,376	6,395	6,415	6,436	6,458	6,480	6,504	6,528	4,873
Darlington Reservoir																
Total Construction Cost																
Without Federal Funding																
State Share 100%																
Debt Service 8%, 30 Yrs.	25,466	25,466	25,466	25,466	25,466	25,466	25,466	25,466	25,466	25,466	25,466	25,466	25,466	25,466	25,466	25,466
Annual O,M&R	1,717	1,777	1,840	1,904	1,971	2,040	2,111	2,185	2,261	2,340	2,422	2,507	2,595	2,686	2,780	2,877
Subtotal State Cost	27,183	27,243	27,306	27,370	27,437	27,506	27,577	27,651	27,727	27,807	27,888	27,973	28,061	28,152	28,246	28,343
With Federal Funding																
Land Acquisition																
Relocations																
Mitigation																
Recreation 50%																
Dam, reservoir, other 5%																
State Share 28.13% b/																
Debt Service 8%, 30 Yrs.	6,910	6,910	6,910	6,910	6,910	6,910	6,910	6,910	6,910	6,910	6,910	6,910	6,910	6,910	6,910	6,910
Annual O,M&R	1,717	1,777	1,840	1,904	1,971	2,040	2,111	2,185	2,261	2,340	2,422	2,507	2,595	2,686	2,780	2,877
Subtotal State Cost	8,627	8,688	8,750	8,814	8,881	8,950	9,021	9,095	9,171	9,251	9,333	9,417	9,505	9,596	9,690	9,787
State Cost Summary																
Without Federal Darlington	33,440	33,515	33,593	33,674	33,758	33,844	33,934	34,027	34,123	34,222	34,325	34,431	34,541	34,655	34,773	33,216
With Federal Darlington	14,884	14,959	15,037	15,118	15,202	15,288	15,378	15,471	15,567	15,666	15,769	15,875	15,985	16,099	16,218	14,660

Table 9. Financing Plan 1996–2033 (continued)

Schedule of State of Louisiana Expenditures and Debt Financing
($000, current prices)

Feature	2,027	2,028	2,029	2,030	2,031	2,032	2,033	Total State Cost
Comite Diversion								
Total Construction Cost								
State Share 50% a/								65,491
Debt Service 8%, 30 Yrs.	2,283	124	0	0	0	0	0	174,526
Annual O,M&R	761	787	815	843	873	903	935	19,067
Subtotal State Cost	3,044	911	815	843	873	903	935	193,593
Darlington Reservoir								
Total Construction Cost								
State Share 100%								286,683
Debt Service 8%, 30 Yrs.	25,466	24,832	23,146	20,574	15,501	8,717	0	763,983
Annual O,M&R	2,978	3,082	3,190	3,301	3,417	3,537	3,660	69,677
Subtotal State Cost	28,444	27,914	26,336	23,876	18,918	12,253	3,660	833,659
With Federal Funding								
Land Acquisition								
Relocations								
Mitigation								
Recreation 50%								
Dam, reservoir, other 5%								
State Share 28.13% b/								
Debt Service 8%, 30 Yrs.	6,910	6,494	5,447	3,809	2,608	1,438	0	207,305
Annual O,M&R	2,978	3,082	3,190	3,301	3,417	3,537	3,660	69,677
Subtotal State Cost	9,888	9,576	8,637	7,110	6,025	4,974	3,660	276,982
State Cost Summary								
Without Federal Darlington	31,488	28,825	27,151	24,719	19,791	13,157	4,595	1,027,252
With Federal Darlington	12,932	10,487	9,452	7,954	6,898	5,878	4,595	470,575

Public Involvement

The first public meeting on the Amite River and Tributaries study was held on October 30, 1984. Public scoping meetings for the environmental impact statement on the permit application for Darlington dam and reservoir were held on June 25, 1985 in Baton Rouge and on June 27, 1985 in Greensburg, Louisiana. Subsequently, representatives of the USCE and DOTD participated in regular meetings of the Amite River Basin Drainage and Water Conservation District.

The Corps Comite diversion study documents numerous meetings with "state legislators and mayors of major cities,...Chamber of Commerce, Amite River Basin Drainage and Conservation Commission, Lower Amite River Protection Association," and several state and federal agencies. Also, they participated in 41 meetings of the Amite River Basin Drainage and Conservation Commission over the period 1986 to 1990 (Corps, 1990).

The Corps EBRP projects study documents several meetings over the period 1992-93 with the Baton Rouge mayor, Chamber of Commerce, Department of Public Works, Recreation and Park Commission, and the Metro Council; state and federal agencies; Louisiana State University; and the Citizens of Baton Rouge Councils for Districts 1, 3, and 12.

Livingston Parish Projects

The Corps is continuing its study of flood protection in the parish, focusing on direct-rainfall flooding along Grays Creek and its main tributary, Miller's Canal. Ring levees (including pump stations) around Denham Springs and around Port Vincent-French Settlement previously were all found to be unjustified by the Corps. A draft detailed design report is due out in October (Corps-LDOTD, 1995).

Design of the Miller canal diversion project is almost complete. It would divert water from the existing Miller canal, which drains southwest Denham Springs and runs southeast to Grays Creek, west to the Amite River floodplain. The Statewide Flood Control Program is to fund 75 percent and Denham Springs 25 percent. In 1993, Denham Springs obtained approval to proceed.

Ascension Parish Projects

Over the last several years, Ascension Parish has been in the process of implementing a flood control program in the eastern part of the parish that consists of a series of levees combined with channel improvements. Flooding in the parish is caused by headwater overflow of the Amite river, New River, and the major bayous. Backwater flooding occurs in the lower reaches of all of those watercourses and is especially severe when the Amite River floods. The program will protect the Gonzales and Sorrento areas against flows in the Amite river up to the 100-year flood. The channel improvements will

improve the interior drainage and, combined with a 4,000-cfs pump station, provide protection against about a 5-year interior or headwater flood.

The levee system consists of a ring dike around Sorrento with a pump station discharging into Bayou Conway, a continuation of the levee from the ring dike along Bayou Francois and New River to the Saveiro Canal, north along the Canal and beyond to near the Amite River diversion, and west to the Black Bayou area. The channel improvements will primarily be implemented along Bayou Francois, Bayou Narcisse, Black Bayou, Saveiro Canal, and the New River Canal and include a new crossover from Bayou Narcisse to New River. The pump station is located immediately downstream from the confluence of the Saveiro Canal and New River and includes four 1,000-cfs pumps and one spare pump. The program essentially follows the plan for improvements in Ascension Parish that was outlined in the Corp's Initial Evaluation Study (1984).

The levee system has made use of levees constructed under earlier programs to provide the indicated protection. As such, the system is a combination of new levees, existing levees that have to be raised, and existing levees that were originally constructed to height suitable for the current program.

The levee system is essentially complete with the exception of a reach on the east side of the protected area near the Amite River diversion and a reach on the north end of the system. The pump station has been completed and has operated successfully; and much of the interior channel improvements have been completed. The original permit has expired, and construction is on-hold until a new permit can be obtained.

The flood protection has operated successfully. Minor problems have occurred as a result of the operation of the pump station whereby rapid drawdown of the channel reaches immediately upstream of the station has caused bank sloughing. These problems are considered to be temporary and will be corrected when operating rules for the pump station have been finalized.

No studies have been done to assess the impact of the Ascension Parish program on flooding in the Amite River basin. As a result of the dikes, a major area of the Amite River floodplain has been eliminated. The impact of reducing the floodplain may be to exacerbate the flooding conditions in other parts of the basin.

Lower Amite Channel Dredging

Significant sedimentation has occurred in the lower Amite River, primarily between four miles north of Bayou Manchac and Colyell Creek south of Port Vincent and between the Amite diversion weir and Lake Maurepas, in the last 30 years or so, causing increases in flood levels of up to three feet. Dredging would restore the river channel to the original improvement conditions.

Channel dredging between Port Vincent and French Settlement may affect locations and habitat of the Alabama heelsplitter mussel, P. inflatus, which is a federally endangered species. Whether it actually exists in the reach proposed for dredging is not presently known. The mussel is known to exist downstream of Darlington to the area of Bayou Manchac (Espey, Houston, 1989). However, the recent literature does not indicate the presence of any identified location of P. inflatus below Bayou Manchac (Hartfield, 1988, 1989). A 1976 survey did identify the mussel near the State Route 22 bridge (Stern, 1976). Recent surveys (summer 1994) by the Natural Heritage Program of the state Department of Fish and Game did not cover the area below Port Vincent; however, those surveys did find a continual contraction of the mussel's habitat, primarily due to sand and gravel mining (Shively, Feb. 1995, Dept. Fish and Game, per. comm.). Considering the upstream degradation of P. inflatus habitat, a similar situation may exist here, particularly considering the sediment deposition that has occurred in the last ten years. Channel dredging may be possible in areas where P. inflatus has been eliminated by sand and gravel mining or by other modification of the riverbed. The river downstream of Port Vincent has not been affected by sand and gravel mining. The channel is U-shaped with a silt substrate, which is the habitat favored by P. inflatus (Dr. Ken Brown, Feb. 1995, per. comm.). Detailed mussel surveys need to be conducted from Lake Maurepas up to Bayou Manchac to determine locations of the mussel and potential habitat areas.

Placement of dredged spoil along the banks of the river will most likely result in filling of wetlands. The bottomland forests along the Amite River downstream of Port Vincent are likely classed as forested wetlands, like they are in the Darlington reservoir area (Espey, Houston, 1989). Filling of wetlands is regulated under the Clean Water Act, and spoil placement along the banks may require in-kind replacement of wetlands as compensation. If dredging could be confined to within the channel, placing spoil within the channel might not significantly impact wetlands.

Other Structural Measures

Inversion, or groundwater injection, wells along the streams, terracing, and detention ponds limiting outflow to predevelopment rates, all have been suggested. No technical effectiveness analyses or costs were presented (La Fleur, 1990). For example, runoff of three inches from the basin above Port Vincent would amount to about 250,000 ac-ft to be drained into wells along the streams; with 100-ft-deep wells 3 ft in diameter, holding about 700 ft^3, about 15 million wells would be required to absorb the water, which is impractical. Terracing has been applied for depths of a few inches on agricultural land; terracing provides increased depression storage over an extensive area, thereby attenuating flood runoff; based on an estimate for a terraced, tile-drained area in Iowa, the cost would be approximately $180 per acre; if, for example, 10 percent of the basin above Port Vincent were terraced, the initial cost would be $18 million. Detention ponds have been used in many suburban areas; they are sized for small storms; they would help alleviate headwater flooding but not, directly, backwater flooding.

Floodplain Management

A regulatory floodway for all the significant streams in the basin, in addition to the Amite in East Baton Rouge Parish, could be determined by FEMA. That could result in the prevention of new development or significant rebuilding within the boundaries of the regulatory floodway. The regulatory floodway boundaries would be narrower than the 100-year floodplain boundaries. The floodway permits passage of the 100-year flood with only a small increase in water surface elevation. The maximum federally allowable increase is one foot, which is large for a flat area such as Baton Rouge. Further, filling in the area between the floodway and 100-year floodplain boundaries (the floodway fringe) would require compensatory storage.

A flood disclosure law would require those selling or leasing property or structures to provide accurate information as to its flood history and flood rating to purchasers or leasers. The law would prevent future occurrences of the common scenario in which new occupants find out the location has a history of flooding or is below the base flood (100-year) elevation.

Such floodplain management practices must consider current federal proposals which would require compensation to land owners for "land takings" through regulation. The impacts of such federal legislation should be considered in the decision process.

No Action

This is always an alternative. According to the governor's task force (1990), it is the "least desirable and most unacceptable method of dealing with the flooding problem." It has no negative environmental impacts and requires no new funding, though funding will continue for temporary flood control and relief measures and rehabilitation of damaged structures (Johnston, 1990).

Chapter III

RECOMMENDED PROGRAM

In 1995, the Amite River Basin Drainage and Water Conservation District is in an excellent position to carry out its mandate--the coordination of implementation of an Amite River basinwide flood control plan. The Board of Commissioners, citizens within their communities, represent the six parishes located in the watershed of the Amite River basin. The District is the ideal quasi-government agency to form a productive partnership with the Corps of Engineers, Louisiana Department of Transportation and Development, and other pertinent government and private entities to carry out a meaningful plan.

The Corps stated in its 1992 report that all of the agencies involved realize one flood control measure will not solve the flood problem in the Amite basin. A variety of structural and nonstructural measures is required, in addition to the improvements already in place. Criteria for selecting program elements are modified from those of the Chamber of Commerce task force (1991).

Criteria

- Taken as a whole, the measures should benefit the upper, middle, and lower reaches of the basin
- A measure should be technically feasible
- A measure should be economically feasible
- A measure's construction and operation can be funded
- A measure should have no significant unmitigated environmental impact

Program

- Build East Baton Rouge tributaries projects
- Build Comite diversion
- Build Darlington dam including recreation development
- Build Miller canal diversion, Denham Springs
- Continue Grays Creek, Livingston Parish flood control feasibility study
- Perform additional modeling of Amite basin to determine effects of individual flood control projects with and without basinwide flood control project
- Conduct detailed survey of Amite River for Alabama heelsplitter mussel from Bayou Manchac to Lake Maurepas
- Dredge Amite River from Bayou Manchac to Colyell Creek
- Perform regular maintenance of river channels
- Determine regulatory floodway for all siginificant streams in the basin

- Pass local ordinance in each parish in the basin protecting the integrity of the floodway
- Pass a law in the state, otherwise in each parish in the basin, requiring public disclosure of the flood hazard
- Continue public education programs - Flood Awareness Week, "Watermark," public speaking engagements
- Complete house slab raising demonstration project; evaluate costs when done
- Supply technical information to individuals about flood proofing on request.

The Ascension Parish projects have been essentially completed.

The EBRP tributaries projects appears to be economically justified by a substantial margin and, in addition, protects areas that cannot be protected by any other identified project. It is recommended that the project be implemented.

Similarly, the Comite diversion project is economically justified, protects some areas not protected by any other identified project, and should proceed as soon as possible to construction.

We recommend that feasibility study of Darlington Reservoir be resumed upon receipt of the State's comments. We are optimistic of a viable federal project for several reasons:

1. Even with Comite and East Baton Rouge Parish tributaries projects in place, reductions in costs and increases in benefits as estimated above give a Darlington Reservoir benefit/cost ratio exceeding 1.00.

2. There are more cost savings to be realized for the project, specifically in the spillway, the mitigation costs, and the recreation costs.

3. There may be more damages in the unprotected portions of the basin as a result of the construction of the Ascension Parish project.

4. There are benefits that can be claimed that have not been considered in the Corps study.

A second option is to implement the Darlington Reservoir project on a state level foregoing federal assistance.

There will still be areas that do not receive protection. For these local areas, we recommend that local measures be implemented. Nonstructural or structural measures should be carefully analyzed for financial feasibility, such as slab raising and relocation. Local ordinances should reflect a basinwide perspective on flooding.

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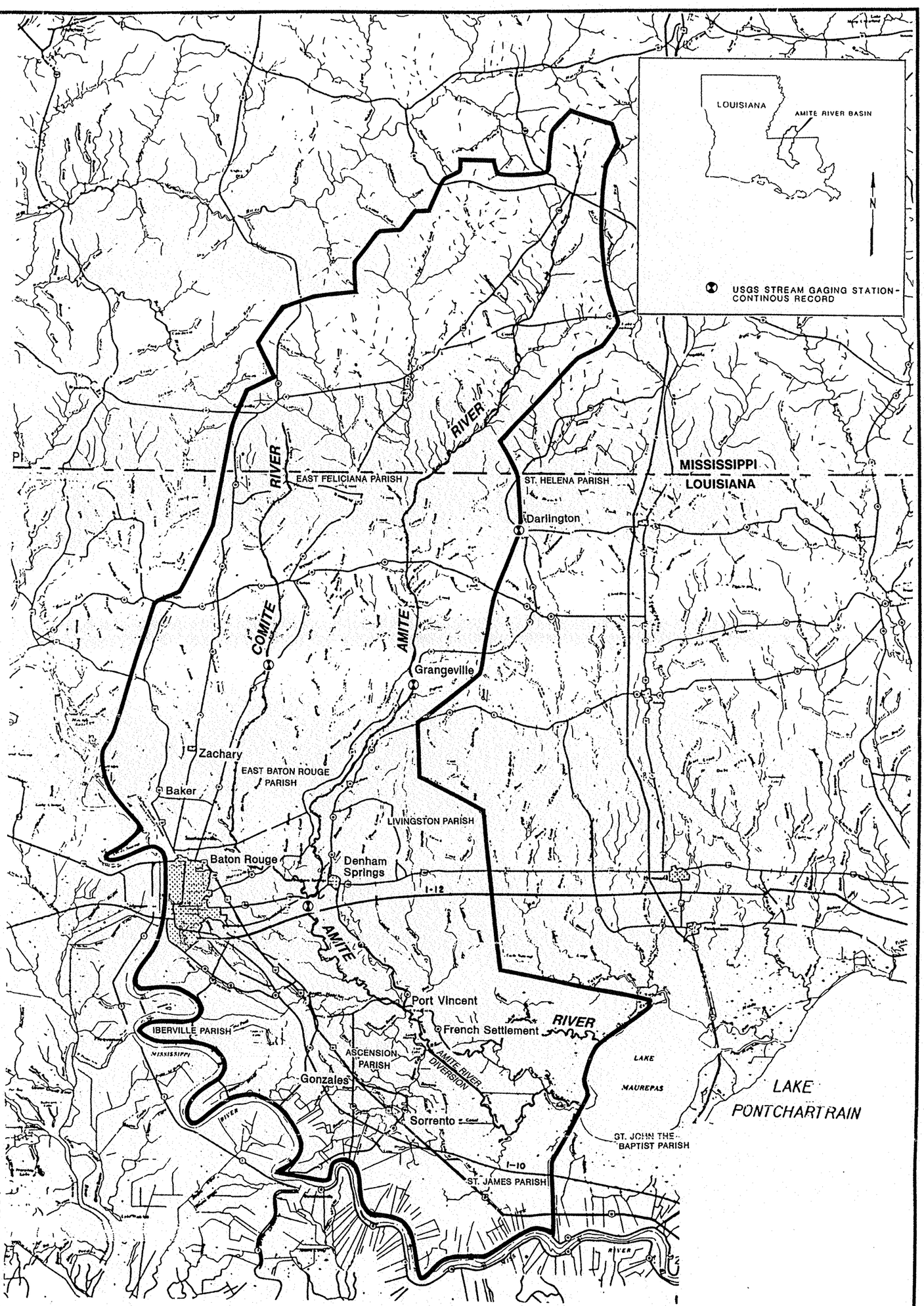
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HARZA ENGINEERING COMPANY



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1:500,000
SCALE IN MILES

Source: U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
DATE AUGUST 1993

AMITE RIVER FLOOD CONTROL
AMITE RIVER DRAINAGE AREA

EXHIBIT 1

EXHIBIT 2

AMITE RIVER ANNUAL MAXIMUM FLOOD PEAKS

USGS gage	Darlington		Denham Springs	
Period of record	1949-94		1921, 1939-94	
Drainage area, mi²	580		1280	
Water Year	Date	Peak, cfs	Date	Peak, cfs
1921			Mar 15	93,000
1939			Jun 6	12,100
1940			Jul 7	16,000
1941			Dec 17	20,800
1942			Sep 19	12,200
1943			Mar 23	40,200
1944			Mar 23	11,000
1945			Jan 10	11,600
1946			Jul 7	15,500
1947			Mar 14	27,800
1948			Mar 5	45,100
1949	–	20,000	Mar 24	28,800
1950	Jan 7	43,400	Jan 9	40,800
1951	Mar 30	31,600	Apr 1	36,900
1952	Dec 21	3,180	Apr 6	8,230
1953	May 20	18,900	May 20	67,000
1954	Jan 16	3,280	Dec 11	15,200
1955	Apr 13	55,700	Apr 15	54,300
1956	Mar 12	20,400	Mar 14	23,400
1957	Jun 29	20,200	Jul 1	12,300
1958	Sep 23	22,400	Nov 17	14,700
1959	Feb 3	6,900	Feb 4	19,100
1960	Dec 19	9,800	Dec 19	18,800
1961	Mar 18	37,900	Mar 20	49,100
1962	Apr 28	15,400	Apr 29	49,700
1963	Jan 21	4,530	Jan 22	5,150
1964	Mar 3	44,500	Mar 4	40,500
1965	Oct 5	44,500	Oct 7	49,900
1966	Feb 13	20,000	Feb 13	39,700
1967	Apr 15	39,300	Apr 17	47,800
1968	Dec 17	8,000	May 13	6,290
1969	Apr 14	8,600	Apr 14	23,000
1970	Mar 21	3,630	Oct 8	21,700
1971	Sep 18	10,100	Sep 20	12,600
1972	Dec 7	45,500	Dec 8	51,800
1973	Mar 25	62,100	Mar 27	60,200
1974	Apr 14	22,400	Dec 27	21,300
1975	Jun 9	40,700	May 10	29,900
1976	Apr 1	7,660	Mar 27	16,100
1977	Apr 22	76,400	Apr 23	110,000
1978	Nov 30	30,500	Dec 2	31,300
1979	Apr 23	43,400	Apr 24	68,600
1980	Mar 28	47,500	Mar 30	64,200
1981	Dec 11	8,320	Dec 12	11,300
1982	Feb 17	18,100	Feb 18	23,900
1983	Apr 7	63,300	Apr 8	112,000
1984	Feb 14	13,000	Dec 12	23,600
1985	Feb 27	8,970	Oct 24	26,400
1986	Oct 30	17,500	Oct 31	43,900
1987	Mar 1	21,200	Jan 20	31,300
1988	Apr 3	22,000	Apr 4	41,300
1989	May 20	16,000	May 21	29,400
1990	Jan 25	104,000	Jan 27	96,700
1991	May 1	19,500	Feb 22	37,700
1992	Mar 6	26,900	Mar 7	48,600
1993	Jan 21	19,400	Jan 22	81,900
1994	Jan 28	72,000	Jan 30	57,000
Number of years	46		57	

EXHIBIT 3

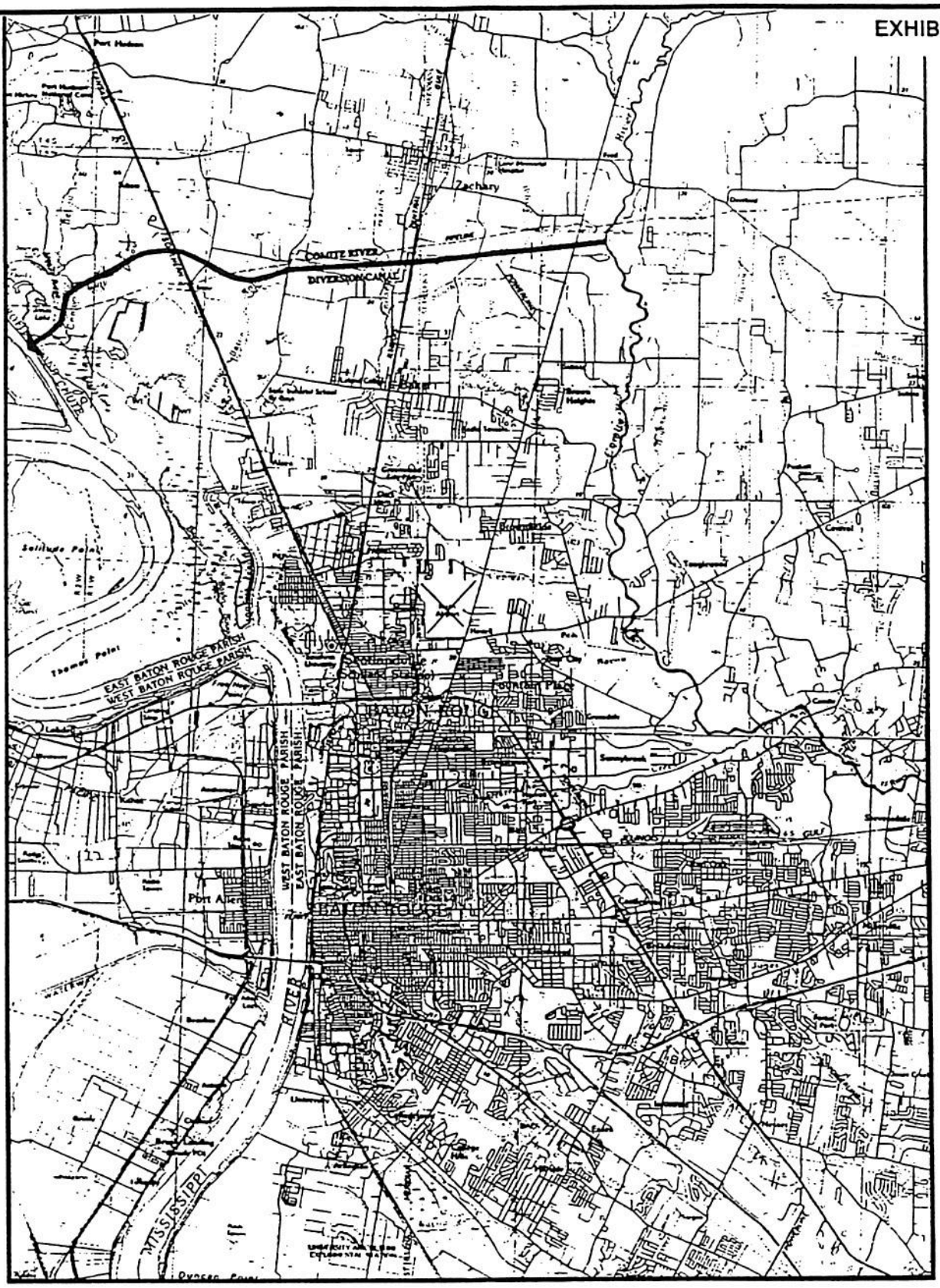


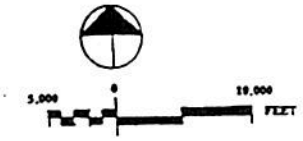
Source: Brown & Butler, Baton Rouge

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DARLINGTON RESERVOIR
PROPOSED LOCATION

EXHIBIT 4





HARZA ENGINEERING COMPANY

COMITE RIVER DIVERSION CANAL
VICINITY MAP

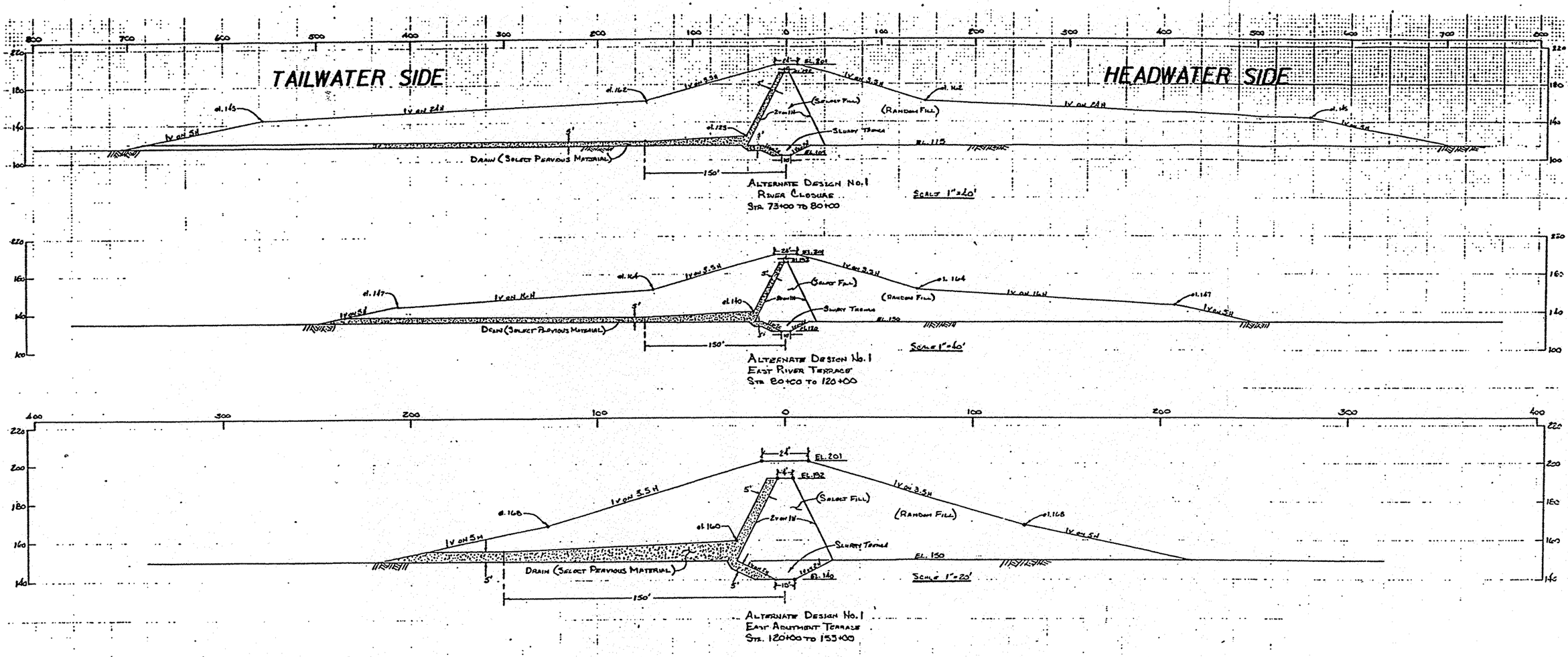
EXHIBIT 5



PROPOSED
CORPS OF ENGINEERS
EAST BATON ROUGE PARISH
TRIBUTARIES PROJECTS

HARZA ENGINEERING COMPANY

EXHIBIT 6



ALTERNATE DESIGN No.1
RIVER CLOSURE
STA. 73+00 TO 80+00

ALTERNATE DESIGN No.1
EAST RIVER TERRACE
STA. 80+00 TO 120+00

ALTERNATE DESIGN No.1
EAST ABUTMENT TERRACE
STA. 120+00 TO 153+00

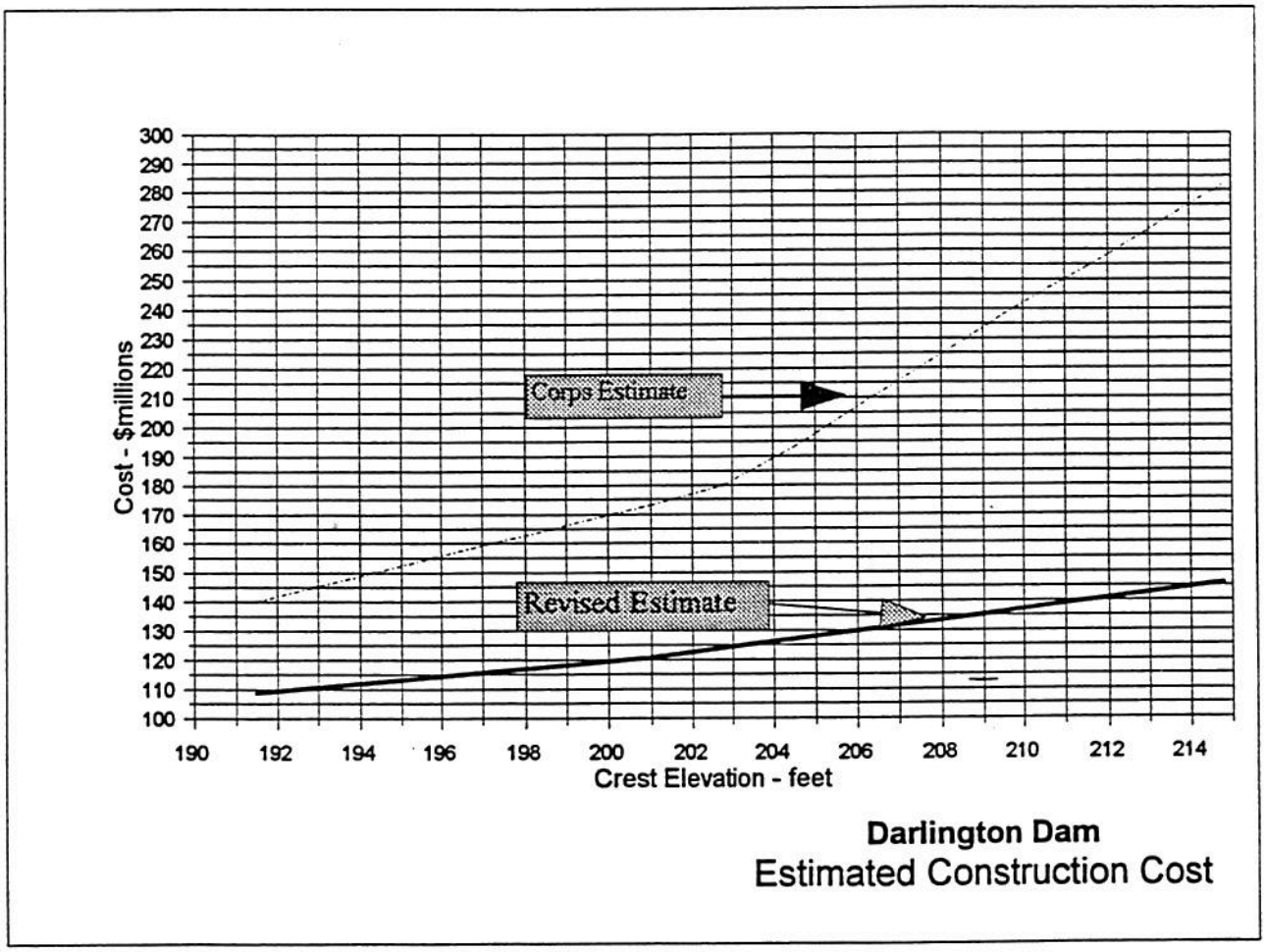
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESEVOIR FEASIBILITY STUDY

TYPICAL CROSS SECTIONS
ALTERNATE DESIGN NO. 1

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

DESIGNED BY: F. V.	PLOT SCALE:	PLOT DATE:	CADD FILE:	
DRAWN BY: L. M. P.			FILE NO.	
CHECKED BY: F. M.	DATE: AUGUST 1992			

EXHIBIT 7



Darlington Dam
Estimated Construction Cost

TABLE 43*

HABITAT IMPACT ANALYSIS SCREENING COMPUTATION
WET RESERVOIR
(CORRECTED FROM 1992 REPORT)
WOODLAND HABITAT LOSSES

WITHOUT

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	8628	0.65	5608		
1	8628	0.65	5608	5608	
2	8628	0.65	5608	5608	
3	8628	0.65	5608	5608	
4	8628	0.65	5608	5608	
5	8628	0.65	5608	5608	
6	8628	0.65	5608	5608	
13	8628	0.65	5608	39257	
23	8628	0.65	5608	56082	
33	8628	0.65	5608	56082	
43	8628	0.65	5608	56082	
63	8628	0.65	5608	112164	
83	8628	0.65	5608	112164	
103	8628	0.65	5608	112164	
				577645	5608
				SUM	CUM/TY

WITH

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	8628	0.65	5608		
1	8628	0.65	5608	5608	
2	8628	0.65	5608	5608	
3	8193	0.65	5325	5467	
4	8193	0.65	5325	5325	
5	8193	0.65	5325	5325	
6	400	0.65	260	2793	
13	400	0.65	260	1820	
23	400	0.65	260	2600	
33	400	0.65	260	2600	
43	400	0.65	260	2600	
63	400	0.65	260	5200	
83	400	0.65	260	5200	
103	400	0.65	260	5200	
				55347	537
				SUM	CUM/TY

PROJECT IMPACT OF AAUH'S= 5071

TABLE 46*

FUTURE
WITH MITIGATION

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	7450	0.00	0		
1	7450	0.00	0	0	
2	7450	0.15	1118	559	
3	7450	0.20	1490	1304	
4	7450	0.20	1490	1490	
5	7450	0.25	1863	1676	
6	7450	0.38	2831	2347	
13	7450	0.45	3353	21642	
23	7450	0.48	3576	34643	
33	7450	0.55	4098	38368	
43	7450	0.65	4843	44700	
63	7450	0.85	6333	111750	
83	7450	0.90	6705	130375	
103	7450	0.90	6705	134100	
				522953	5077
				SUM	CUM/TY

TABLE 43**

HABITAT IMPACT ANALYSIS SCREENING COMPUTATION
WET RESERVOIR 1995 EVALUATION
(BASED ON AREA ESTIMATE OF 6300)
WOODLAND HABITAT LOSSES

WITHOUT

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	6300	0.65	4095		
1	6300	0.65	4095	4095	
2	6300	0.65	4095	4095	
3	6300	0.65	4095	4095	
4	6300	0.65	4095	4095	
5	6300	0.65	4095	4095	
6	6300	0.65	4095	4095	
13	6300	0.65	4095	28665	
23	6300	0.65	4095	40950	
33	6300	0.65	4095	40950	
43	6300	0.65	4095	40950	
63	6300	0.65	4095	81900	
83	6300	0.65	4095	81900	
103	6300	0.65	4095	81900	
	6300				
				421785	4095
				SUM	CUM/TY
	435				

WITH

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	6300	0.65	4095		
1	6300	0.65	4095	4095	
2	6300	0.65	4095	4095	
3	5865	0.65	3812	3954	
4	5865	0.65	3812	3812	
5	5865	0.65	3812	3812	
6	400	0.65	260	2036	
13	400	0.65	260	1820	
23	400	0.65	260	2600	
33	400	0.65	260	2600	
43	400	0.65	260	2600	
63	400	0.65	260	5200	
83	400	0.65	260	5200	
103	400	0.65	260	5200	
				47024	457
				SUM	CUM/TY

PROJECT IMPACT OF AAUH'S= 3638

TABLE 46**

FUTURE
WITH MITIGATION

TY	ACRES	HSI	HU'S	CUMHU'S	AAUH'S
0	5340	0.00	0		
1	5340	0.00	0	0	
2	5340	0.15	801	401	
3	5340	0.20	1068	935	
4	5340	0.20	1068	1068	
5	5340	0.25	1335	1202	
6	5340	0.38	2029	1682	
13	5340	0.45	2403	15513	
23	5340	0.48	2563	24831	
33	5340	0.55	2937	27501	
43	5340	0.65	3471	32040	
63	5340	0.85	4539	80100	
83	5340	0.90	4806	93450	
103	5340	0.90	4806	96120	
			PROJECT MITIGATION =	374841	3639
				SUM	CUM/TY

* These tables are corrected versions of the tables found in the 1992 USACE Report

** These tables update the adjacent corrected versions of the tables found in the 1992 USACE report by providing an updated quantity of acres for "Woodland Habitat Losses".

EXHIBIT 9

TABLE 49

ENVIRONMENTAL MITIGATION COST ESTIMATE
(REVISED FROM ORIGINAL TABLE IN 1992 CORPS STUDY)

	Full Wet Reservoir
REAL ESTATE	
I. TERRESTRIAL	
Woodland Losses in acres	6300
Cleared Lands Needed in acres (see Table 43)	5340
Cost: $1,000/ac x 1.25 + $100000 Acquisition	$6,775,000
II. AQUATIC	
20 Miles Downstream (300 feet each side) in acres	1500
Cost: $1100/ac x 1.25 + $15000 Acquisition	$2,077,500
TOTAL REAL ESTATE	$8,852,500
CONSTRUCTION	
I. TERRESTRIAL	
Reforestation Costs**: $200/ac x (5340+1500+500 ac)	$1,468,000
II. AQUATIC	
Management (river mi = 20)	$739,200
TOTAL CONSTRUCTION	$2,207,200
TOTAL MITIGATION COSTS***	$11,059,700

** A cost of $200/ac is applied to the areas of the cleared land and downstream land for mitigation plus 500 ac of the proposed reservoir shoreline.

*** Previous mitigation costs from 1992 Corps report for the full wet reservoir alternative were $27,449,000.

APPENDIX C

GEOTECHNICAL APPENDIX, SEPTEMBER 1997

GEOTECHNICAL REEVALUATION
OF THE
DARLINGTON RESERVOIR

EXECUTIVE SUMMARY

This report presents the results of the geotechnical design reevaluation of the Darlington Reservoir. In a letter dated 1 March 1996, the Amite River Basin Commission asked the Corps to review the Darlington Reservoir Feasibility Study Report and to address the design concerns expressed by their consultant, Harza Engineering. The main point of contention was the size of the earth sections, since the size of the sections resulted in an unacceptable benefit to cost ratio (B/C) for the project.

Authorization for this study provided an excellent opportunity to look beyond the main point of contention in Harza's letter and investigate other areas where significant cost savings in the geotechnical design are possible. The three major areas that were investigated for cost savings are the size of the design sections, substitution of a zoned embankment for the gravel chimney drain, and a thinner layer of concrete for the spillway section together with drainage relief to accommodate uplift pressures.

Funding for the revaluation allowed for additinal soil borings to be taken and samples to be tested for shear strength. The higher shear strengths obtained for this study, compared to the assumed strengths in the Feasibility Report, resulted in smaller reservoir cross-sections. The current sections provide adequate margins of safety for stability and seepage requirements, for this level of investigation. Three major changes were made based on the geotechnical data obtained for this reevaluation and data from the Feasibility Report. These changes will reduce the cost of the reservoir significantly. The changes are:

1. Smaller reservoir sections.
2. Elimination of the horizontal and inclined gravel drain.
3. A much thinner concrete layer for the spillway.

A part of the investigation consisted of evaluating and comparing the size of the proposed reservoir to other reservoirs in the region, or to reservoirs that have similar foundation soils. This effort will show if the proposed design is overly conservative, unrealistic, and costly. Based on the comparisons, the proposed sections are similar in size, or smaller than other reservoirs that were designed and constructed at other sites by the Corps.

AMITE RIVER AND TRIBUTARIES
DARLINGTON RESERVOIR
GEOTECHNICAL REEVALUATION

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APPENDIX

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GEOTECHNICAL REEVALUATION
OF THE
DARLINGTON RESERVOIR

1. **Introduction.** This report presents the results of the geotechnical design reevaluation of the Darlington Reservoir. In a letter dated 1 March 1996, the Amite River Basin Commission asked the Corps to review the Darlington Reservoir Feasibility Study Report and to address the design concerns expressed by their consultant, Harza Engineering. Harza personnel expressed concern about the geotechnical design methods, design shear strengths in clay layers, and size of the earth sections during a meeting held in New Orleans District on 12 February 1996, among personnel from the Amite River Basin Commission, D.O.T.D., Harza Engineering, and NOD representatives. The main point of contention was the size of the earth sections, since the size of the sections resulted in an unacceptable benefit to cost (B/C) ratio for the project. Authorization for this study provided an excellent opportunity to look beyond the main point of contention in Harza's letter and investigate other areas where significant cost savings in the geotechnical design are possible. The three major areas that were investigated for cost savings are the size of the design sections, substitution of a zoned embankment for the gravel chimney drain, and a thinner layer of concrete for the spillway section together with drainage relief to accommodate uplift pressures. Two alternatives were analyzed, the 25-year reduced wet reservoir without service spillway and the 25-year dry reservoir. Presented are the geotechnical analyses for the reduced wet reservoir which has a crest elevation of 202.8. Since the other reservoir has a crest elevation of 201, whatever works for the higher reservoir will also work for the lower one.

2. **Geotechnical Design Differences.** Harza engineers believe that a dam with slopes of 1V on 3H on the pool side, and 1V on 2.5H on the outflow side can be constructed safely at the site. They stated in the meeting and in a subsequent letter that the shear strength values selected by the Corps for clay appear to be conservative, especially considering the fact that foundation clays had been reported as very stiff to hard by others. Stability analyses in the Feasibility Study Report, dated Sept

92, account for soft clay layers that were encountered by government and private drillers. Plate 13 presents a graphical representation of the difference between the section in Plate 20 of the Feasibility Report and the section proposed by Harza. In reaches where the terraces are shallow, the side slopes for the NOD design sections are 1V on 3.5 H, resulting in minor differences between the two designs. Some borings along the alignment revealed conflicting information. Most of the data showed high blow counts and strong clay layers, but in a few areas the soft clays were clearly marked. Layers with soft consistencies are identified next to the boring logs in the Feasibility Report. Refer to Appendix A for consistency details for the borings taken by private drillers for the Amite River Flood Control Project, which are presented in the Espey Huston and Associates report. Soft clays are found in each boring in the Appendix. One boring shows both soft and very soft clay layers. Plate A lists shear strength values based on consistency. The range of shear strength for a soft clay is 250 to 500 psf. Such low shear strengths could not be justified for this geologic formation, therefore, the lowest value used for design in the Feasibility Report was 1200 psf. This value is significantly higher than the upper bound strength of a soft clay. At the time, it was impossible to justify a higher strength without completely ignoring the soft consistencies encountered by both crews.

3. **Geotechnical Data Obtained for the Reevaluation.** Soil Borings were taken to determine if soft clays existed in the foundation and to recover specimens for shear strength tests. Drillers were given specific instructions to minimize sample disturbance, especially at clay interfaces and throughout clay deposits.

a. Soil Borings. Four undisturbed borings were taken for the study, borings DD-8U, DD-9U, DD-10U, and DD-11U. Boring DD-8U was taken on the west bank of the river and the others on the east bank. The area where the soft consistencies had been found is in the vicinity where boring DD-9U was taken. Boring DD-10U was taken to detect the extent of the clay layer to the east; DD-8U to detect the extent to the west. Since the spillway is a major cost item, boring DD-11U was taken at the proposed site to provide design information to reevaluate the spillway design and

possibly reduce the construction cost. No soft clays were encountered using the more stringent drilling requirements. Boring locations are marked on Plate 1. Plate 2 presents the plotted logs for the borings that were taken for this study. Borings from Plate 16 of the Feasibility Report are presented on Plate 3. These two plates contain all of the borings taken by the Corps crews for this project.

b. Trench Excavations. A bulldozer was used to excavate two trenches to find the types of soil that are available for embankment construction close to the ground surface. The location of trenches DD-12T and DD-13T are marked on Plate 1, relative to the proposed reservoir site. Trench 12T is comprised mainly of sands and gravels. Lean clay was found throughout trench 13T. Soils from trench 12T can be used for zone embankment construction, while soils from 13T are adequate for core construction.

4. Shear Strength Data. Shear strength data was not available when the sections in the Feasibility Report were analyzed for stability. Design strengths in the Feasibility Report were estimated based on judgement. This led to disagreement about design strengths for the clay layers in the foundation, especially since there was conflicting information about strength. One of the main objectives of the reevaluation is to obtain reasonable design strengths for these layers. Reliable strengths must be used to insure that the foundation clays are not over stressed to the point where excess pore pressure buildup results in embankment slides on the clay layers during construction. If the embankment can be constructed to design height, then the chance of stability failure during operation is relatively small, since most embankment and foundation materials will gain strength with time due to consolidation and pore pressure dissipation. The strength of the clay governed the design and determined the size of the reservoir sections. For this reason, the primary focus will be on clay layers in the foundation. Granular materials appear to have sufficient strength. In general, sand deposits yielded lower than average friction angle values, due to the amount of clay mixed in the specimen.

a. Shear Strength Tests. Specimens from the borings were

sent to the Waterways Experiment Station (WES) for special tests to obtain design values. Visual classifications, water content, Atterburg limits, unconfined compression tests, sieve, and other routine tests were performed during soil processing at New Orleans District. Compaction tests were performed on lean clay samples from trench 13T to determine the optimum moisture content. Shear tests of compacted specimens on the wet, near optimum, and dry side of optimum were conducted to obtain shear values for the core. Unconsolidated undrained (Q), consolidated undrained (R), direct shear (S), consolidation, and various other minor tests were performed on select specimens from the boring logs. Test results are presented in Appendix B.

b. Shear Strength Concerns. Some degree of conservatism is required when selecting design strength values for foundation clays because a reservoir imposes high loads on the uncontrolled natural materials of abutment and foundation which are generally complex, heterogeneous and often erratic. Designs, even with good exploration and testing, are based on very meager data and on idealizations. Assumptions may be only partly correct or even completely misleading. Material properties can have a wide range of values. Adjacent material properties may be completely different in compressibility, strength, and permeability. Often pertinent features of abutment and foundation are not discovered during exploration, design, and construction. As a result, unforseen difficulties can arise during operation.

c. Shear Strength of Clay Core. Compaction tests of lean clay samples from trench 13T show that the optimum water content is 15.8% at a maximum dry density of 110.4 pcf. Unconsolidated undrained tests at water contents on the dry, near optimum; and wet side yielded very large shear strengths that increase with increasing confining pressure. The behavior is more representative of a consolidated undrained test. It is very unlikely that the clay will retain such high strengths once water reaches the core, or that the clay can be compacted at such low moisture with the amount of rain that occurs in the region. There are two main reasons not to compact the clay to maximum compaction. The first is to prevent brittle failures at low strains. The second is to prevent failures that result from softening when the clay is exposed to water and absorbs water in excess of that at which it was compacted. Based on these concerns, the full strengths of the test results were not used,

but the clay design strength was increased from 600 to 1000 psf.

d. Strength of Embankment Soils. Information from the soil borings and trench 12T reveals that the most abundant soil types close to the ground surface that are available to construct the embankment are sands, gravels, and silts. The deposits are not clean, but instead consist of blends of the various types of soils with one type being the predominant material. Clay is mixed in varying quantities throughout the various deposits in the foundation. Some lower than average friction angles measured by the direct shear tests for foundation sands are believed to be due to clay content in the sample. Embankment materials from these deposits will contain the same amount of clay and will result in approximately the same friction angle values. A friction angle value of 28 degrees was used to account for some of the lower values measured in the foundation sands.

e. Strength of Foundation Clays. The shear strength of foundation clays was determined by Q and R tests. In general, Q tests reached peak strength within 2% axial strain and decreased in strength thereafter. Strength reductions of more than 50% were measured when tests were carried out to 20% strain. Peak undrained shear strengths for the top of the clay layer in borings DD-9U and DD-10U ranged from 1750 to 1850 psf, while the residual strengths ranged from 1300 to 1400. At the bottom of the layer, peak strengths ranged from 2400 to 3100, and residuals from 800 to 1200. The peak value of 3100 was reduced to 800 when the test was carried to 20% strain. All of the Q specimens exhibited strain softening. Friction angles, for the R tests in the same layer, ranged from friction angles of 10 to 15 degrees with cohesion intercepts ranging from 800 to 1000 psf.

f. Strength of Granular Soils in the Foundation. Direct shear tests and R tests were used to measure the shear strength of sands and lean clay specimens. Some R tests could not be performed on the lean clays because the specimens could not be trimmed due to the amount of gravel in the specimen. The friction angles were lower than expected for many specimens. Some S tests have broken back strength envelopes, a friction angle decrease with increasing normal stress. A friction angle value of 32 was used for sands that contained gravel specimens, and 35 degrees for specimens consisting mainly of gravel.

5. Stability Analyses. The length along the alignment of the reservoir was divided into reaches to account for different geologic condition and ground surface elevations. All of the reaches are shown on Plate 4. Stability analyses are presented for the construction case for the East River Terrace Reach, Plate 20 of the Feasibility Report. This plate represents the low ground conditions on both sides of the river. The east side was chosen because it has clay closer to the ground surface and is more critical from a stability viewpoint. Ground water is below the ground surface, except for one of the cases that are presented. Shear strengths are considered to be on the average to high side. This should be considered when looking at the factors of safety. The analyses that are presented are routine and do not fulfill most of the requirements of the Earth and Rock-Fill design manual. Earthquake and other more rigorous cases involving mixed strength envelopes were not evaluated at this time. All of the more complex and not routine analyses will be performed during feasibility. However, the analysis that were performed are considered adequate for cost estimates.

a. Design Manual for Dams. During the investigation, the manual entitled " Engineering and Design Stability of Earth and Rock-Fill Dams", EM 1110-2-1902, dated 1 April 1970 was used for guidance about the various methods of analyses. The recommended methods for slope stability analyses are the Wedge Method, or the optional use of the Modified Swedish Method. A minimum factor of safety of 1.4 is required for dams over 50 ft high on relatively weak foundations. The designer has the option to ignore the forces on the vertical sides of slices. The factors of safety recommended are for the Wedge and Modified Swedish methods. It is questionable whether the factors of safety are compatible with those obtained by the Spencer Method, which uses side forces between slices. Sensitivity analyses clearly show that some stability methods yield much higher safety factors than others for the same embankment configuration and loading conditions.

b. Method of Slope Stability Computation. The Corps program Utexas3 (University of Texas Analysis of Slopes - Version 3) was used to analyze slope stability. It is a general purpose two-dimensional program that is available to the government and private sector for slope stability computation analysis. By using a program that is widely used and available

to all, it will allow input data and assumptions to be checked and verified by any party when there is a dispute. Spencer's procedure is the recommended procedure and is automatically selected by the program, unless input data designates a different method. In Spencer's procedure all side forces are assumed to have the same inclination and all requirements for static equilibrium are satisfied. Although, Spencer's method of stability analysis is the recommended method for the Utexas3 program because it satisfies both force and moment equilibrium, the fact that there are other methods in the program reveals it may not yield the most accurate factor of safety for all sites and conditions. We believe that the Corps of Engineers' Modified Swedish Procedure with zero inclination for the side forces between slices will provide accurate stability values in the clay layers. This method was chosen for slope stability analyses. An automatic search with noncircular shear surfaces was used to locate the critical shear surface. No tension crack was allowed in the embankment or core, resulting in less conservative factors of safety.

c. Design Section. Various iterations were performed to find a section with common slopes that would work for the various ground surface elevations. The design section consists of a reservoir with a 24 ft. wide crown at elevation 202.8, side slopes of 1V on 3H from the crown to elevation 172.8, the elevation of the flood control pool. On the flood side, from the flood control elevation to the conservation pool elevation, the slope is 1V on 6H. The flatter slope is to reduce the chances of sudden drawdown failures that tend to occur in this zone. Below the conservation pool elevation, the slope is 1V on 4H. On the protected side, from the flood pool elevation to the conservation pool, the slope is 1V on 5H. The flatter slope in this area will increase stability and will resist seepage forces that may concentrate in the lower portion of the dam. Below the conservation pool, the slope is 1V on 3H.

d. Stability Plates. Various shear strength conditions and one high water condition are presented on Plates 5 through 8.

(1) Plate 5. This plate presents an analysis for the tailwater side using peak unconsolidated undrained strengths for the core and clay layer between elevation 92 and 110. It represents the best possible construction condition.

(2) Plate 6. The clay core strength was changed from 1000 psf, unconsolidated undrained value, to 23 degrees. This simulates strength derived from friction rather than cohesion. The strength of the foundation clay was reduced to 1300 psf to represent the residual value. These two changes simulate a direct shear value for the core and strain softening of the foundation clay.

(3) Plate 7. Plate 7 presents a relatively severe case that can occur immediately after construction and reservoir filling. The core has a friction value which is affected by the high water. Clay in the foundation still has excess pore pressure and is undergoing strain softening.

(4) Plate 8. This plate presents the results of the stability analyses towards the headwater side. Unconsolidated undrained strengths were used for the core and peak strengths for the clay layer. It shows the most favorable construction condition towards this side.

(5) Plates 9 through 11. Presented on these plates are the proposed design sections for the other reaches along the reservoir alignment.

e. Conclusions about Stability Analyses. The proposed design sections are significantly smaller than those in the Feasibility Report. The various cases show that a reservoir with the proposed slopes can be constructed at the site. Flatter slopes in the mid portion of the reservoir reduce the chances of sudden drawdown failures in the headwater side, and help control any seepage uplift forces that develop on the tailwater side. A broader base, relative to Harza's section, provides additional relief from under seepage by increasing the seepage length.

6. **Seepage Analysis.** A thorough seepage analysis is well beyond the scope and cost of this investigation. Rough calculations were performed to learn if the proposed sections have a reasonable margin of safety relative to seepage. Plates and analyses are not included due to lack of sufficient information and details for a formal presentation.

a. Seepage Related Problems. Failures attributed to

seepage and piping through embankments, foundations, abutments, and at contact points between structures and embankments are numerous. Seepage is the second leading cause of dam failures, second only to spillway capacity failures. In 1984, the Bureau of Reclamation reported that 37% of failures are attributed to seepage related problems. Seepage problems can be categorized as piping caused by local migration of materials due to the concentration of seepage forces, or instability due to water pressure. If at a particular exit point, seepage forces begin to move material, an easier path is formed and more seepage concentrates, thus, a void is formed in or under the structure. As material is removed the void or pipe increases in width and becomes more efficient at carrying alluvium. Piping begins at the downstream end of the seepage path and progresses upstream at an accelerating rate. Once piping begins it can progress to complete failure before remedial action can be taken. Seepage may be a nuisance, but it is not an automatic index of danger, since a large volume through high resistant materials at low pressure may be completely safe, while a small volume through low resistance material can lead to complete failure. A dam that has a lot of seepage may not fail, but it does not offer much reassurance to visitors and to the people who live on the downstream side. Seepage will have to be kept to a minimum to maintain the required amount of water in the reservoir.

b. Seepage Control Methods. All dams and earth foundations are subject to under seepage, seepage through the foundation. Seepage control in earth foundations is necessary to prevent excessive uplift pressures and piping through the foundation. The methods for control of under seepage in dam foundations are horizontal drains, cutoffs (compacted backfill trenches, slurry walls, concrete walls, and steel sheet piling), upstream impervious blankets, downstream seepage berms, toe drains, and relief wells. Theory and model tests show that it is necessary for a cutoff to penetrate a homogeneous foundation at least 95 percent of the full depth, before there is any appreciable reduction in seepage. To be effective the cutoff should extend through pervious to suitable impervious materials. Old abandoned river channels filled with pervious materials may be encountered in the floodplain portion of the dam foundation. The extent of these deposits is often impossible to delineate accurately during exploratory stages, and in some cases an entire deposit may be missed. Such channels filled with coarse-grained material

provide the effect of an open pipe under the dam which can be a potential hazard.

c. Proposed Methods for Seepage Control. The intent of the seepage control methods is to prevent piping through and beneath the embankment, reduce uplift pressures, minimize the amount of seepage out of the reservoir, and provide a seepage control system that is maintenance and trouble free.

(1) Seepage Through Embankment. Seepage through the embankment will be controlled mainly by the clay core. A ten foot deep inspection trench with a ten-foot bottom width will be excavated along the alignment centerline to inspect the foundation and to be backfilled with impervious soil. The core has a 4 foot crest width at elevation 192, and a 30 foot width at the original ground surface. The width at the ground surface was decreased to reduce the amount of clay required for the core. Although all of the deposits appear to have mixtures of clay, abundant homogeneous clay deposits are hard to find close to the ground surface, where it can be easily obtained. The replacement of other materials for the clay decreases the cost of the reservoir.

(2) Seepage Through the Foundation. The 70 feet deep slurry trench that was proposed in the Feasibility Report remains the method of seepage control through the foundation. Significant effort will be required during future studies to evaluate the effectiveness of the trench. In some areas, the trench may not penetrate 95% of the aquifer to become effective. Other areas may not require such a deep trench.

(3) Seepage in the Vicinity of Abutments. Abutment areas will have to be investigated in future studies to evaluate the effectiveness of the slurry trench and the possibility of seepage at contact surfaces. Upstream drainage control blankets may have to be added to the sections. The money appropriated for the slurry trench should be sufficient to provide an effective means to control seepage in these areas.

(4) Seepage in the Spillway Section. A major change was made to the seepage control method in this portion of the reservoir. The methodology used in the Feasibility Report to balance the uplift pressures along the bottom of the concrete

spillway was to add enough weight to balance the forces. Roller compacted and conventional concrete were added to provide the required weight. In some portions of the spillway, the concrete thickness is 20 feet. The cost for both types of concrete in the Feasibility Report is approximately 34.6 million dollars, for a reservoir with a crest elevation at 201 feet. The proposed method on Plate 12 controls seepage by using various methods in sequence. Seepage in the spillway section will be controlled by the steel sheet piles at the upstream and downstream ends, the clay core, slurry trench, and the gravel filter and collection pipes. Boring DD-11U, which was taken near the spillway, shows a clay layer of approximately 20 ft. thick starting at the ground surface. This layer in combination with the clay core should reduce seepage significantly and reduce the requirements on the other systems. Seepage that comes through the clay and sand stratum beneath the clay will be intercepted and collected by the gravel and pipe system. A three foot wearing surface of conventional concrete is required at the top and inclined portion of the spillway. The bottom portion, where water velocity peaks and the baffle blocks are attached, consists of a four foot thick layer of conventional concrete. Guide walls at each end of the spillway will be made of two foot thick conventional concrete anchored to roller compacted concrete. Walls are vertical and perpendicular to the reservoir alignment. At the bottom of the wall, the roller compacted concrete extends into the embankment a length equal to the height of the wall and tappers to zero thickness at the top. As a result of the reevaluation, the roller compacted concrete layer beneath the conventional concrete wearing surface was eliminated. During a more in-depth investigation, anchors or some other methods should be analyzed for cost comparisons. This is another area where the effectiveness of the slurry trench has to be evaluated. It may not be required.

(5) **Elimination of the Gravel Chimney Drain.** The inclined and horizontal gravel drains on Plates 24 through 27 of the Feasibility Report cost 23.3 million dollars for a reservoir with a crest at elevation 201 feet. This is a significant portion of the total cost. Review of all of the available boring and trench data reveals that the only type of soil that is scarce is homogeneous clay. There are many deposits of silt, sand, and gravel with sufficient permeabilities to be used effectively in embankment construction. With sufficient engineering effort in

selection of borrow pits during the design phase together with close construction and inspection supervision, these materials can be used in zones without drains. Materials will be zoned such that the permeability will progressively increase from the core out toward each slope.

d. Conclusions about Seepage Analyses. Two major changes resulted from the reevaluation of all of the data and seepage control methods. The first change eliminates the inclined and horizontal gravel chimney drains. Elimination of the roller compacted concrete beneath the conventional spillway concrete wearing surface constitutes the second major change. These changes result in significant cost savings without reducing the safety or effectiveness of the reservoir. Future studies should determine the effectiveness of the slurry trench and whether seepage control blankets are required in the abutment areas.

7. Foundation Settlement. Settlement is another area that is beyond the scope and money restraints of this study. The consolidation tests that were performed were to determine the stiffness of the deposit and aid in shear strength reliability evaluation. Consolidation tests reveal a stiff clay deposit with high preload values. Excessive foundation settlement is not expected, even in the areas where the clay layers are thicker and closer to the ground surface. The fill volume was increased by one percent to account for settlement.

8. Comparison of Proposed Reservoir to Other Reservoirs. A part of the investigation consisted of evaluating and comparing the size of the proposed reservoir to other reservoirs in the region, or to reservoirs that have similar foundation soils. This effort will show if the proposed design is overly conservative, unrealistic, and costly.

a. Ross Barnett and Arkabutla Dams. Two dams in the state of Mississippi are of similar height, the Ross Barnett and the Arkabutla. Both dams are shown on Plate 14. Harza engineers designed the Ross Barnett which was constructed in 1962. A sketch of the Ross Barnett Dam, which was received from Harza, describes the foundation soils to be alluvial deposits (sandy and silty clays over sand) overlying a thick homogeneous clay deposit of Yazoo clay. Vicksburg District personnel designed the Arkabutla dam. Stiff clay deposits make up the top 39 feet of

foundation of the dam. A cohesion value of 2400 psf was used in design for this stiff clay. The cohesion value of the top layer of Darlington clay is 1800 psf. The weaker clay should require a larger reservoir section to fulfill stability requirements. This is not the case, the Arkabutla section is larger than the proposed section for the Darlington Reservoir. It reveals that the reevaluation assumptions and design are not conservative, but on the liberal side.

b. Milford Dam. A sketch of the Milford Dam was found in Engineering Manual 1110-2-2300, and is shown on Plate 15. The dam is on the Republican River in Kansas, on what appears to be a foundation that is very similar to that under investigation. It consists of 0 to 10 feet of silt overlying 20 to 45 feet of sand. The portion of the Darlington foundation under investigation has clay layers embedded in the granular deposits, which makes it more sensitive to stability requirements than the Milford Dam foundation. The clay layers should require flatter side slopes to maintain stability. There is not much difference between the overall slope of the Milford Dam and the proposed section. The Milford Dam has additional safety features to combat seepage , such as pressure relief wells on the downstream side, and a 500 feet long impervious blanket on the upstream side. The blanket reduces the amount of seepage that enters the foundation and the relief wells reduce the uplift pressures at the downstream toe.

c. Conclusions about the Comparisons. The section proposed by Harza is the smallest section, even smaller than the Ross Barnett which was designed by Harza and constructed in Mississippi. The proposed section is smaller than the Arkabutla section. It is very similar to the Milford Dam, even though the foundation at the Darlington site is not as good. The Milford Dam also has more seepage control measures, such as a 500 ft. impervious blanket on the upstream side, a chimney drain, and pressure relief wells on the downstream side. Based on the comparisons, the proposed section is a good staring point for this level of investigation.

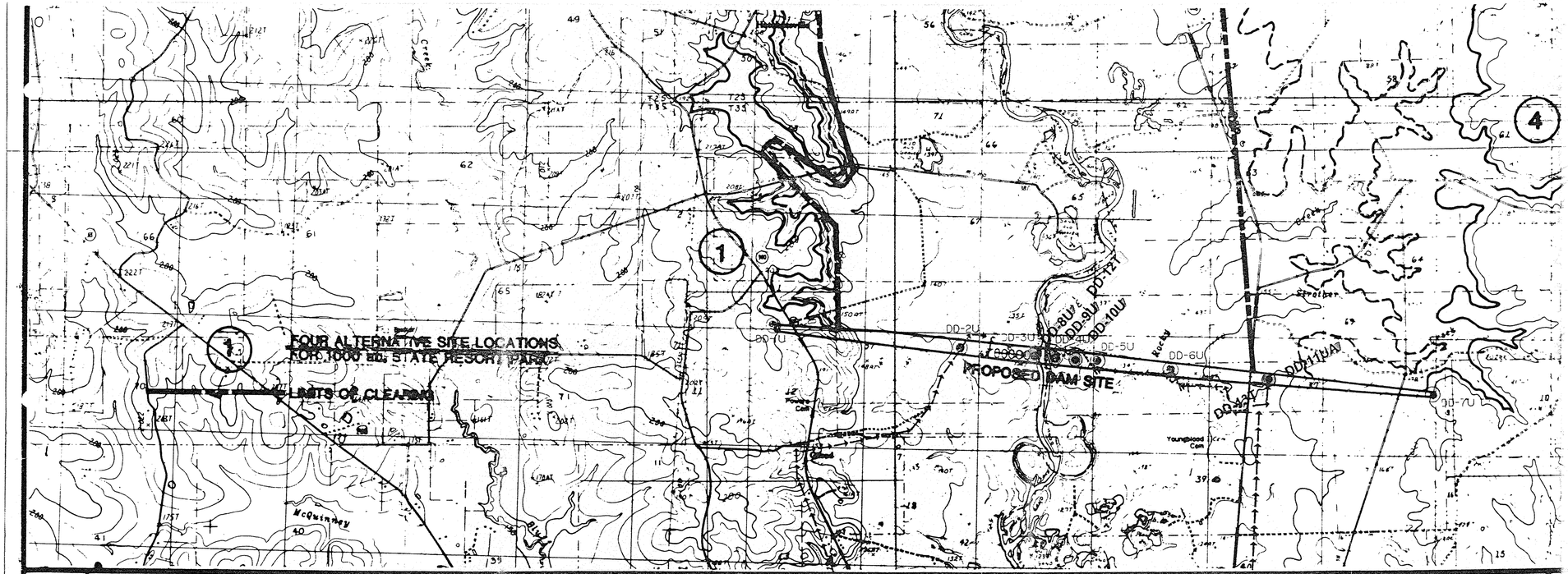
9. Conclusions. Every effort was made to provide a realistic design for a safe, efficient and low maintenance reservoir. The current sections provide adequate margins of safety for stability and seepage requirements, for this level of

investigation. Three major changes were made based on the geotechnical data obtained for this reevaluation and data from the Feasibility Report. These changes will reduce the cost of the reservoir significantly. The changes are:

1. Smaller reservoir sections.
2. Elimination of the horizontal and inclined gravel drain.
3. A much thinner concrete layer for the spillway.

The design sections that are proposed are realistic in comparison with other reservoirs that were designed and constructed at other sites by the Corps. Any reduction in size will be at the expense of safety. Possible cost savings may be achieved during future analyses of the slurry trench, but the savings could be offset by additional requirements in some portions of the foundation.

10. **Recommendations**. Future geotechnical studies should focus on the slurry trench beneath the reservoir. Additional soil borings are required, especially in the higher terraces to evaluate stability, seepage, and the effectiveness of the slurry trench. The final design may require impervious seepage control blankets, on the upstream side in the vicinity of the abutments, together with other seepage measures to insure that no seepage develops in these areas.



0 MILES

1000 0 2000 4000 6000 8000 10,000 FEET

0 KILOMETERS

Base Map: USGS 7.5 Min. Quad Sheets, Woodland, Chipola, Hatchersville, and Bluff Creek, La.

ADDITIONAL BORINGS & TRENCHES

BOR	STA	OFFSET
DD-8U	75+80	on B/L
DD-9U	82+10	on B/L
DD-10U	88+00	on B/L
DD-11UA	143+00	on B/L
DD-12T	75+80	10' N of B/L
DD-13T	143+00	10' N of B/L

LEGEND: ⊕ UNDISTURBED TYPE SOIL BORING

X (xxx.yyy) GPS POINT AND COORDINATES

→→→→→→ APROXIMATE ACCESS ROUTE

000000000000 APROXIMATE AREA TO BE DOZED

Additional Boring Locations 6/96

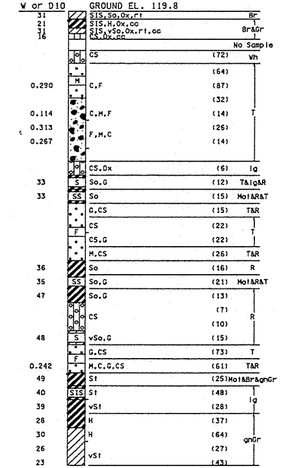
AMITE RIVER FLOOD CONTROL

PLATE G-1

CONCEPTUAL RESERVOIR PLAN ALTERNATIVE R-1A

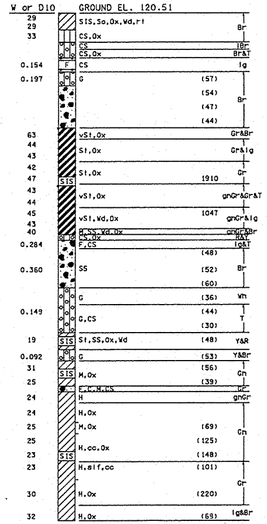
PLATE:1

BOR. DD-8U (97-08)
STA.0+00
ON C/L
WATER TABLE 1.5FT.
27JAN97

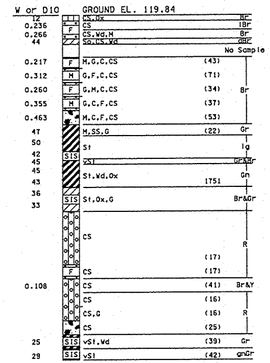


BOR. DD-9U (96-24)
STA.82+00
ON B°L

21SEP96

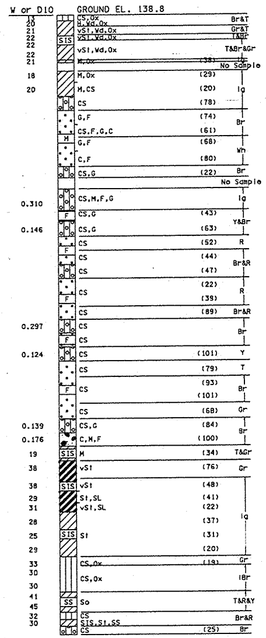


BOR. DD-10U (96-24)
STA.88+00
ON B/L
WATER TABLE 8.6 FT.
27SEP96



BOR. DD-11U (96-24)
STA.143+00
ON B°L

23SEP96



CH - Fat Clay
CL - Lean Clay
GP - Poorly Graded Gravel
ML - Silt, Low Plasticity
SM - Silty Sand
SP - Sand, Poorly Graded

AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR RE-EVALUATION STUDY

UNDISTURBED BORINGS

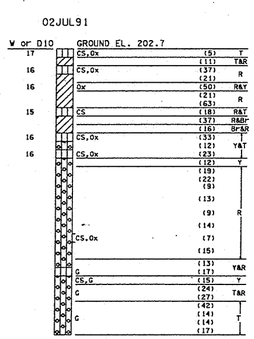
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

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DRAWN BY:	GTH		CADD FILE: DDUB.DGN
CHECKED BY:	GTH	DATE:	FILE NO.

PLATE 2

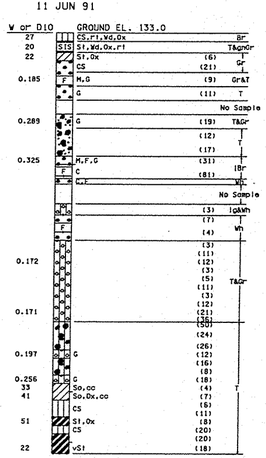
BOR. DD-1U
STA. 0+17
SEE MAP

02JUL91



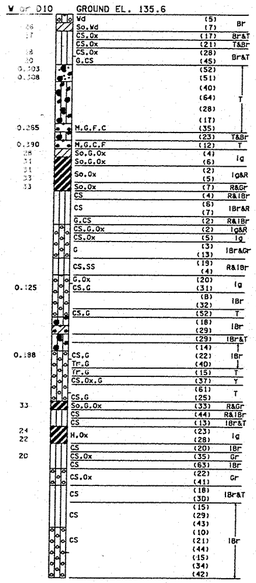
BOR. DD-2U
STA. 53+90
SEE MAP

11 JUN 91

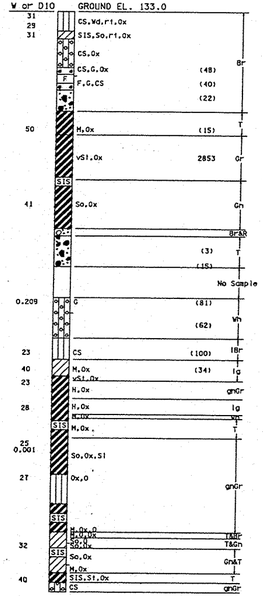


BOR. DD-3U
STA.75+90
SEE MAP

21JUN91

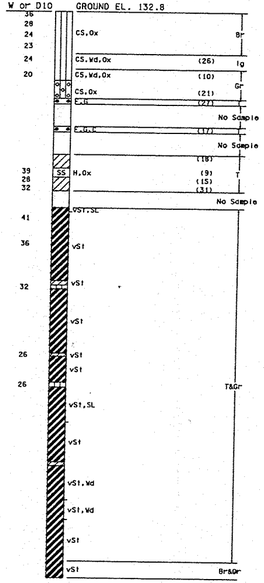


BOR. DD-4U
STA.82+00
SEE MAP
WATER TABLE 13.0 FT.
15MAY91



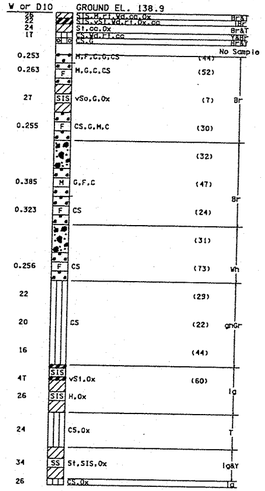
BOR. DD-5U
STA.93+12
ON C/L LEVEE

05JUN92

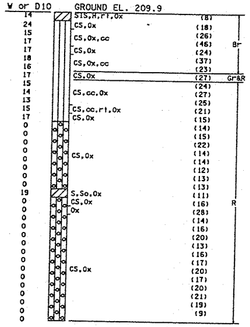


BOR. DD-6U
STA.115+00
SEE MAP

24APR91



BOR. DD-7U
STA.190+82
SEE MAP
WATER TABLE 8.0 FT.
11JUL91



ELEVATIONS IN FEET N.G.V.D.

210 200 190 180 170 160 150 140 130 120 110 100 90 80 70 60 50 40

CH - Fat Clay
CL - Lean Clay
GC - Clayey Gravel
GM - Silty Gravel
GP - Poorly Graded Gravel
GW - Well Graded Gravel
ML - Silt, Low Plasticity
SM - Silty Sand
SP - Sand, Poorly Graded

ELEVATIONS IN FEET N.G.V.D.

210 200 190 180 170 160 150 140 130 120 110 100 90 80 70

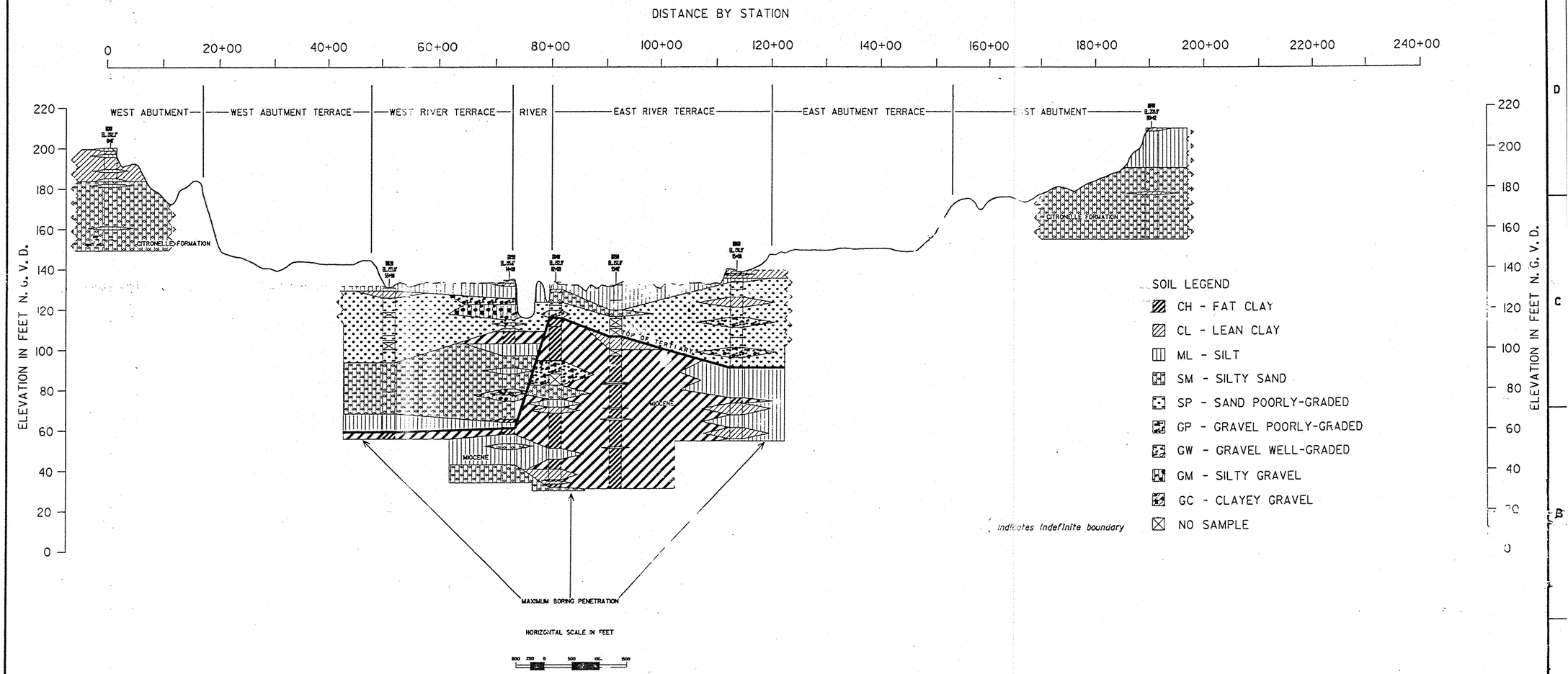
DARLINGTON RESERVOIR
FEASABILITY STUDY

GENERAL BORINGS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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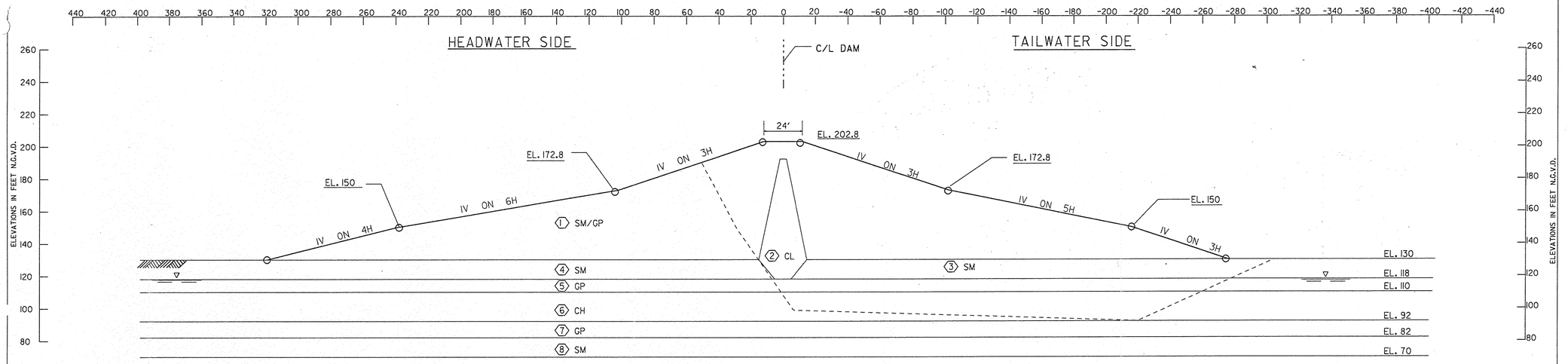
PLATE I



GENERALIZED CROSS SECTION OF DARLINGTON RESEVOIR AREA

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

DESIGNED BY: T. CREASY	PLOT SCALE: 1:20	PLOT DATE:	CADD FILE: darlington
DRAWN BY: T. CREASY			FILE NO.
CHECKED BY: D. BRITSCH	DATE: 07 JULY 1992		



FACTOR OF SAFETY = 1.808

MATERIAL No.	SOIL TYPE	UNIT WEIGHT pcf	COHESION psf	FRICTION ANGLE DEGREES
1	SM/GP	120	0	28
2	CL	115	1000	0
3	SM	122	0	32
4	SM	122	0	32
5	GP	125	0	35
6	CH	115	1800	0
7	GP	125	0	35
8	SM	122	0	29

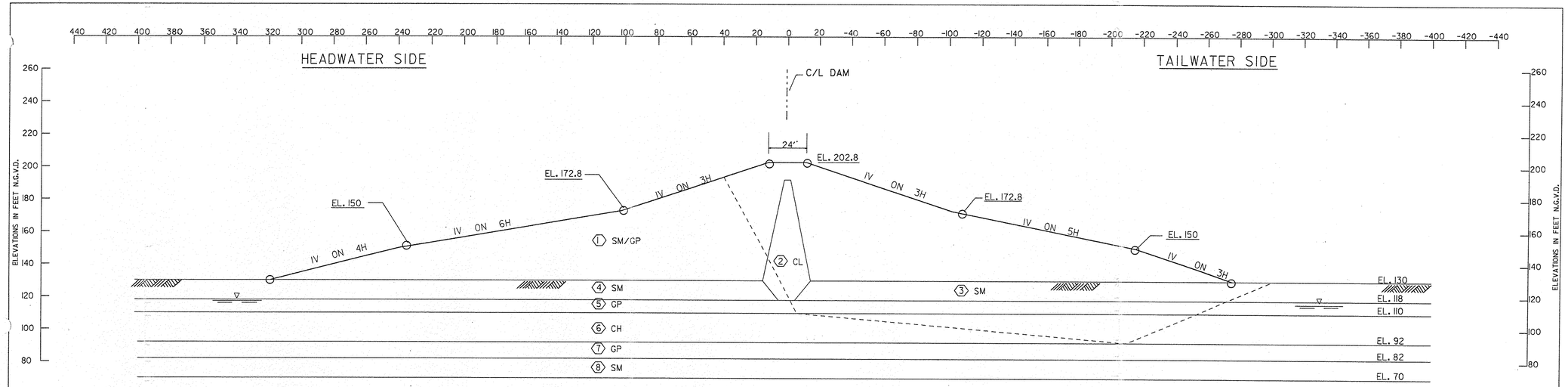
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR REEVALUATION STUDY

EAST RIVER TERRACE STABILITY ANALYSIS

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

DESIGNED BY:	PLOT SCALE:	PLOT DATE: JUNE 1997	CADD FILE: DARCORS.DGN
DRAWN BY:			FILE NO.
CHECKED BY:		DATE:	

PLATE: 5



FACTOR OF SAFETY = 1.453

MATERIAL No.	SOIL TYPE	UNIT WEIGHT pcf	COHESION psf	FRICTION ANGLE DEGREES
1	SM/GP	120	0	28
2	CL	115	0	23
3	SM	122	0	32
4	SM	122	0	32
5	GP	125	0	35
6	CH	115	1300	0
7	GP	125	0	35
8	SM	122	0	29

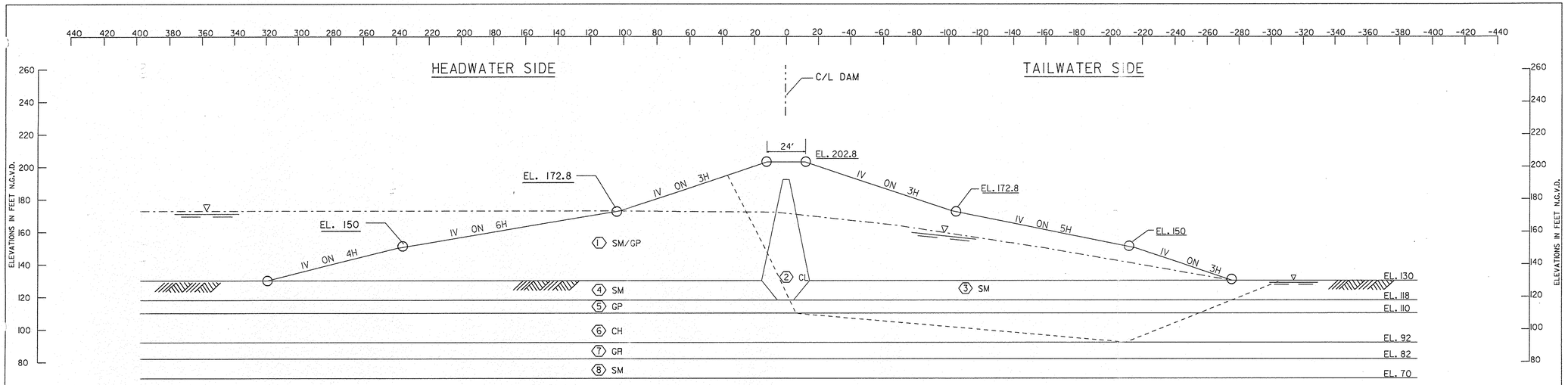
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR REEVALUATION STUDY

EAST RIVER TERRACE STABILITY ANALYSIS

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

DESIGNED BY:	PLOT SCALE:	PLOT DATE: JUNE 1997	CADD FILE: DARCOR4.DGN
DRAWN BY:			FILE NO.
CHECKED BY:	DATE:		

PLATE: 6



FACTOR OF SAFETY = 1.194

MATERIAL No.	SOIL TYPE	UNIT WEIGHT pcf	COHESION psf	FRICTION ANGLE DEGREES
1	SM/GP	120	0	28
2	CL	115	0	23
3	SM	122	0	32
4	SM	122	0	32
5	GP	125	0	35
6	CH	115	1300	0
7	GP	125	0	35
8	SM	122	0	29

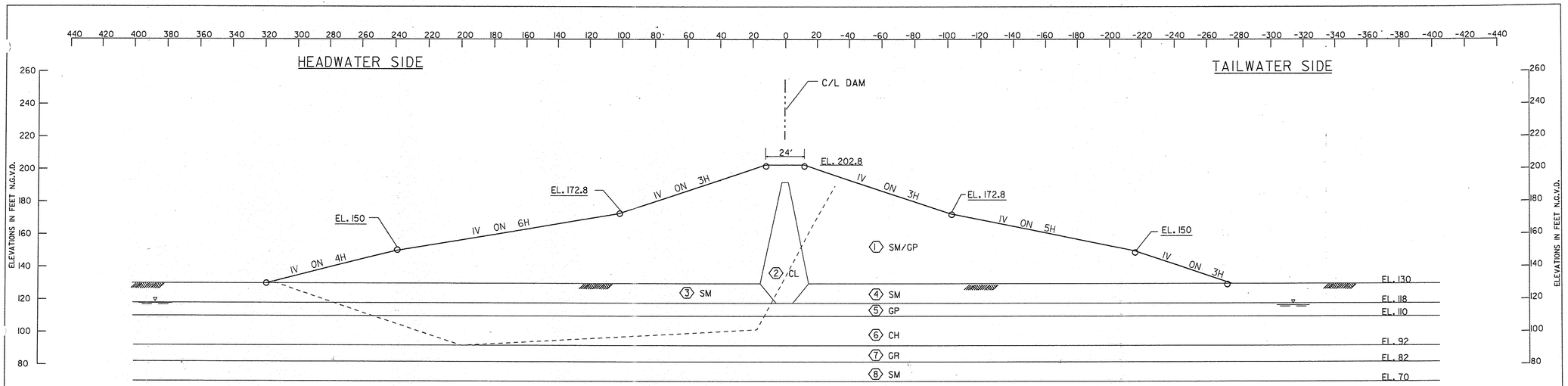
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR REEVALUATION STUDY

EAST RIVER TERRACE STABILITY ANALYSIS

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

DESIGNED BY:	PLOT SCALE:	PLOT DATE: JUNE 1997	CADD FILE: DARCORE.DGN
DRAWN BY:			FILE NO.
CHECKED BY:	DATE:		

PLATE: 7



FACTOR OF SAFETY = 1.678

MATERIAL NO.	SOIL TYPE	UNIT WEIGHT pcf	COHESION psf	FRICTION ANGLE DEGREES
1	SM/GP	120	0	28
2	CL	115	1000	0
3	SM	122	0	32
4	SM	122	0	32
5	GP	125	0	35
6	CH	115	1800	0
7	GP	125	0	35
8	SM	122	0	29

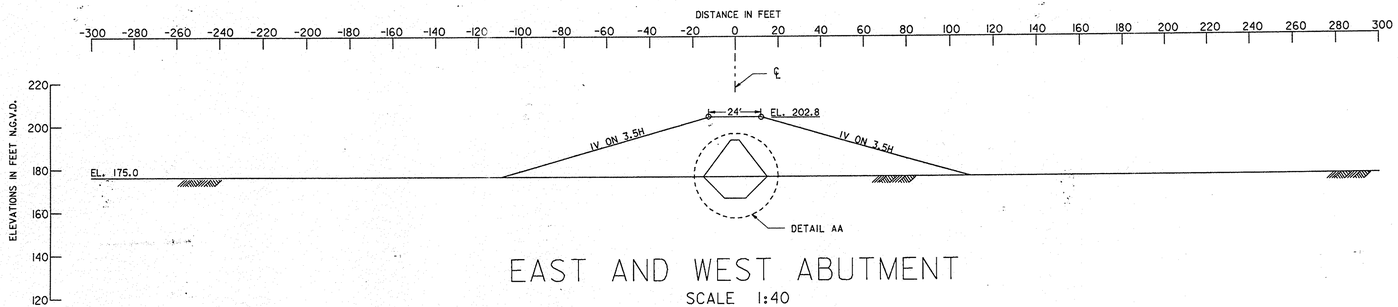
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR REEVALUATION STUDY

EAST RIVER TERRACE STABILITY ANALYSIS

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

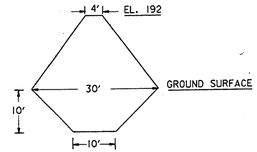
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PLATE: 8



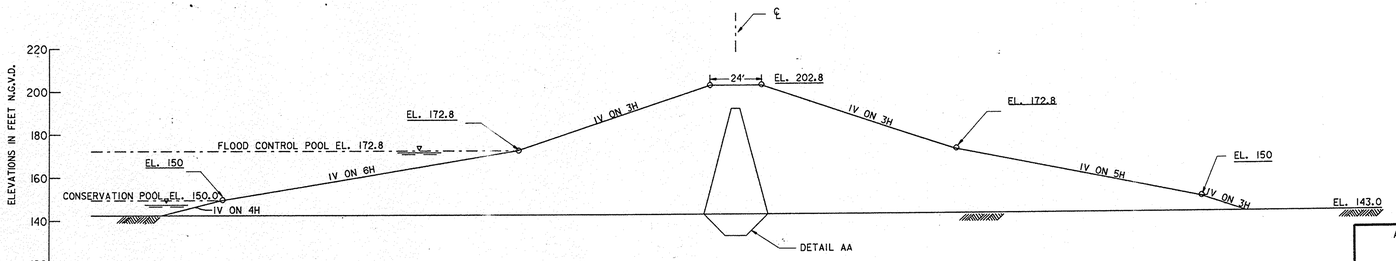
EAST AND WEST ABUTMENT

SCALE 1:40



DETAIL AA

N.T.S.



WEST ABUTMENT TERRACE

SCALE 1:40

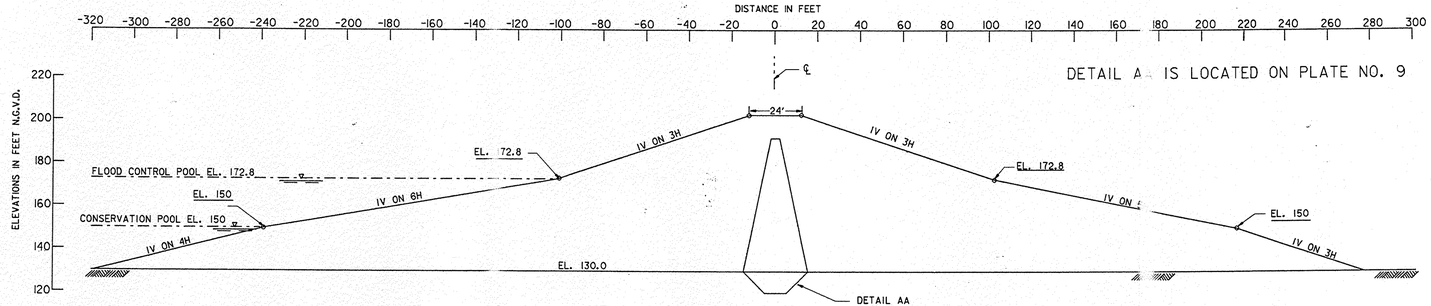
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR REEVALUATION STUDY

DESIGN SECTIONS

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

DESIGNED BY:	PLOT SCALE:	PLOT DATE: JUNE 1997	CADD FILE: PLATE1.DGN
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CHECKED BY:	DATE:		

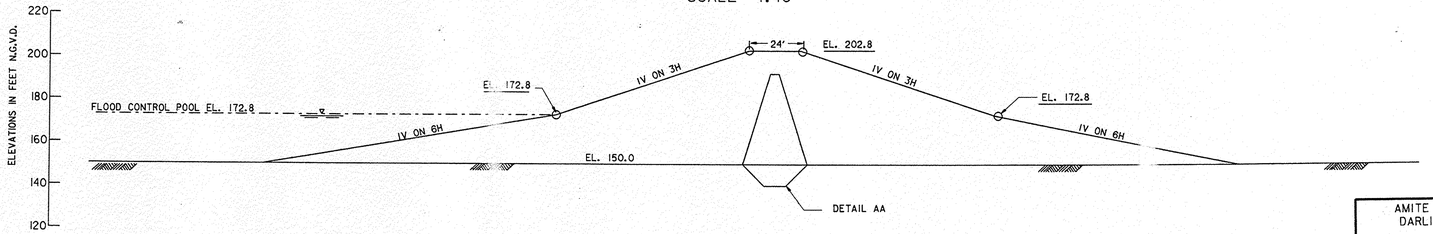
PLATE: 9



DETAIL A IS LOCATED ON PLATE NO. 9

EAST AND WEST RIVER TERRACE

SCALE 1:40



EAST ABUTMENT TERRACE

SCALE 1:40

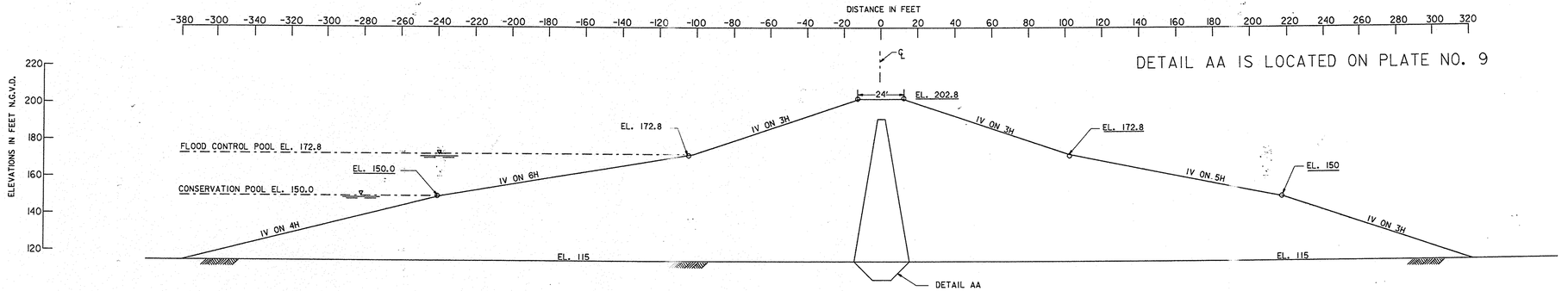
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR REEVALUATION STUDY

DESIGN SECTIONS

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

DESIGNED BY:	PLOT SCALE:	PLOT DATE: JUNE 1997	CADD FILE: PLATE2.DGN
DRAWN BY:			FILE NO.
CHECKED BY:	DATE:		

PLATE: 10



RIVER CLOSURE

SCALE 1:40

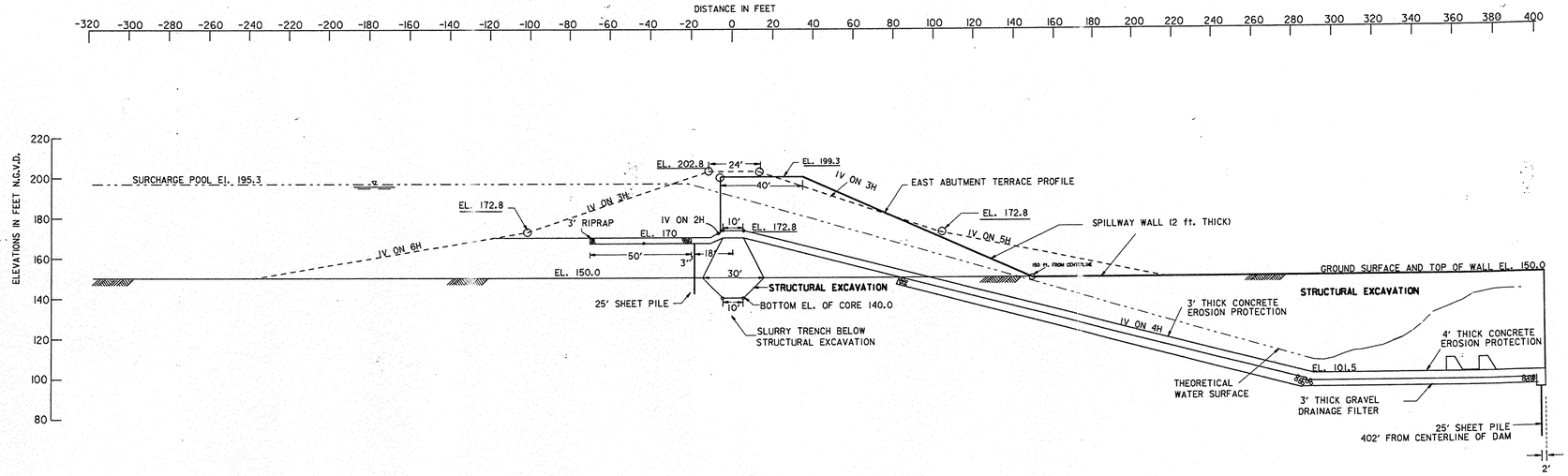
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR REEVALUATION STUDY

DESIGN SECTIONS

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

DESIGNED BY:	PLOT SCALE:	PLOT DATE: JUNE 1997	CADD FILE: PLATE3.DGN
DRAWN BY:			FILE NO.
CHECKED BY:	DATE:		

PLATE: 11



SPILLWAY SECTION

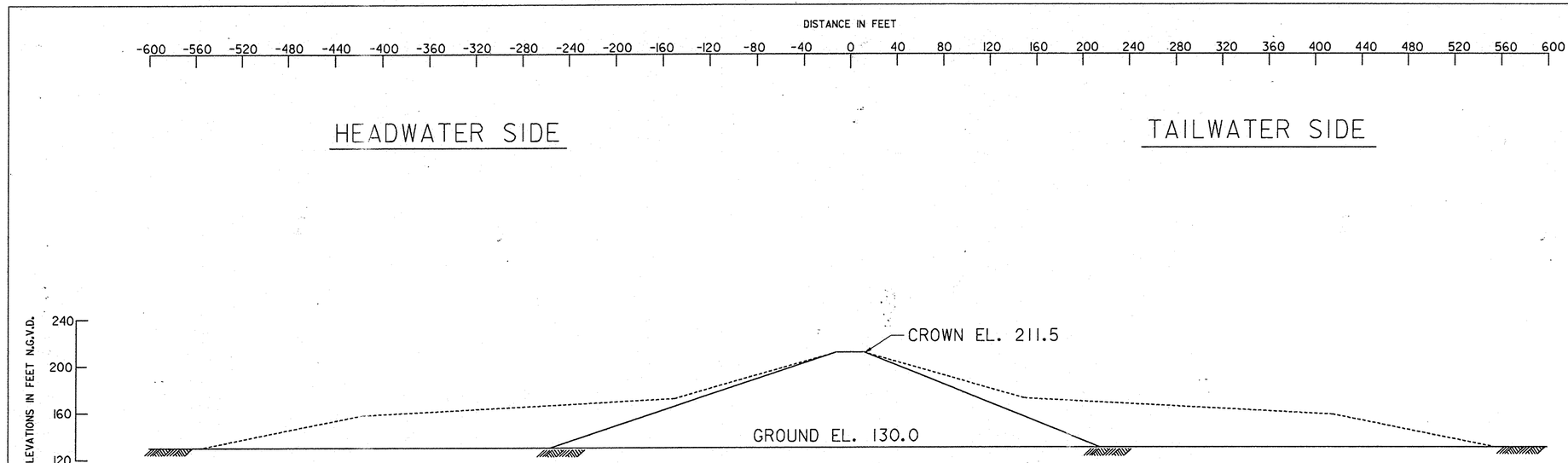
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR REEVALUATION STUDY

SPILLWAY SECTION

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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PLATE: 12



LEGEND

---------- USACE FEASIBILITY STUDY (1992)

———— HARZA's DESIGN

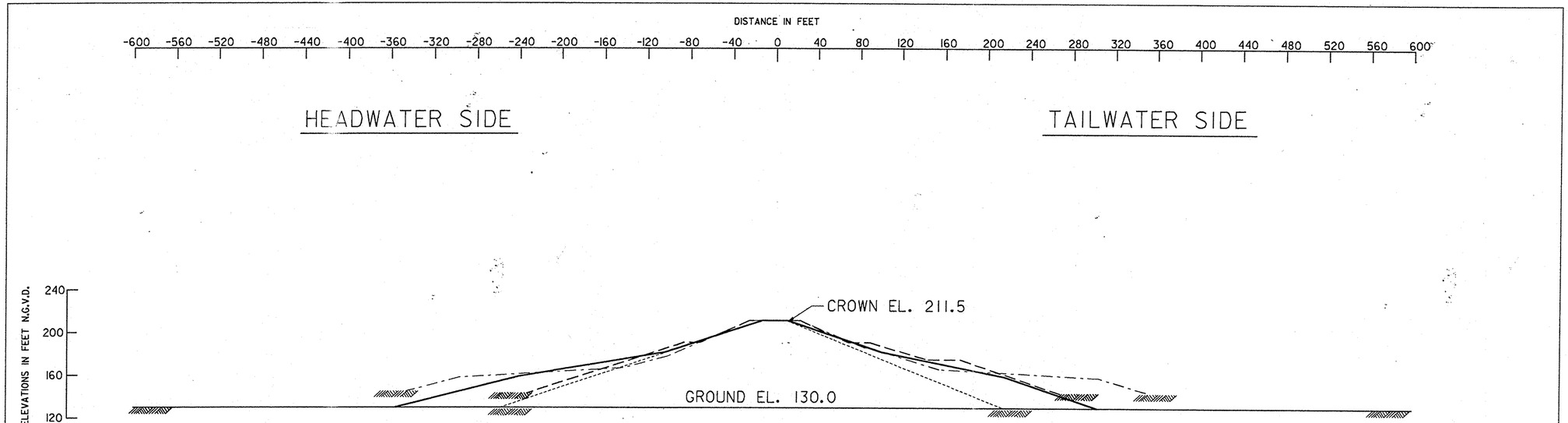
AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR REEVALUATION STUDY

COMPARISON OF FEASIBILITY DESIGNS

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PLATE: 13



LEGEND

- REVISED USACE DESIGN (1997)
- ARKABUTLA DAM (MISSISSIPPI)
- HARZA's DESIGN
- ROSS BARNETT DAM

AMITE RIVER AND TRIBUTARIES FLOOD CONTROL
DARLINGTON RESERVOIR REEVALUATION STUDY

COMPARISON OF VARIOUS DESIGNS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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DRAWN BY:			FILE NO.
CHECKED BY:	DATE:		

PLATE: 14

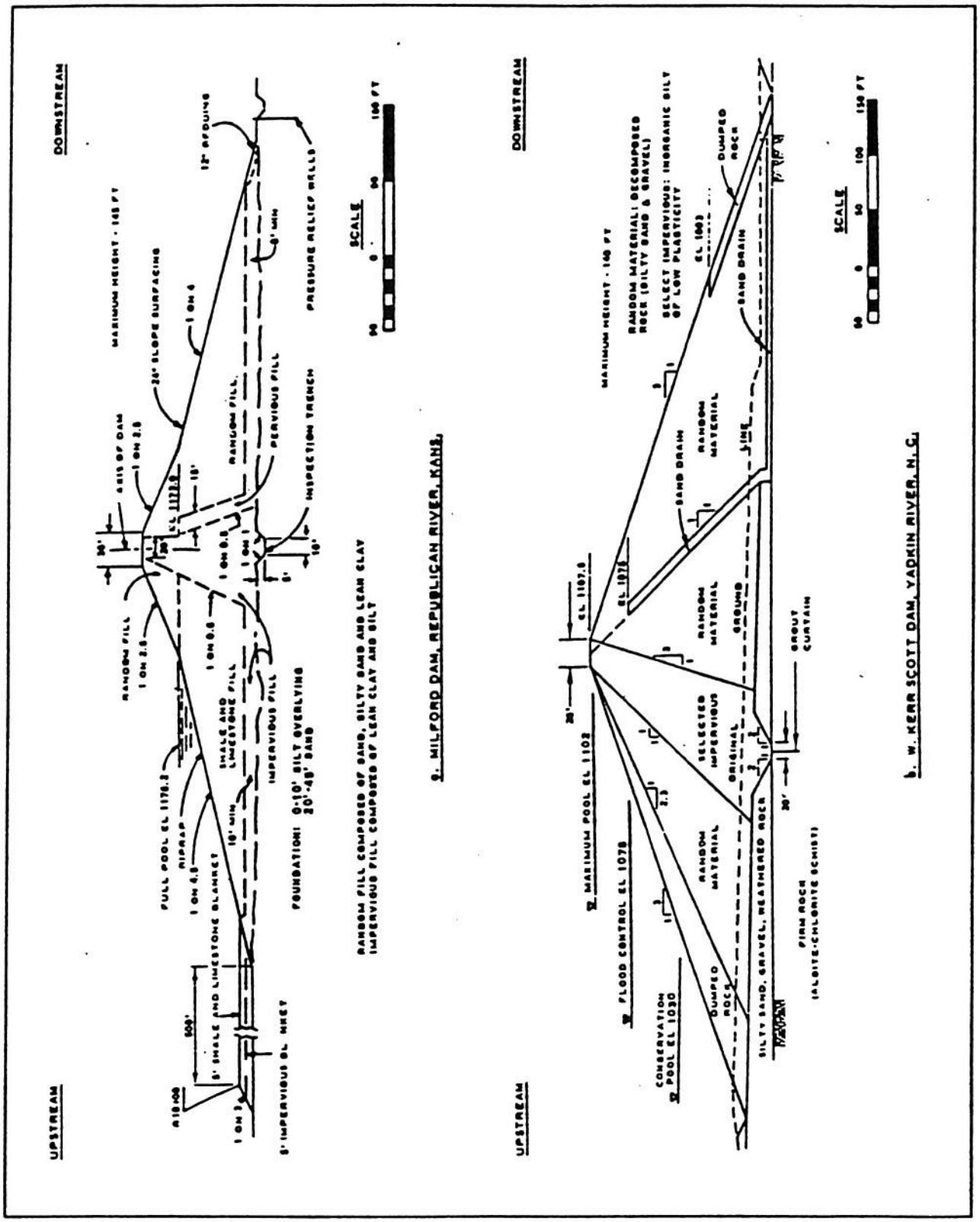


Figure 7-2. Typical embankment sections, earth dams (Milford and W. Kerr Scott Dams)

PLATE: 15

UNIFIED SOIL CLASSIFICATION

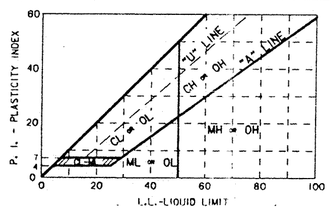
MAJOR DIVISION		TYPE	LETTER SYMBOL	SYMBOL	TYPICAL NAMES
COARSE - GRAINED SOILS (More than half of material is larger than No. 200 sieve size.)	GRAVELS (More than half of coarse fraction is larger than No. 4 sieve size.)	GRAVEL WITH FINES (Appreciable Amount of Fines)	GW		GRAVEL, Well Graded, gravel-sand mixtures, little or no fines
			GP		GRAVEL, Poorly Graded, gravel-sand mixtures, little or no fines
			GM		SILTY GRAVEL, gravel-sand-silt mixtures
			GC		CLAYEY GRAVEL, gravel-sand-clay mixtures
	SANDS (More than half of coarse fraction is smaller than No. 4 sieve size.)	CLEAN SAND (Little or No Fines)	SW		SAND, Well-Graded, gravelly sands
			SP		SAND, Poorly-Graded, gravelly sands
		SANDS WITH FINES (Appreciable Amount of Fines)	SM		SILTY SAND, sand-silt mixtures
			SC		CLAYEY SAND, sand-clay mixtures
FINE - GRAINED SOILS (More than half the material is smaller than No. 200 sieve size.)		SILTS AND CLAYS (Liquid Limit < 50)	ML		SILT & very fine sand, silty or clayey fine sand or clayey silt with slight plasticity
			CL		LEAN CLAY, Sandy Clay, Silty Clay, of low to medium plasticity
			OL		ORGANIC SILTS, and organic silty clays of low plasticity
		SILTS AND CLAYS (Liquid Limit > 50)	MH		SILT, fine sandy or silty soil with high plasticity
			CH		FAT CLAY, Inorganic clay of high plasticity
			OH		ORGANIC CLAYS of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt		PEAT, and other highly organic soil
WOOD			Wd		WOOD
SHELLS			Sl		SHELLS
NO SAMPLE			NS		No Sample Retrieved

NOTE: Soils possessing characteristics of two groups are designated by combinations of group symbols.

DESCRIPTIVE SYMBOLS

COLOR	SYMBOL
TAN	T
YELLOW	Y
RED	R
BLACK	Bk
GRAY	Gr
LIGHT GRAY	lGr
DARK GRAY	dGr
BROWN	Br
LIGHT BROWN	lBr
DARK BROWN	dBr
BROWNISH-GRAY	brGr
GRAYISH-BROWN	gyBr
GREENISH-GRAY	gnGr
GRAYISH-GREEN	gyGn
GREEN	Gn
BLUE	Bl
BLUE-GREEN	BlGn
WHITE	Wh
MOTTLED	Mot

CONSISTENCY FOR COHESIVE SOILS		
CONSISTENCY	COHESION IN LBS./SQ.FT. FROM UNCONFINED COMPRESSION TEST	SYMBOL
VERY SOFT	< 250	vSo
SOFT	250-500	So
MEDIUM	500-1000	M
STIFF	1000-2000	St
VERY STIFF	2000-4000	vSt
HARD	> 4000	H



PLASTICITY CHART

For classification of fine-grained soils in accordance with ASTM D 2487

MODIFICATIONS	
MODIFICATION	SYMBOL
Traces	Tr
Fine	F
Medium	M
Coarse	C
Concretions	cc
Rootlets	rt
Lignite fragments	lg
Shale fragments	sh
Sandstone fragments	sds
Shell fragments	slf
Organic matter	O
Clay strata or lenses	CS
Silt strata or lenses	SIS
Sand strata or lenses	SS
Sandy	S
Gravelly	G
Boulders	B
Slickensides	SL
Wood	Wd
Oxidized	Ox

NOTES:

FIGURES TO LEFT OF BORING UNDER COLUMN " W OR D_{10}*"

Are natural water contents in percent dry weight

When underlined denotes D_{10} size in mm*

FIGURES TO LEFT OF BORING UNDER COLUMNS " LL" AND " PL"

Are liquid and plastic limits, respectively

SYMBOLS TO LEFT OF BORING

▽	Ground-water surface and date observed
©	Denotes location of consolidation test**
Ⓢ	Denotes location of consolidated-drained direct shear test**
Ⓡ	Denotes location of consolidated-undrained triaxial compression test**
Ⓠ	Denotes location of unconsolidated-undrained triaxial compression test**
Ⓣ	Denotes location of sample subjected to consolidation test and each of the above three types of shear test**
FW	Denotes free water encountered in boring or sample

FIGURES TO RIGHT OF BORING

Are values of cohesion in lbs./sq.ft. from unconfined compression tests

In parenthesis are driving resistances in blows per foot determined with a standard split spoon sampler (1 3/8" I.D., 2" O.D.) and a 140 lb. driving hammer with a 30" drop

Where underlined with a solid line denotes laboratory permeability in centimeters per second of undisturbed sample

Where underlined with a dashed line denotes laboratory permeability in centimeters per second of sample remoulded to the estimated natural void ratio

*The D_{10} size of a soil is the grain diameter in millimeters of which 10% of the soil is finer, and 90% coarser than D_{10}.

**Results of these tests are available for inspection in the U.S. Army Engineer District Office, if these symbols appear beside the boring logs on the drawings.

Safety is a Part of Your Contract

TYPICAL NOTES:

1. While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local variations characteristic of the subsurface materials of the region are anticipated and, if encountered, such variations will not be considered as differing materially within the purview of the contract clause entitled "Differing Site Conditions".
2. Ground-water elevations shown on the boring logs represent ground-water surfaces encountered in such borings on the dates shown. Absence of water surface data on certain borings indicates that no ground-water data are available from the boring but does not necessarily mean that ground-water will not be encountered at the locations or within the vertical reaches of such borings.
3. Consistency of cohesive soils shown on the boring logs is based on driller's log and visual examination and is approximate, except within those vertical reaches of the borings where shear strengths from unconfined compression tests are shown.
4. Unless otherwise noted:
   a. Undisturbed borings, indicated by the letter "U", are taken with a 5" I.D. Piston Type Sampler.
   b. General type borings are taken with a 1 7/8" I.D. Tube Sampler and/or a 1 3/8" I.D. Split Spoon Sampler.

CLARK DIETERICH ENGINEERING SYSTEM
JOB IN 10 HIGHER PROFITS

REVISED 25 JUNE 92

SOIL BORING LEGEND

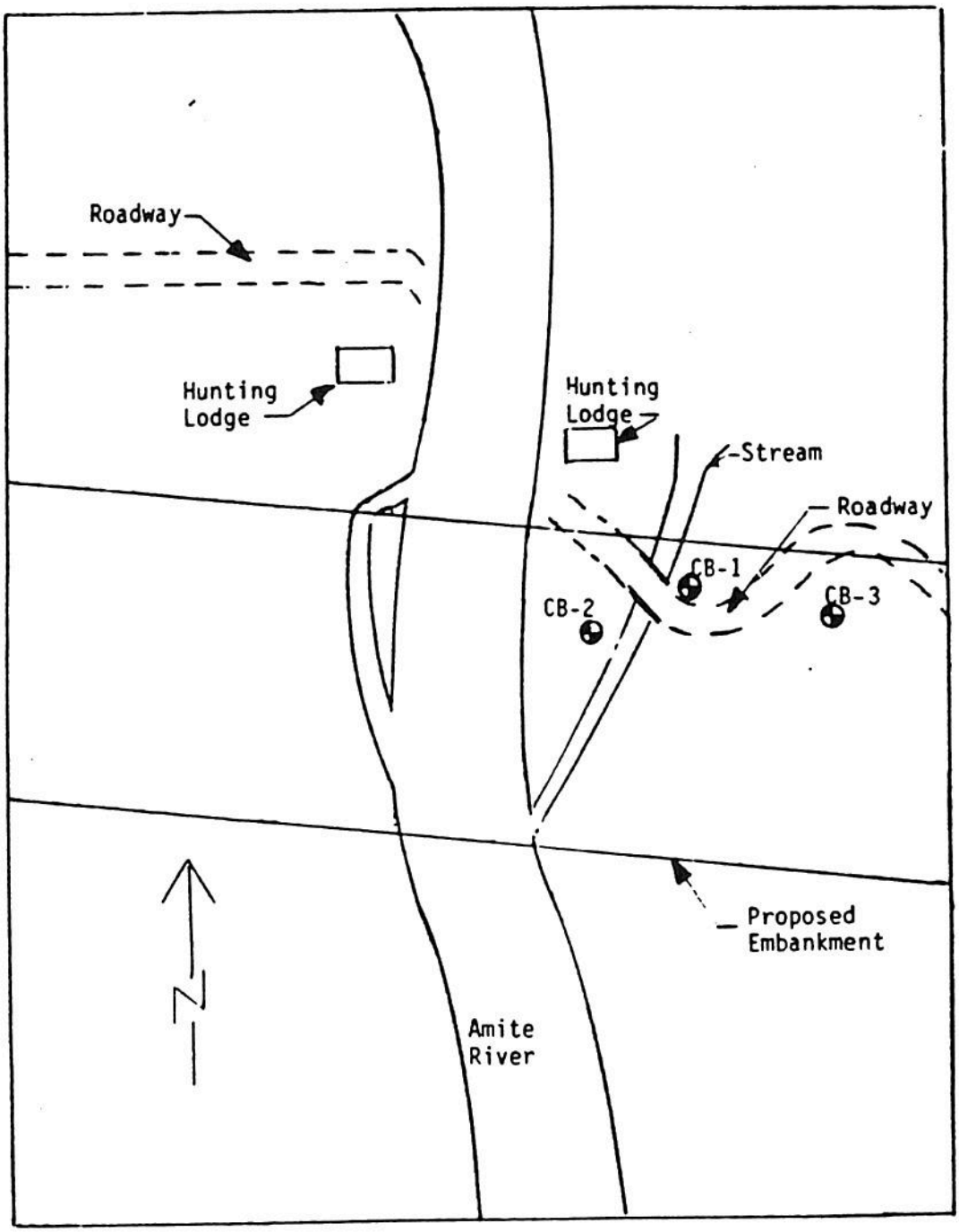
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NEW ORLEANS, LOUISIANA

DESIGNED BY:	DATE:	CADD FILE: BORLEGAL.DGN
DRAWN BY:		FILE NO. H-2-21800
CHECKED BY:		

PLATE A

Appendix A

Professional Service Industries



Deep Borehole Locations

PROJECT NAME	-Borehole Location	
Amite River Flood Control Project		
	PROJECT NO. 254-85113	DATE 7-14-89

PSI A-700-3

PLATE 2

LOG OF BORING NO. CB-1
ESPEY, HUSTON AND ASSOCIATES, INC.
AMITE RIVER FLOOD CONTROL PROJECT
AMITE LOUISIANA

TYPE BORING: Undisturbed and Split Spoon Sampling

LOCATION: See Plate 1

DEPTH	SYMBOL	SAMPLE	SOIL DESCRIPTION	% PASS	N	ATT. LMTS. LL	ATT. LMTS. PL	Mc	SHEAR STRENGTH TONS/FT^2 o Hand Pen. Uncof. Comp. ▲ Torvane 0.5 1.0	σd pcf
			LOOSE TO MEDIUM DENSE BROWN SILT WITH ORGANICS (ML)		4	24	21	34		
					3					
5			- GRAY: 7.5-8.5 ft.		18					
10			MEDIUM DENSE BROWN SAND		21					
			- POORLY GRADED WITH SILT: 9-13 ft. (SP-SM)							
15			- DENSE TO VERY DENSE GRAY WELL GRADED WITH GRAVEL: 13-24 ft. (SW)	4	32			13		
20					55					
25			SOFT RED AND GRAY LEAN CLAY (CL)			32	17	34		
			- DENSE BROWN POORLY GRADED SAND LAYER WITH GRAVEL AND CLAY: 28-29 ft. (SP-SC)							
35			- MEDIUM DENSE RED AND TAN POORLY GRADED SAND LAYER WITH SILT AND GRAVEL: 33-35 ft. (SP-SM)	7	12			17		
40			DENSE BROWN AND TAN POORLY GRADED SAND WITH SILT (SP-SM)	7	31			17		
45			- TAN WITH GRAVEL AND RED CLAY: 43-48 ft. (SP-SC)		40					
50				7	42			18		

COMPLETION DEPTH: 70 feet
DATE: 2-15-89

DEPTH TO WATER: 7 feet
DATE: 2-15-89

CONTINUED ON NEXT PAGE

LOG OF BORING NO. CB-1
ESPEY, HUSTON AND ASSOCIATES, INC.
AMITE RIVER FLOOD CONTROL PROJECT
AMITE, LOUISIANA

TYPE BORING: Undisturbed and Split Spoon Sampling

LOCATION: See Plate 1

DEPTH	SYMBOL	SAMPLE	SOIL DESCRIPTION	% PASS	N	ATT. LMTS. LL	ATT. LMTS. PL	Mc	SHEAR STRENGTH TONS/FT^2 o Hand Pen. Uncof. Comp. ▲ Torvane 0.5 1.0	σ_d pcf
55			DENSE BROWN AND TAN POORLY GRADED SAND WITH SILT (SP-SM)		34					
60										
65			VERY STIFF GRAY FAT CLAY WITH SAND, ORGANICS AND GRAVEL (CH)	80		58	26	27		99
70										
75										
80										
85										
90										
95										
100										

COMPLETION DEPTH: 70 feet
DATE: 2-15-89

DEPTH TO WATER: 7 feet
DATE: 2-15-89

PLATE 4

LOG OF BORING NO. CB-2
ESPEY, HUSTON AND ASSOCIATES, INC.
AMITE RIVER FLOOD CONTROL PROJECT
AMITE, LOUISIANA

TYPE BORING: Undisturbed and Split Spoon Sampling LOCATION: See Plate 1

DEPTH	SYMBOL	SAMPLE	SOIL DESCRIPTION	% PASS	N	ATT. LMTS. LL	ATT. LMTS. PL	Mc	SHEAR STRENGTH TONS/FT^2 o Hand Pen. Uncof. Comp. ▲ Torvane 0.5 1.0	γ_d pcf
			LOOSE BROWN SILT WITH CLAY AND ORGANICS (ML)		6					
				87	2	23	20	34		
5			MEDIUM DENSE TO DENSE BROWN POORLY GRADED SAND (SP)		12					
			- BROWN AND TAN: 8-13 ft.							
10				2	25			14		
			- WITH GRAVEL: 13-17 ft.							
15					11					
20				3	47			19		
25			SOFT RED AND GRAY SANDY LEAN CLAY (CL)							
			- WITH GRAVEL: 23-28 ft.							
30			- RED AND TAN: 28-36 ft.	62		37	17	35		
35										
			MEDIUM DENSE BROWN CLAYEY SAND (SC)							
40				28	20			23		
45			SOFT RED LEAN CLAY WITH SAND AND GRAVEL (CL)					39		
			DENSE RED AND TAN POORLY GRADED SAND WITH SILT (SP-SM)							
50					31					

COMPLETION DEPTH: 75 feet
DATE: 2-15-89

DEPTH TO WATER: 20 feet
DATE: 2-15-89

LOG OF BORING NO. CB-2
ESPEY, HUSTON AND ASSOCIATES, INC.
AMITE RIVER FLOOD CONTROL PROJECT
AMITE, LOUISIANA

TYPE BORING: Undisturbed and Split Spoon Sampling

LOCATION: See Plate 1

DEPTH	SYMBOL	SAMPLE	SOIL DESCRIPTION	% PASS	N	ATT. LMTS. LL	ATT. LMTS. PL	Mc	SHEAR STRENGTH TONS/FT^2 o Hand Pen. Uncof. Comp. ▲ Torvane 0.5 1.0	σ_d pcf
55			DENSE RED AND TAN POORLY GRADED SAND WITH SILT (SP-SM)		43					
			- WITH CLAY 53-56 ft.							
60				7	33			18		
65			VERY STIFF GRAY FAT CLAY WITH ORGANICS (CH)	95		52	24	27		
70										108
75										
80										
85										
90										
95										
100										

COMPLETION DEPTH: 75 feet
DATE: 2-15-89

DEPTH TO WATER: 20 feet
DATE: 2-15-89

LOG OF BORING NO. CB-3
ESLEY, HUSTON AND ASSOCIATES, INC.
AMITE RIVER FLOOD CONTROL PROJECT
AMITE, LOUISIANA

TYPE BORING: Undisturbed and Split Spoon Sampling

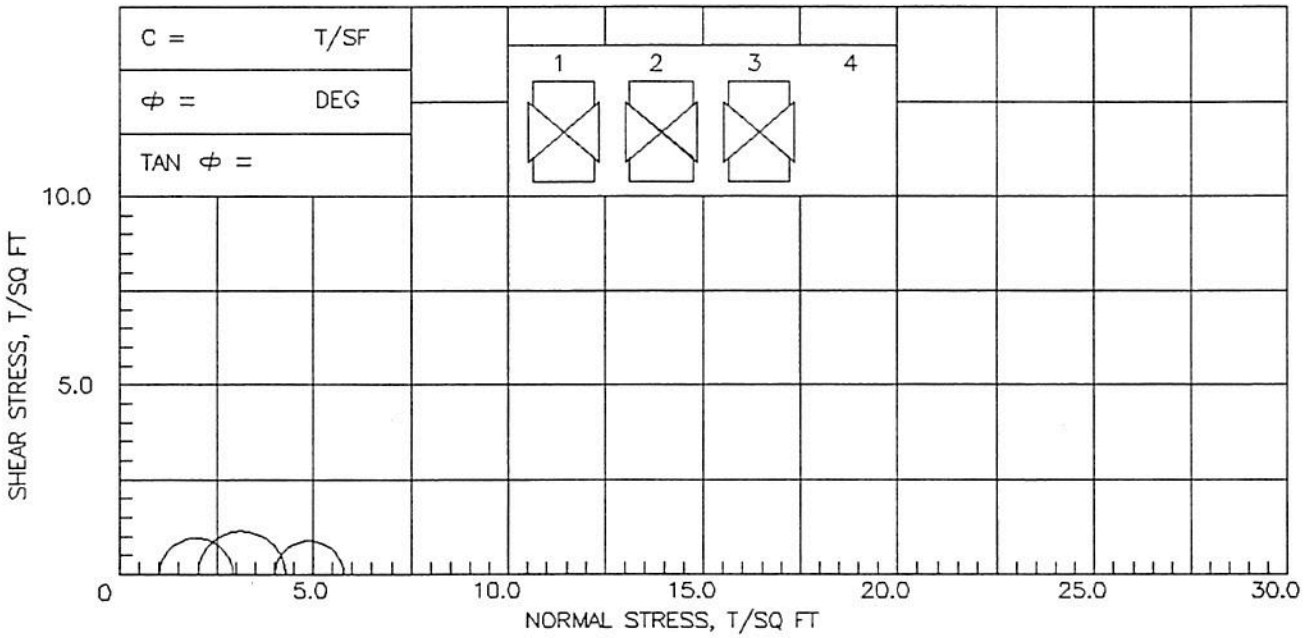
LOCATION: See Plate 1

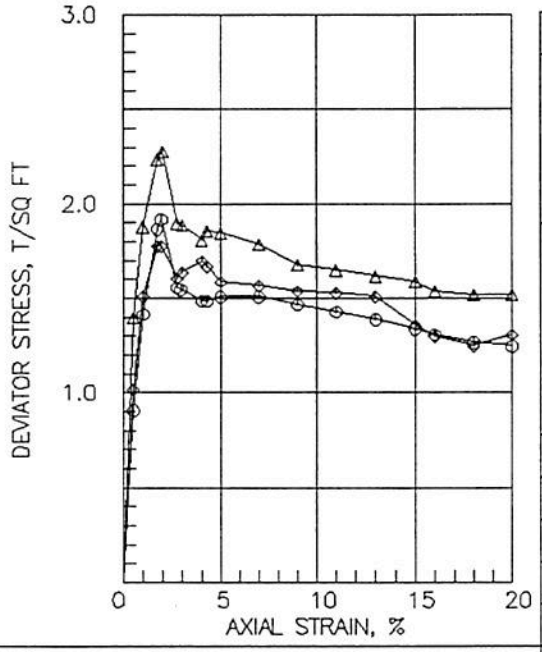
DEPTH	SYMBOL	SAMPLE	SOIL DESCRIPTION	% PASS	N	ATT. LMTS. LL	ATT. LMTS. PL	Mc	SHEAR STRENGTH TONS/FT^2 o Hand Pen. Uncof. Comp. ▲ Torvane 0.5 1.0	σ_d pcf
0			MEDIUM DENSE TO LOOSE BROWN SILT (ML)	92	4	24	21	32		
5			- WITH ORGANICS: 0-2 ft.							
			- GRAY: 4-10 ft.							
10			VERY SOFT DARK GRAY LEAN CLAY WITH ORGANICS (CL)							
15										
20			SOFT DARK GRAY SANDY LEAN CLAY WITH ORGANICS (CL)	55				72		
25			MEDIUM DENSE POORLY GRADED GRAVEL (GP)	1	13			3		
30					17					
35			MEDIUM DENSE GRAY AND TAN POORLY GRADED SAND (SP-SM)		10					
			- WITH SILT: 33-37 ft.							
40			- VERY DENSE RED AND TAN WITH CLAY: 37-42 ft.	12	61			13		
			- DENSE WITH SILT: 42-50 ft							
45					48					
50				6	47			18		

COMPLETION DEPTH: 50 feet
DATE: 2-14-89

DEPTH TO WATER: 5 feet
DATE: 2-13-89

Appendix B





SPECIMEN NO.		○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	39.3	39.3	39.3	
	DRY DENSITY, PCF	82.7	82.6	82.7	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	1.039	1.040	1.039	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		1.00	2.00	4.00	
MAX. DEV. STRESS, TSF		1.92	2.28	1.78	
TIME TO FAILURE, MIN.		4	19	18	
STRAIN RATE INCR., %			4	3	
INITIAL DIAMETER, IN.		1.35	1.35	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED–STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), LIGHT GRAY

LL 54	PL 16	PI 38	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:

RESIDUAL STRENGTH AT 25%

SPEC. 1 = 1.16 TSF

SPEC. 2 = 1.47 TSF

SPEC. 3 = 1.33 TSF

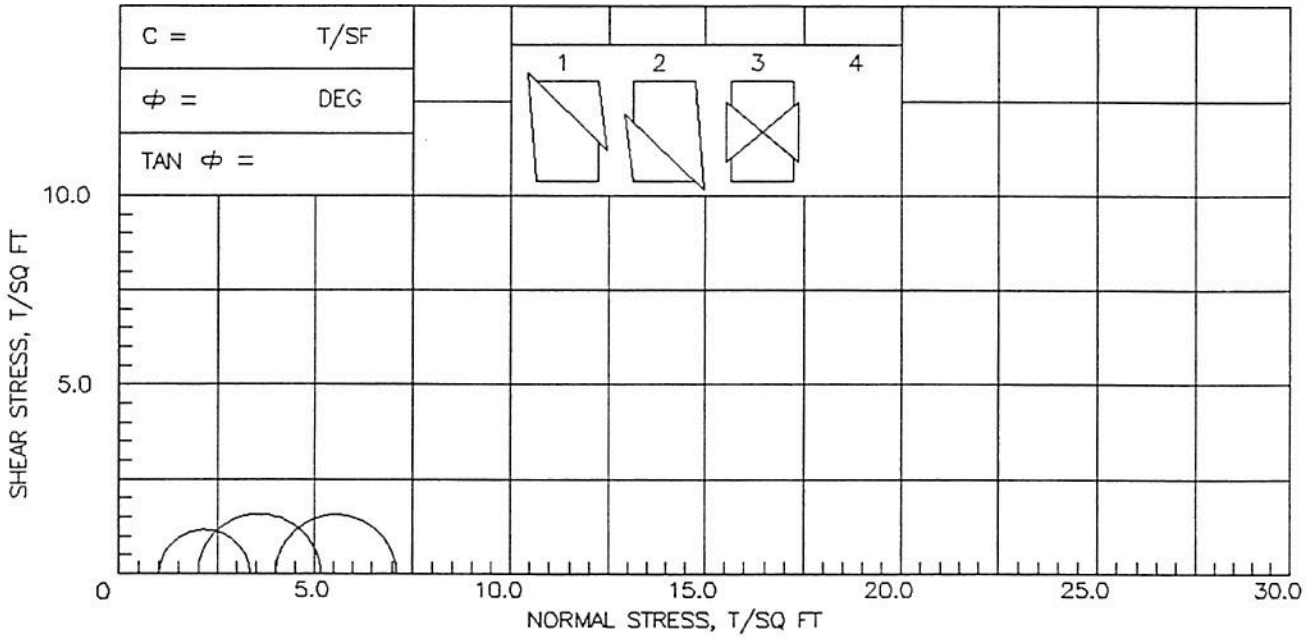
PROJECT DARLINGTON RESERVOIR

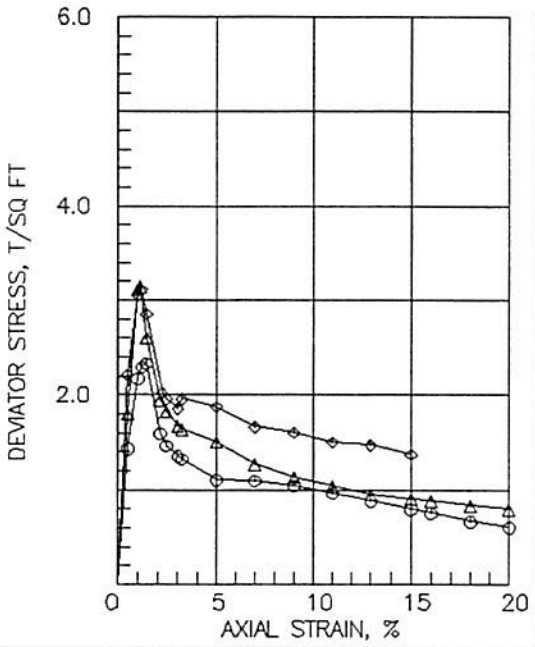
BORING NO. DD-9U | SAMPLE NO. 9C

DEPTH/ELEV 24.9 | TECH. JAL

LABORATORY USAE WES – STF/GL | DATE 24 NOV 96

TRIAXIAL COMPRESSION TEST REPORT





SPECIMEN NO.		⊙ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	50.3	51.0	49.4	
	DRY DENSITY, PCF	72.8	71.9	72.1	
	SATURATION, %	100+	100+	99.8	
	VOID RATIO	1.315	1.343	1.338	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		1.00	2.00	4.00	
MAX. DEV. STRESS, TSF		2.34	3.15	3.12	
TIME TO FAILURE, MIN.		3	12	15	
STRAIN RATE INCR., %			3	3	
INITIAL DIAMETER, IN.		1.35	1.36	1.39	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED–STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), LIGHT GRAY

LL 60	PL 24	PI 36	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:

RESIDUAL STRENGTH AT 25%

SPEC. 1 = 0.50 TSF

SPEC. 2 = 0.08 TSF

SPEC. 3 = 1.65 TSF

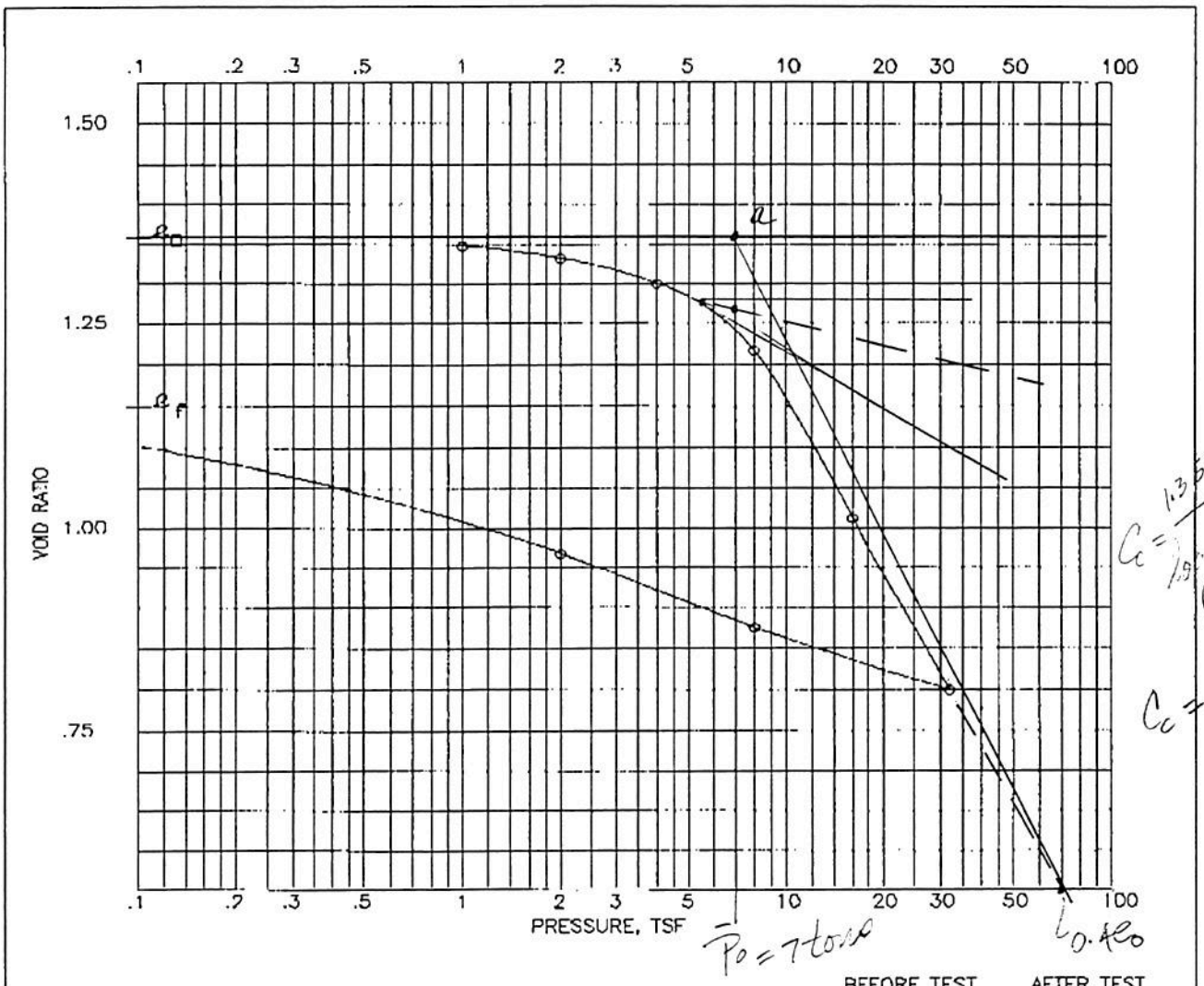
PROJECT DARLINGTON RESERVOIR

BORING NO. DD–9U SAMPLE NO. 11C

DEPTH/ELEV 33.0 TECH. JAL

LABORATORY USAE WES – STF/GL DATE 26 NOV 96

TRIAXIAL COMPRESSION TEST REPORT



	BEFORE TEST	AFTER TEST
OVERBURDEN PRESSURE, TSF	WATER CONTENT, %	48.7	42.3
PRECONSOL. PRESSURE, TSF	DRY DENSITY, PCF	71.5	78.5
COMPRESSION INDEX	SATURATION, %	96.8	99.5
TYPE SPECIMEN UNDISTURBED	VOID RATIO	1.557	1.147
DIA. IN 4.44 HT. IN 1.122	BACK PRESSURE, TSF		

CLASSIFICATION CLAY (CH), GRAY

LL PL PI

GS 2.70 (EST) D_{10}

REMARKS:

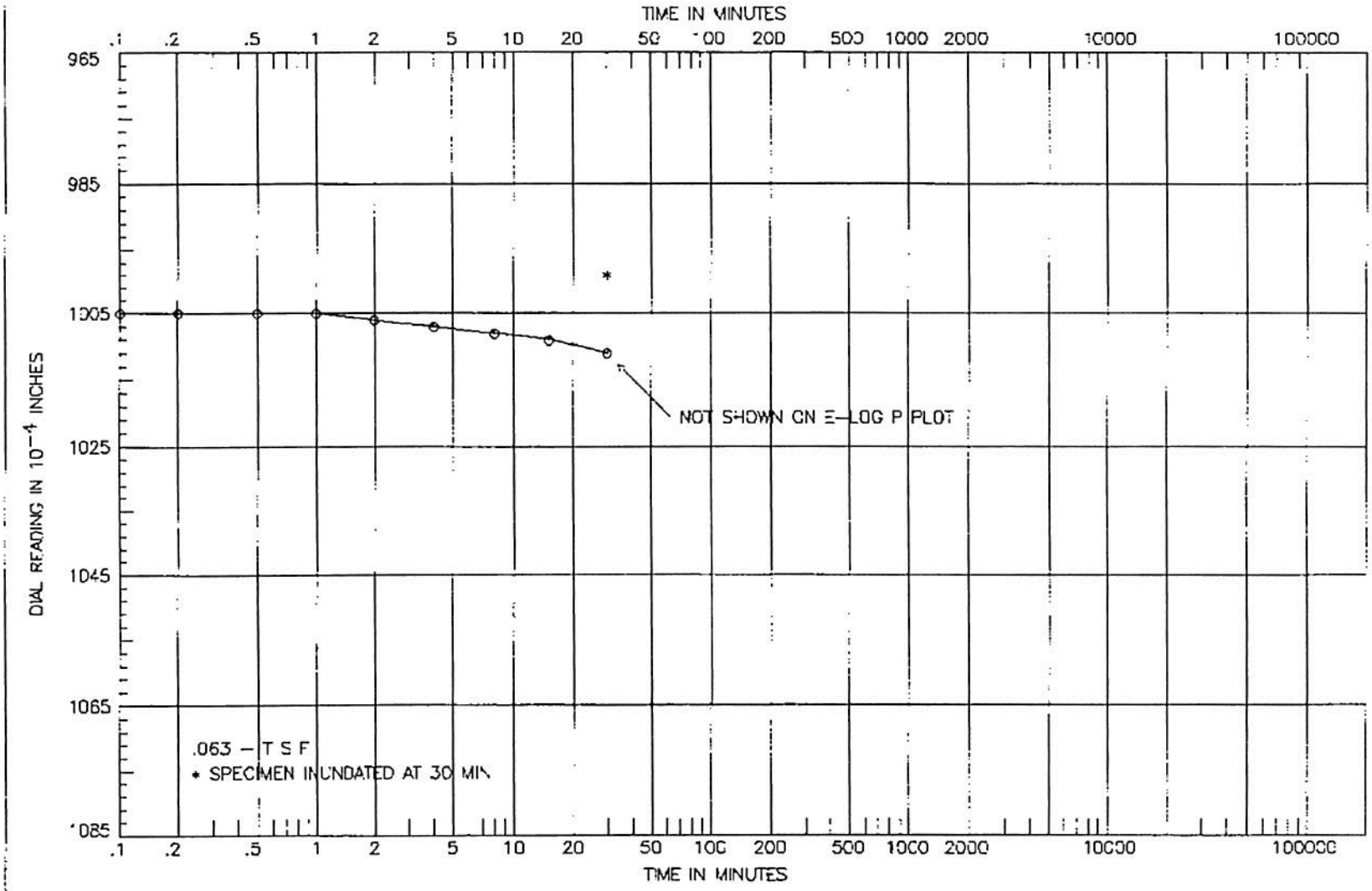
PROJECT DARLINGTON RESERVOIR

BORING NO. DD-9U SAMPLE NO. 10C

DEPTH/ELEV 28.9 TECH. JL

LABORATORY USAE WES – STF/GL DATE 22 NOV 96

CONSOLIDATION TEST REPORT

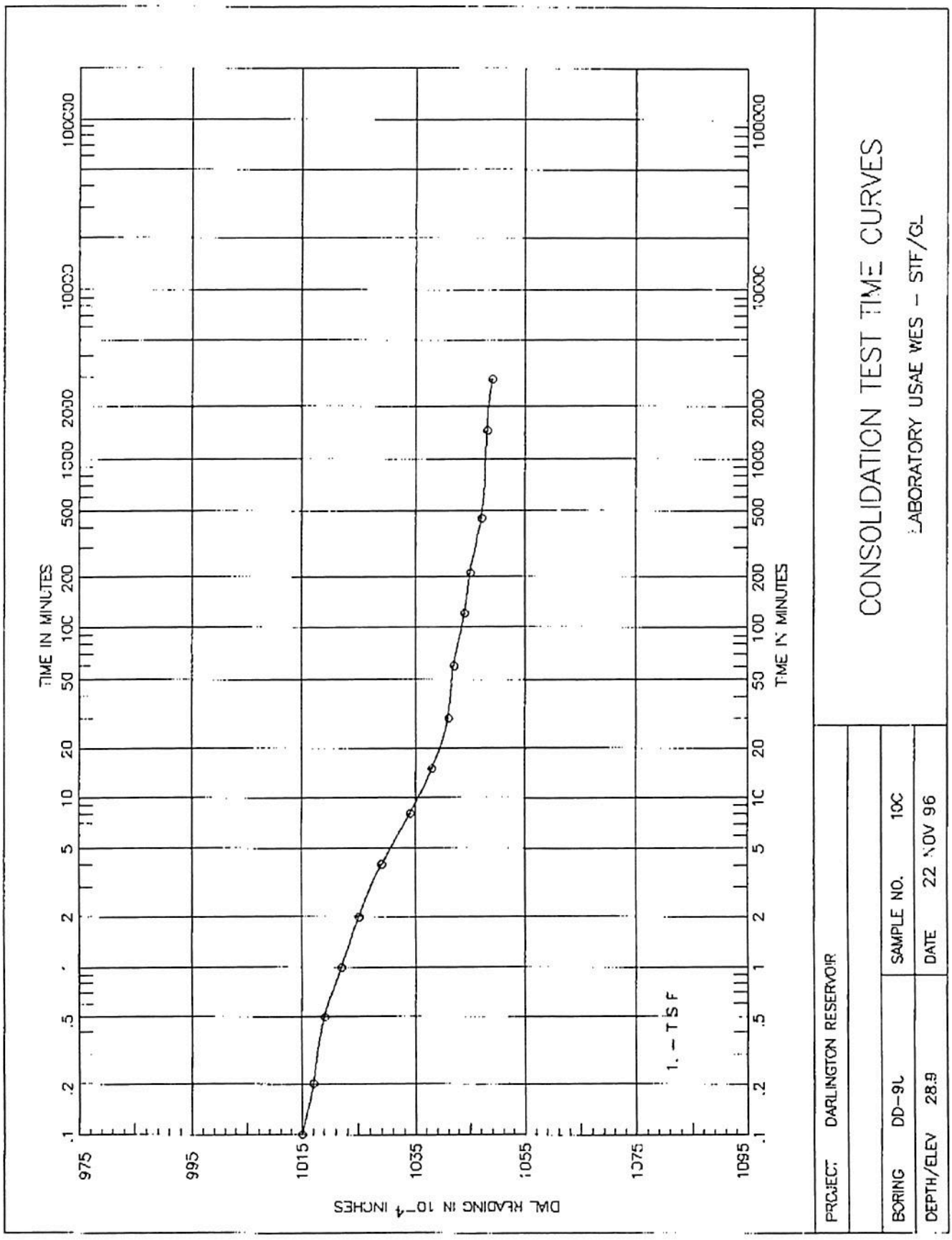


PROJECT DARLINGTON RESERVOIR	
BORING DD-9U	SAMPLE NO. 10C
DEPTH/ELEV 28.9	DATE 22 NOV 96

CONSOLIDATION TEST TIME CURVES

LABORATORY USAE WES – STF/GL

SHEET 2 OF 11

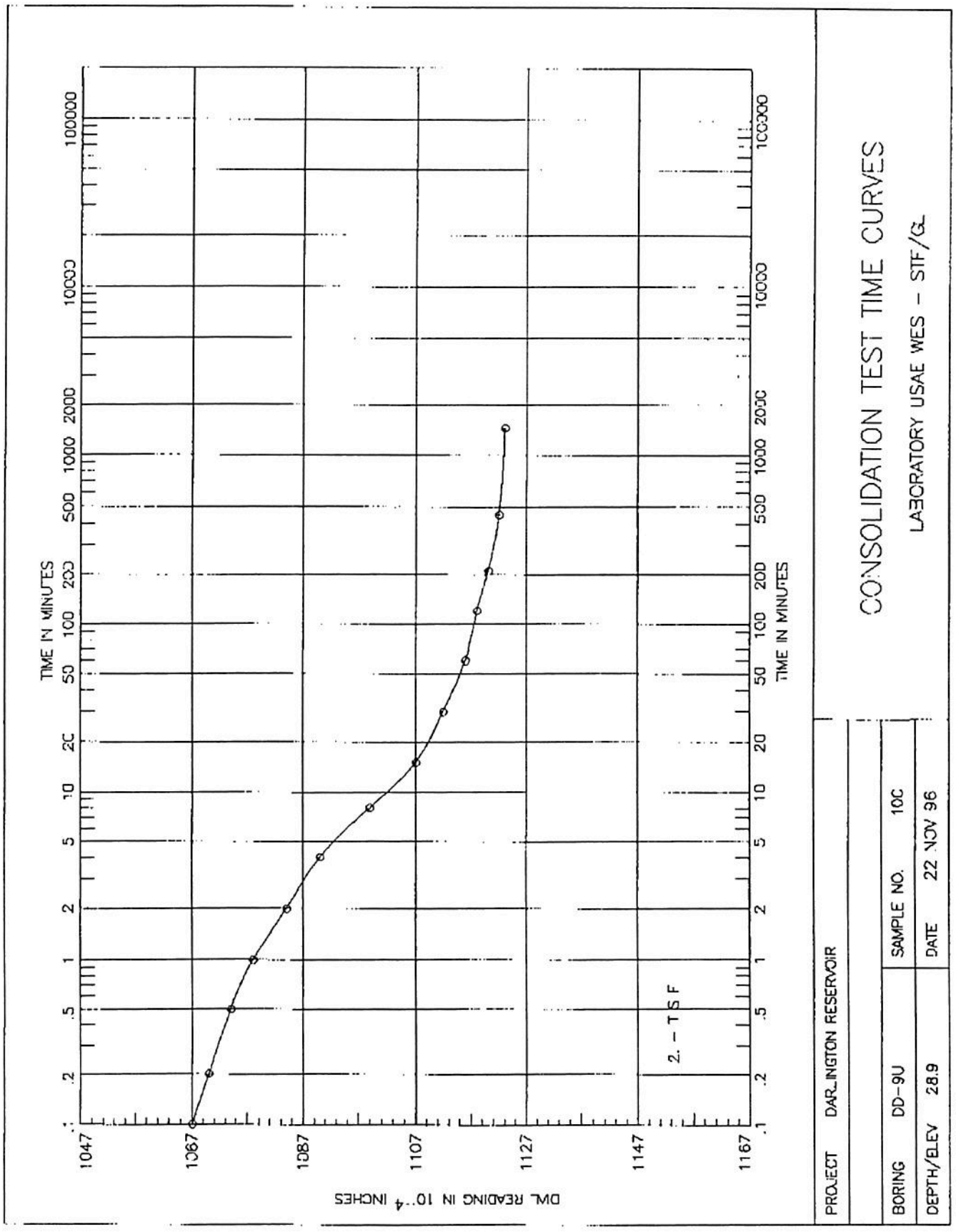


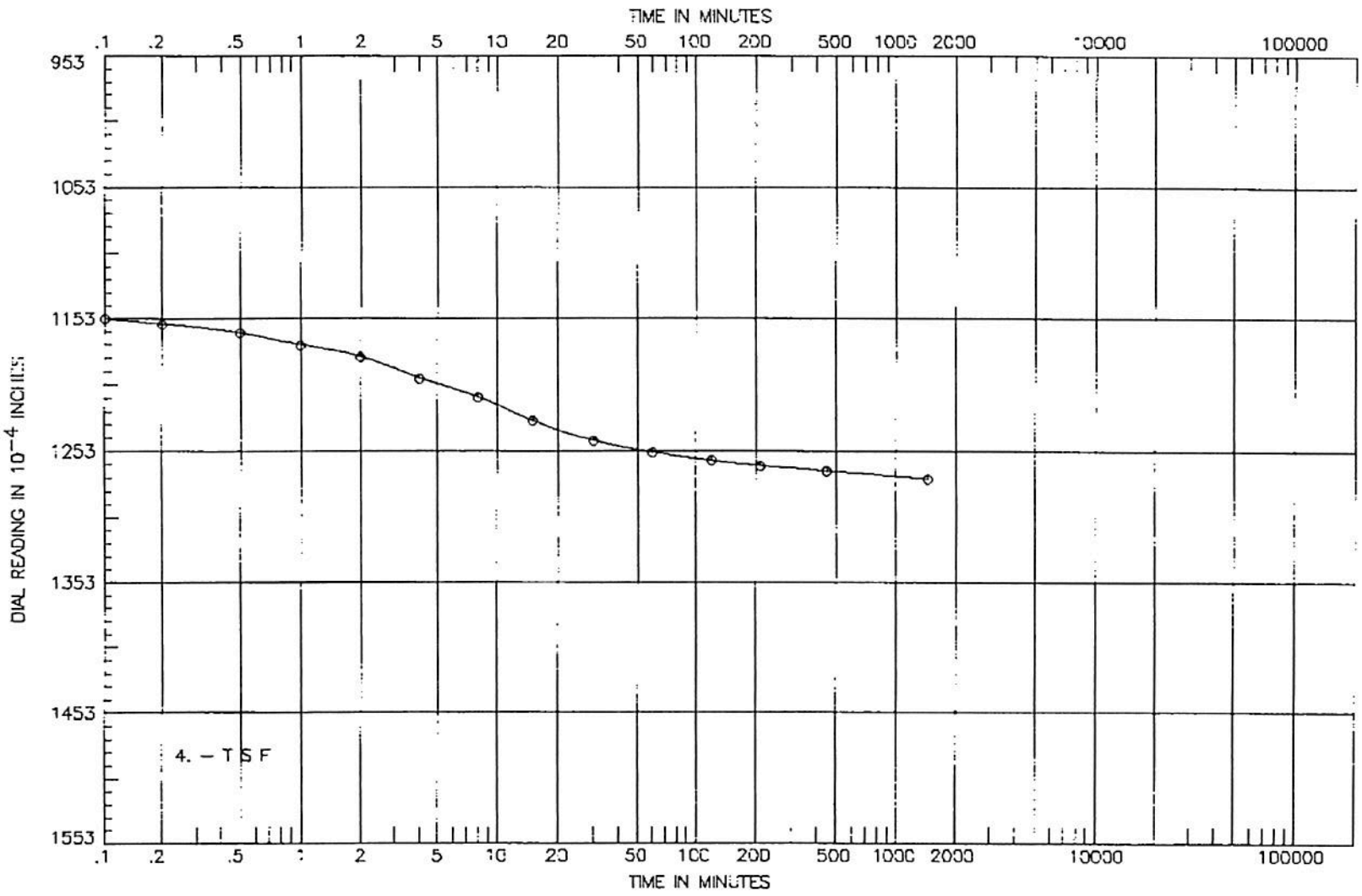
CONSOLIDATION TEST TIME CURVES

LABORATORY USAE WES - STF/GL

PROJECT: DARLINGTON RESERVOIR			
BORING DD-9L	SAMPLE NO. 10C		
DEPTH/ELEV 28.9	DATE 22 NOV 96		

SHEET 3 OF 11


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
PROJECT DARLINGTON RESERVOIR
BORING DD-9U
SAMPLE NO. 10C
DEPTH/ELEV 28.9
DATE 22 NOV 96
SHEET 4 OF 11
2. - TSF
TIME IN MINUTES
TIME IN MINUTES
DWL READING IN 10**-4 INCHES
1047
1067
1087
1107
1127
1147
1167



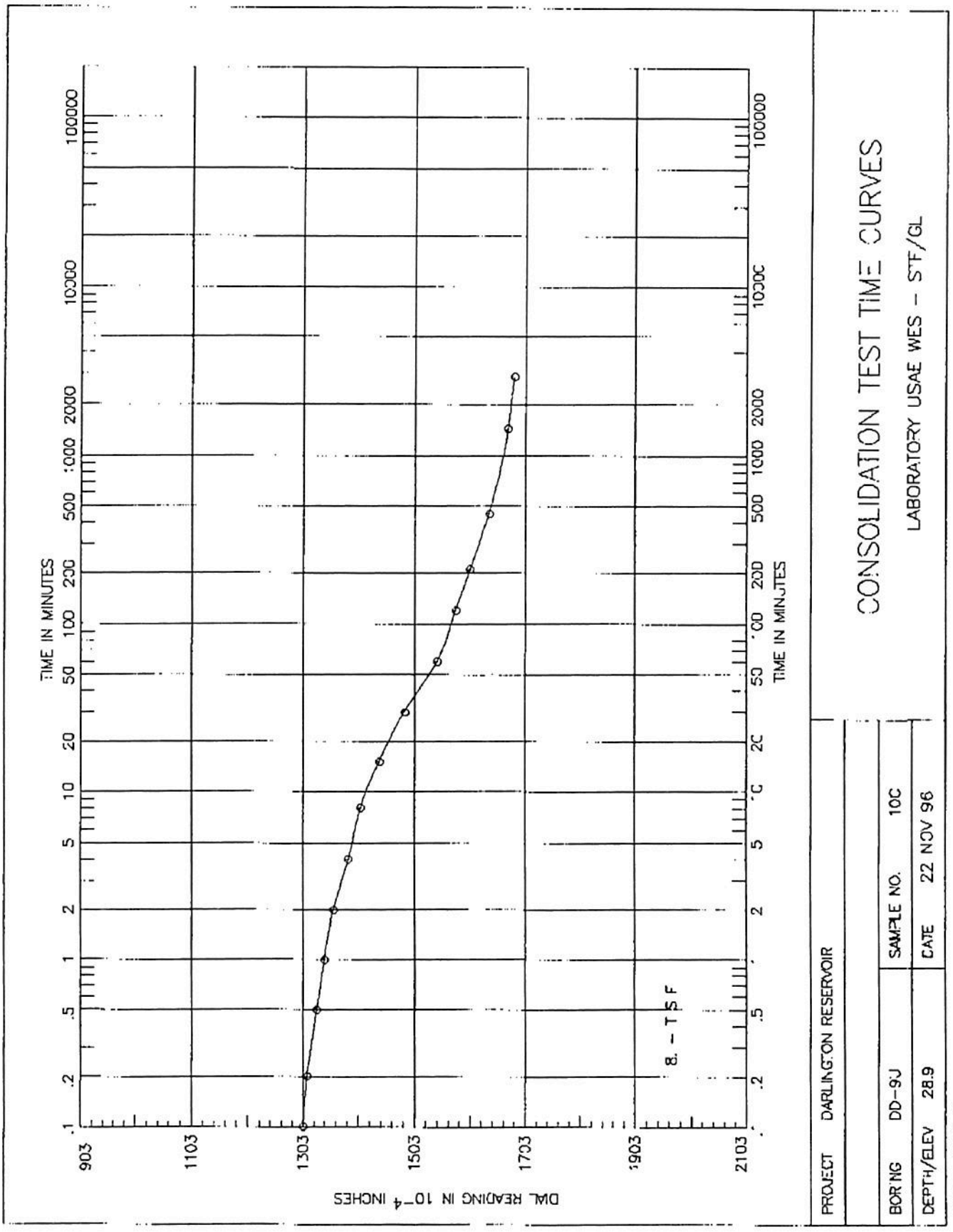
PROJECT DARLINGTON RESERVOIR	
BORING DD-9U	SAMPLE NO. 10C
DEPTH/ELEV 28.9	DATE 22 NOV 96

CONSOLIDATION TEST TIME CURVES

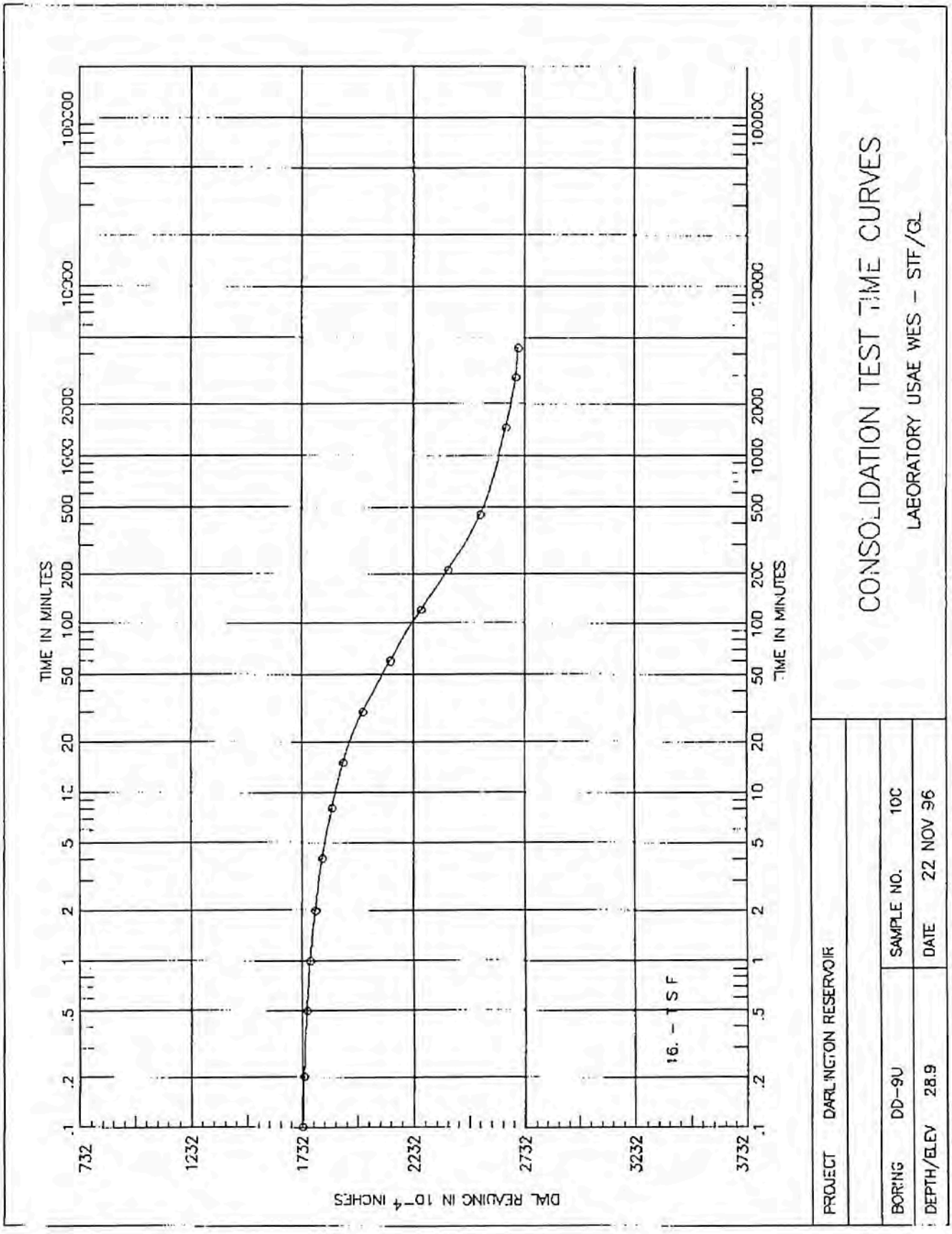
LABORATORY USAE WES – STF/GL

SHEET 5 OF 11

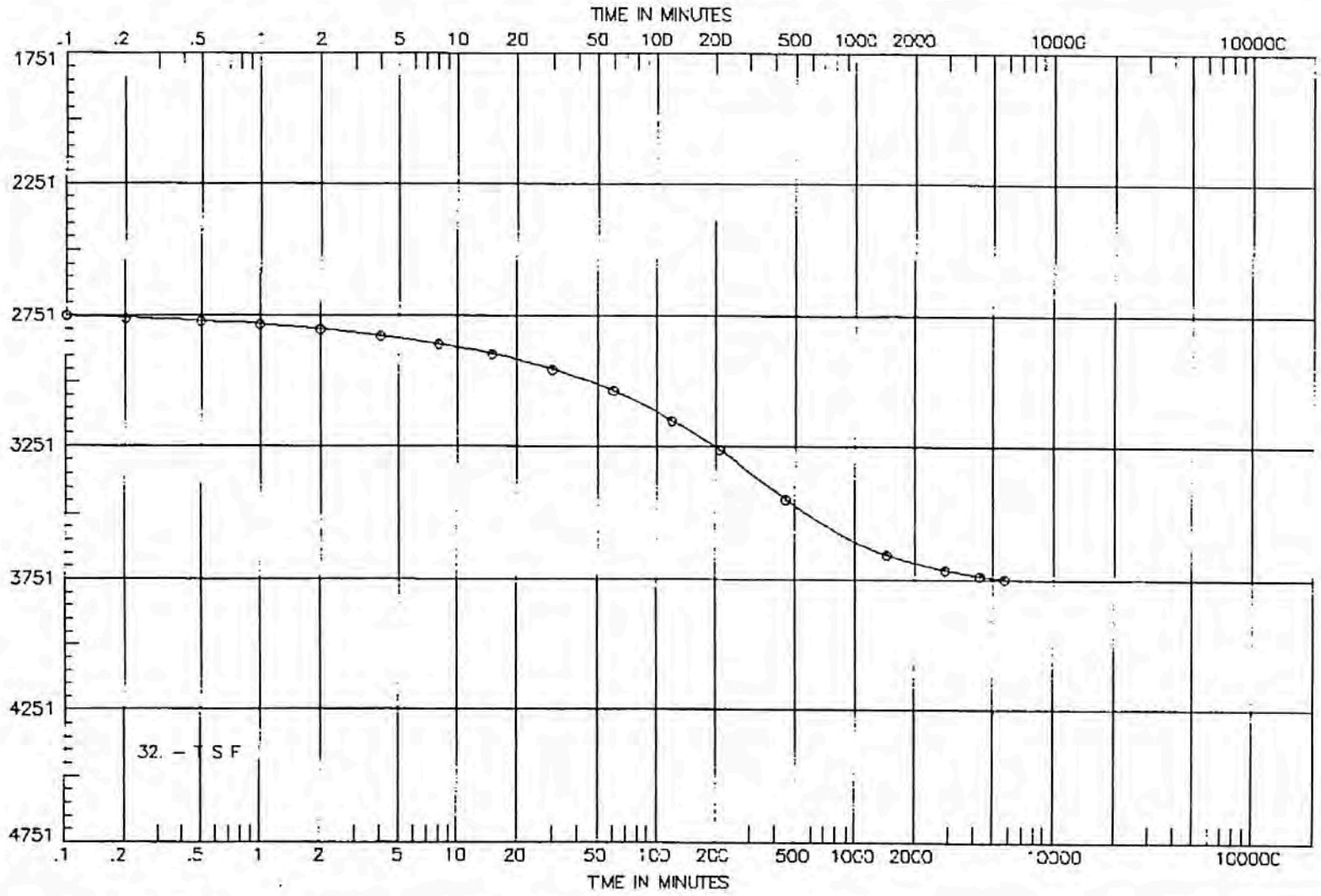
12/13 '96 11:08 ID:CEWES-GS 00 FAX:601-634-4219 PAGE 6


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
PROJECT DARLINGTON RESERVOIR
BORING DD-9J
SAMPLE NO. 10C
DEPTH/ELEV 28.9
DATE 22 NOV 96
TIME IN MINUTES
DIAL READING IN 10⁻⁴ INCHES
8. - TSF

SHEET 6 OF 11


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
PROJECT DARLINGTON RESERVOIR
BORING DD-9U
SAMPLE NO. 10C
DEPTH/ELEV 28.9
DATE 22 NOV 96
16. - T S F
TIME IN MINUTES
DIAL READING IN 10⁻⁴ INCHES
732
1232
1732
2232
2732
3232
3732
.1
.2
.5
1
2
5
10
20
50
100
200
500
1000
2000
10000
100000

SHEET 7 OF 11

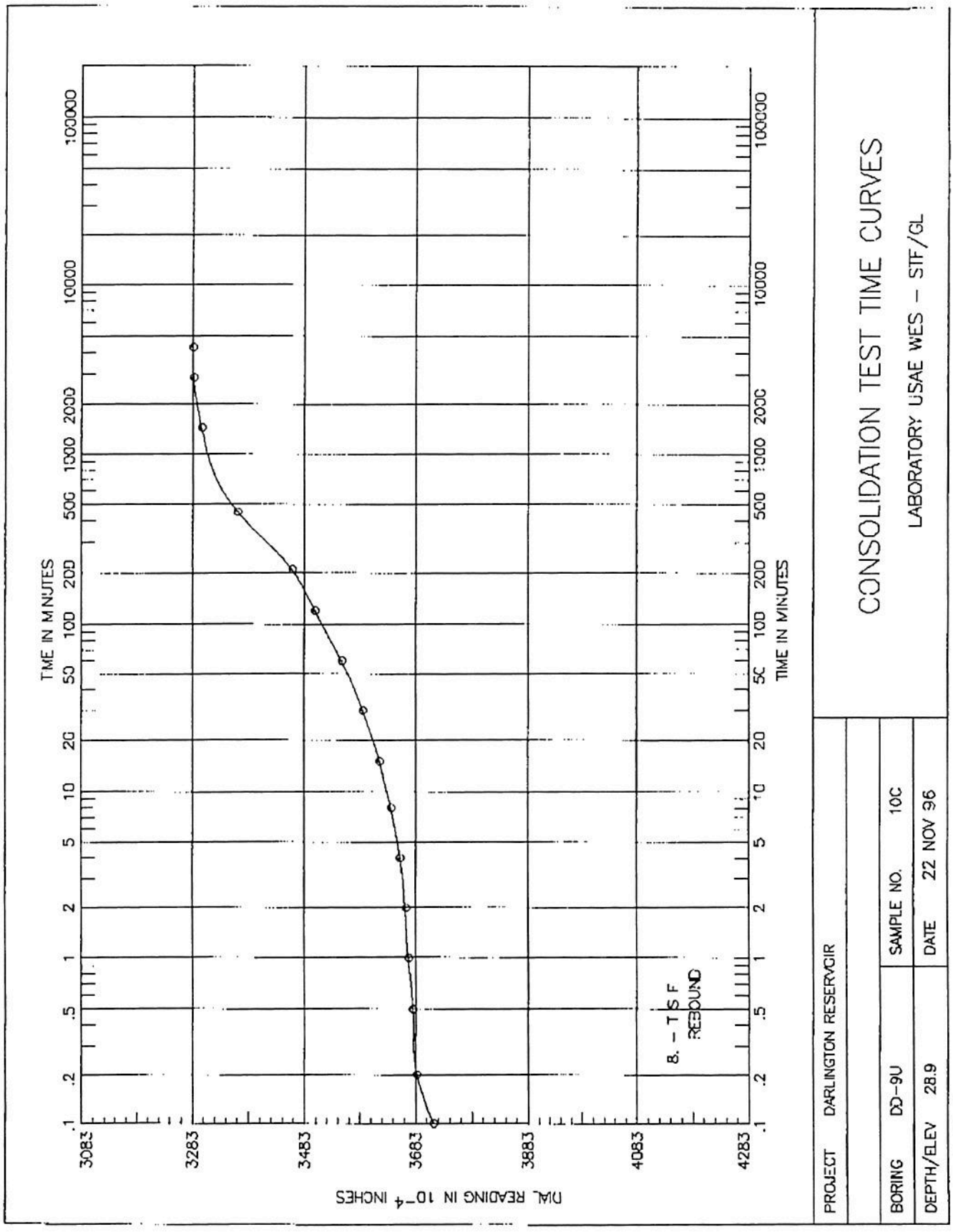


OJECT DARLINGTON RESERVOIR	
RING DD-9U	SAMPLE NO. 10C
PTH/ELEV 28.9	DATE 22 NOV 96

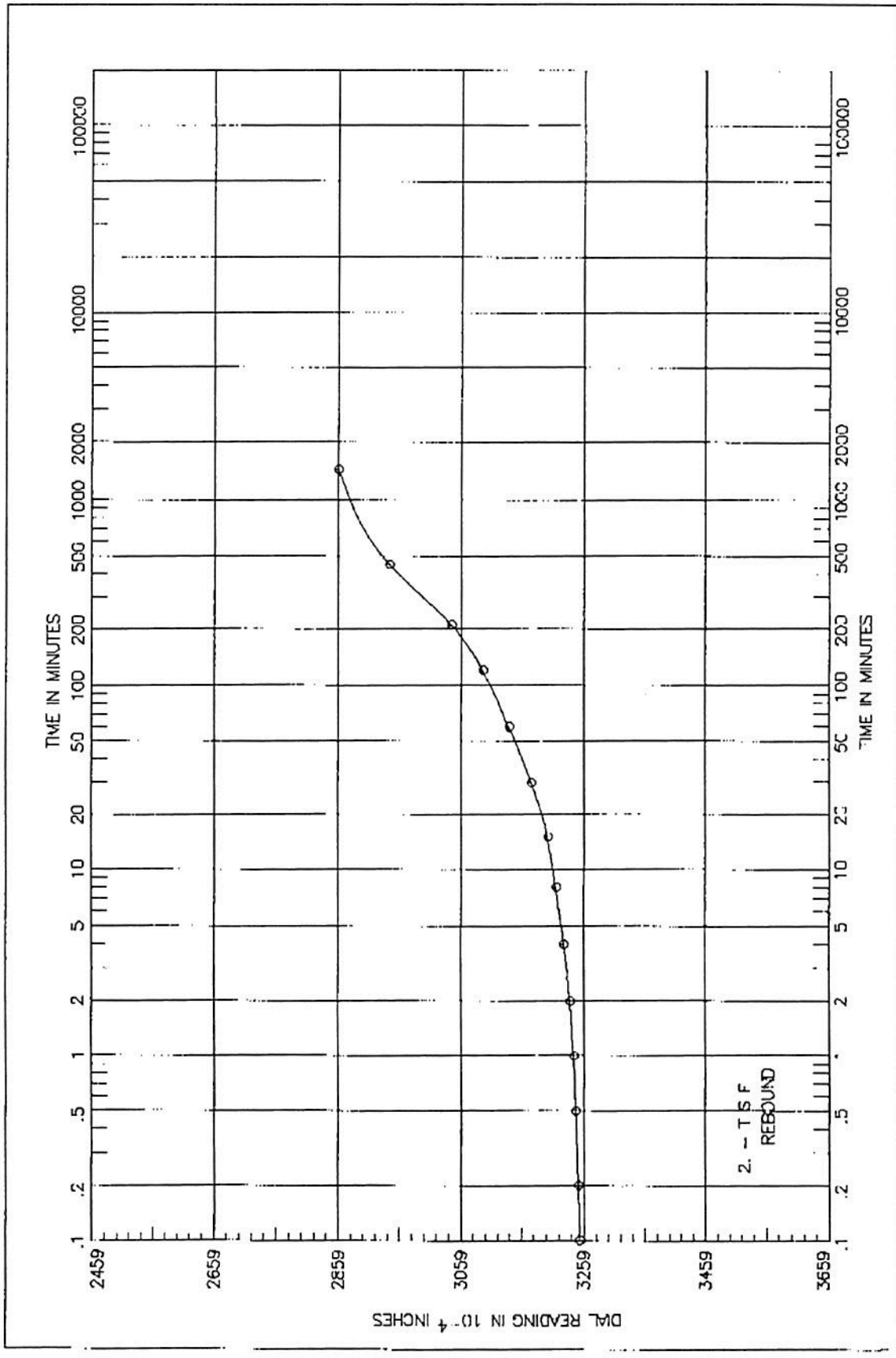
EET 8 OF 11

CONSOLIDATION TEST TIME CURVES

LABORATORY USAE WES – STF/GL


DIAL READING IN 10^{-4} INCHES
3083
3283
3483
3683
3883
4083
4283
TIME IN MINUTES
.1 .2 .5 1 2 5 10 20 50 100 200 500 1000 2000 10000 100000
8. - T S F
REBOUND
CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
PROJECT DARLINGTON RESERVOIR
BORING DD-9U
SAMPLE NO. 10C
DEPTH/ELEV 28.9
DATE 22 NOV 96

SHEET 9 OF 11

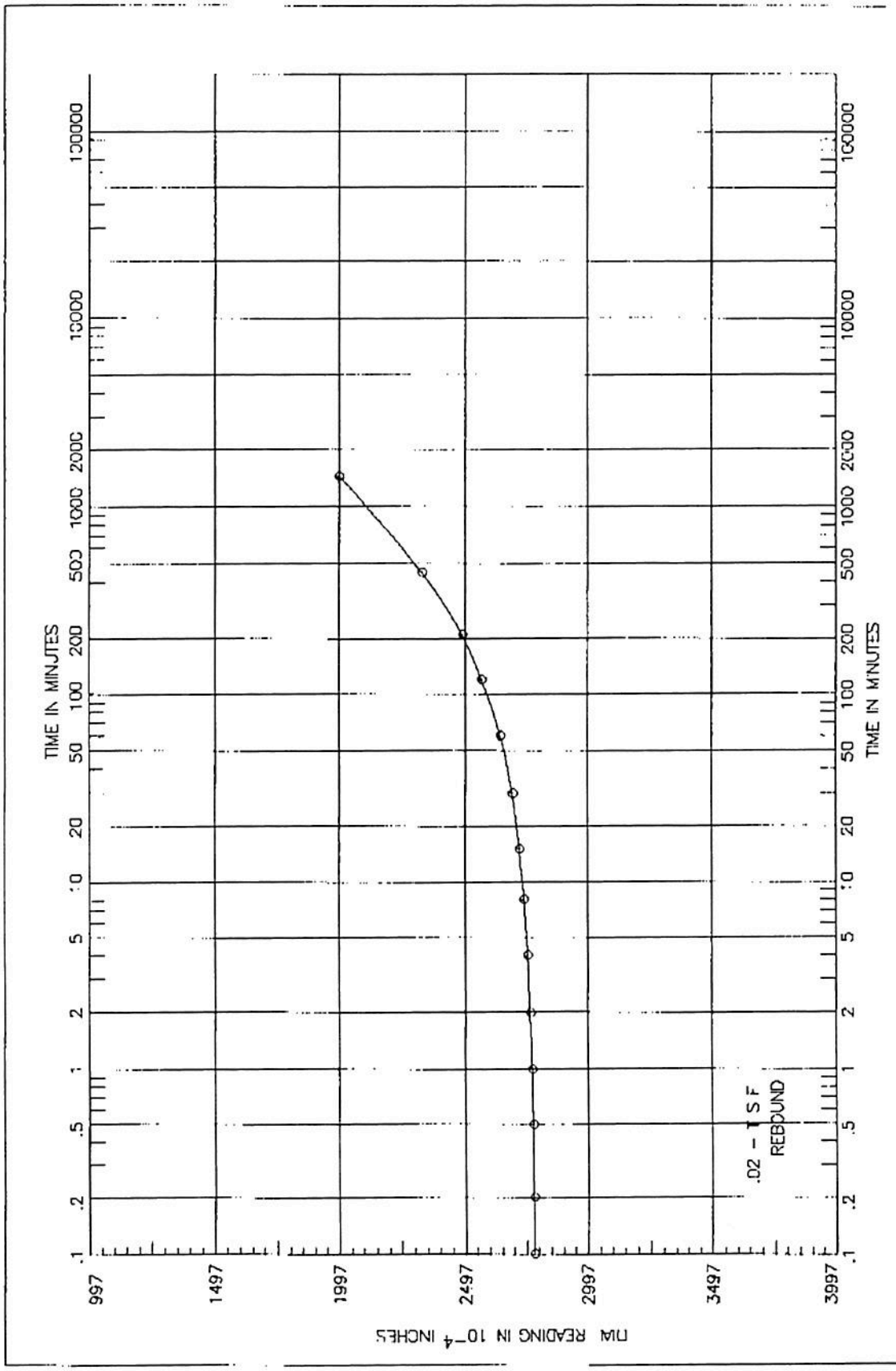


CONSOLIDATION TEST TIME CURVES

LABORATORY USAE WES - STF/GL

PROJECT: DARLINGTON RESERVOIR	
BORING DD-9U	SAMPLE NO. 10C
DEPTH/ELEV 28.9	DATE 22 NOV 96

SHEET 10 OF 11

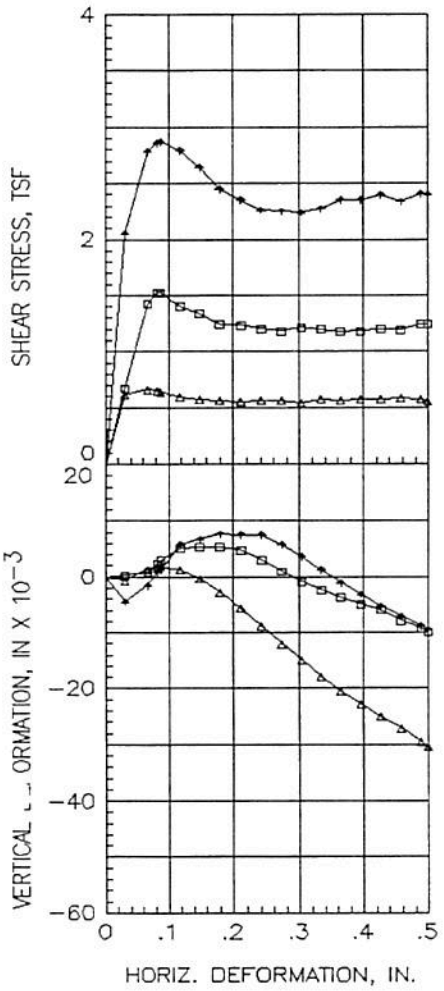


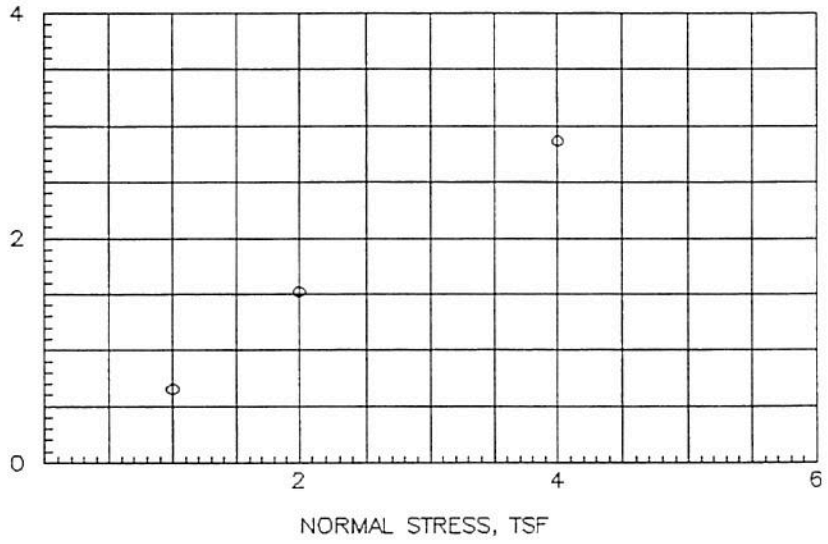
CONSOLIDATION TEST TIME CURVES

LABORATORY USAE WES - STF/GL

PROJECT	DARLINGTON RESERVOIR		
BORING	DD-9U	SAMPLE NO.	10C
DEPTH/ELEV	28.9	DATE	22 NOV 96

SHEET 11 OF 11





ϕ = _____ DEG

TAN ϕ = _____

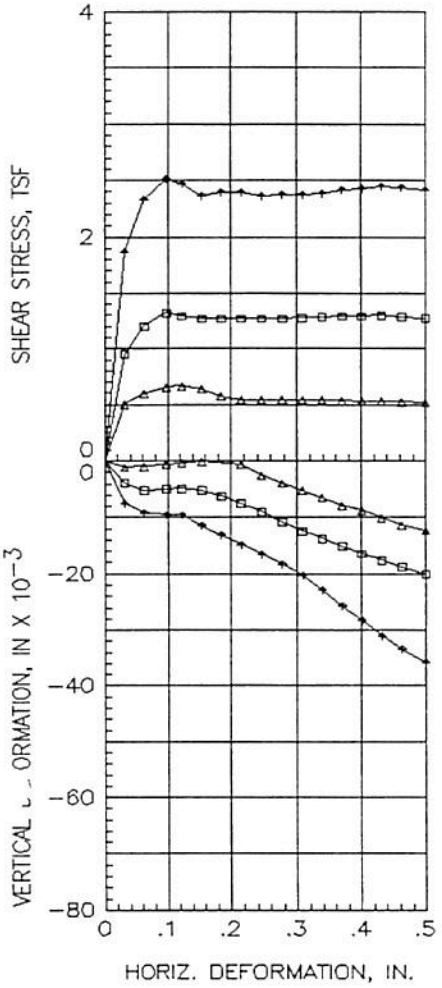
C = _____ T/SF

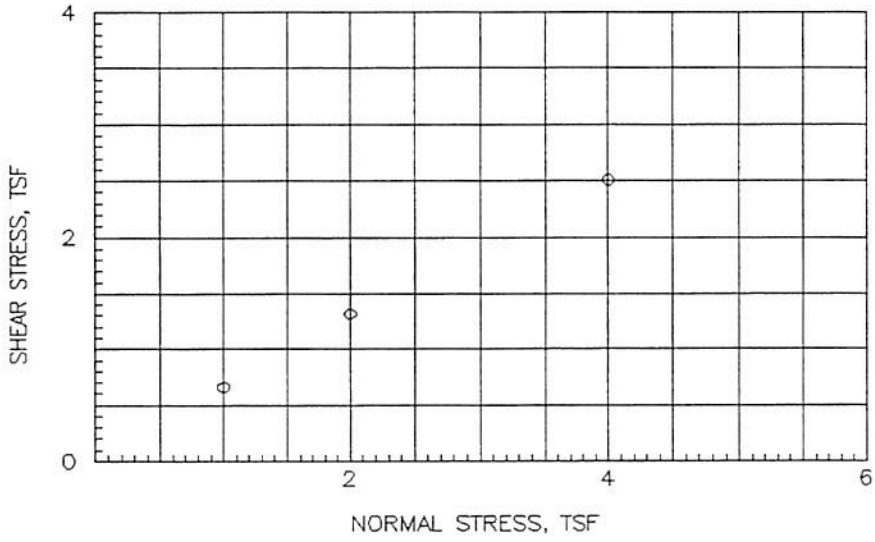
TEST NO.		1 △	2 □	3 +	
INITAL	WATER CONTENT, %	.1	.1	.1	
	VOID RATIO	.433	.434	.432	
	SATURATION, %	.9	.9	.9	
	DRY DENSITY, PCF	115.4	115.3	115.5	
VOID RATIO AFTER CONSOL					
FIFTY PERCENT CONSOL, MIN		< 1	< 1	< 1	
FINAL	WATER CONTENT, %	25.8	25.7	23.6	
	VOID RATIO				
	SATURATION, %				
NORMAL STRESS, TSF		1.00	2.00	4.00	
MAXIMUM SHEAR STRESS, TSF		.66	1.53	2.87	
TIME TO FAILURE, MIN		2	3	3	
RATE OF STRAIN, IN/MIN		.02999	.02999	.02999	
ULTIMATE SHEAR STRESS, TSF		.54	1.18	2.24	

TYPE SPECIMEN UNDISTURBED	3.00 IN. SQUARE	.500 IN. THICK
CLASSIFICATION SAND (SP), BROWN		

LL	PL	PI		GS 2.65 (EST)

REMARKS:	PROJECT DARLINGTON RESERVOIR	
	BORING NO. DD-9U	SAMPLE 3B
	DEPTH/ELEV 8.7	TECH. US
	LABORATORY USAE WES - STF/GL	DATE 18 DEC 96
	DIRECT SHEAR TEST REPORT	





ϕ = _____ DEG

TAN ϕ = _____

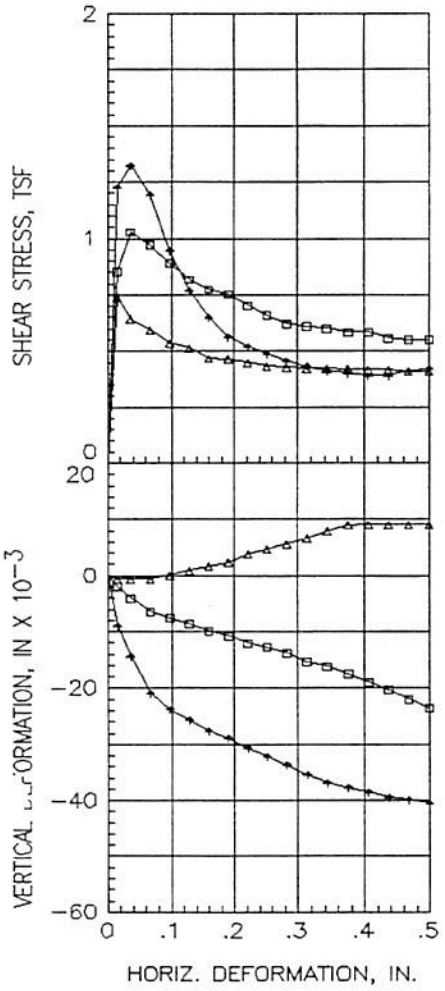
C = _____ T/SF

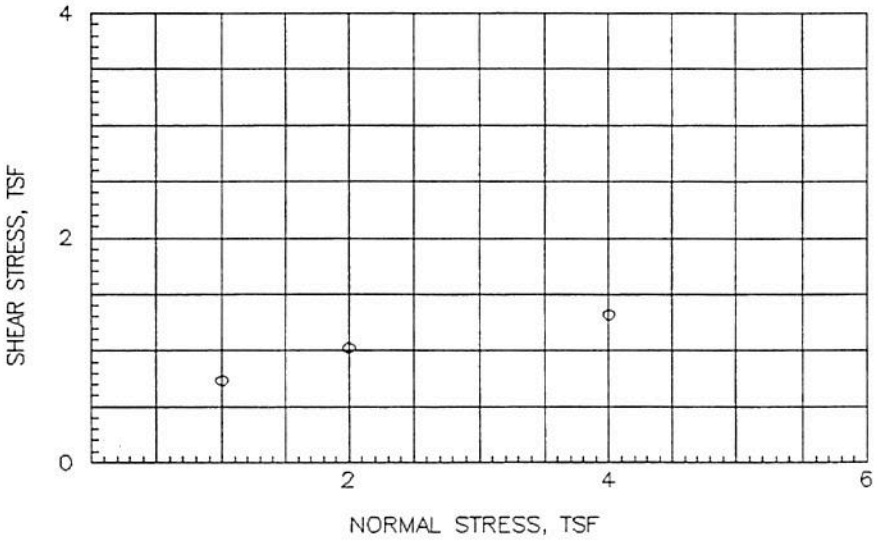
TEST NO.		1 △	2 □	3 +	
INITIAL	WATER CONTENT, %	.1	.1	.1	
	VOID RATIO	.517	.517	.517	
	SATURATION, %	.8	.7	.8	
	DRY DENSITY, PCF	109.8	109.8	109.8	
VOID RATIO AFTER CONSOL					
FIFTY PERCENT CONSOL, MIN		< 1	< 1	< 1	
FINAL	WATER CONTENT, %	23.2	22.1	22.3	
	VOID RATIO				
	SATURATION, %				
NORMAL STRESS, TSF		1.00	2.00	4.00	
MAXIMUM SHEAR STRESS, TSF		.67	1.32	2.52	
TIME TO FAILURE, MIN		4	3	3	
RATE OF STRAIN, IN/MIN		.02997	.02997	.02997	
ULTIMATE SHEAR STRESS, TSF		.52	1.27	2.36	

TYPE SPECIMEN UNDISTURBED	3.00 IN. SQUARE	.500 IN. THICK
CLASSIFICATION SILTY SAND (SP−SM), LIGHT BROWN		

LL	PL	PI		GS 2.67 (EST)

REMARKS;	PROJECT DARLINGTON RESERVOIR	
	BORING NO. DD−9U	SAMPLE 3C
	DEPTH/ELEV 8.7	TECH. US
	LABORATORY USAE WES − STF/GL	DATE 19 DEC 96
	DIRECT SHEAR TEST REPORT	





ϕ = _____ DEG

TAN ϕ = _____

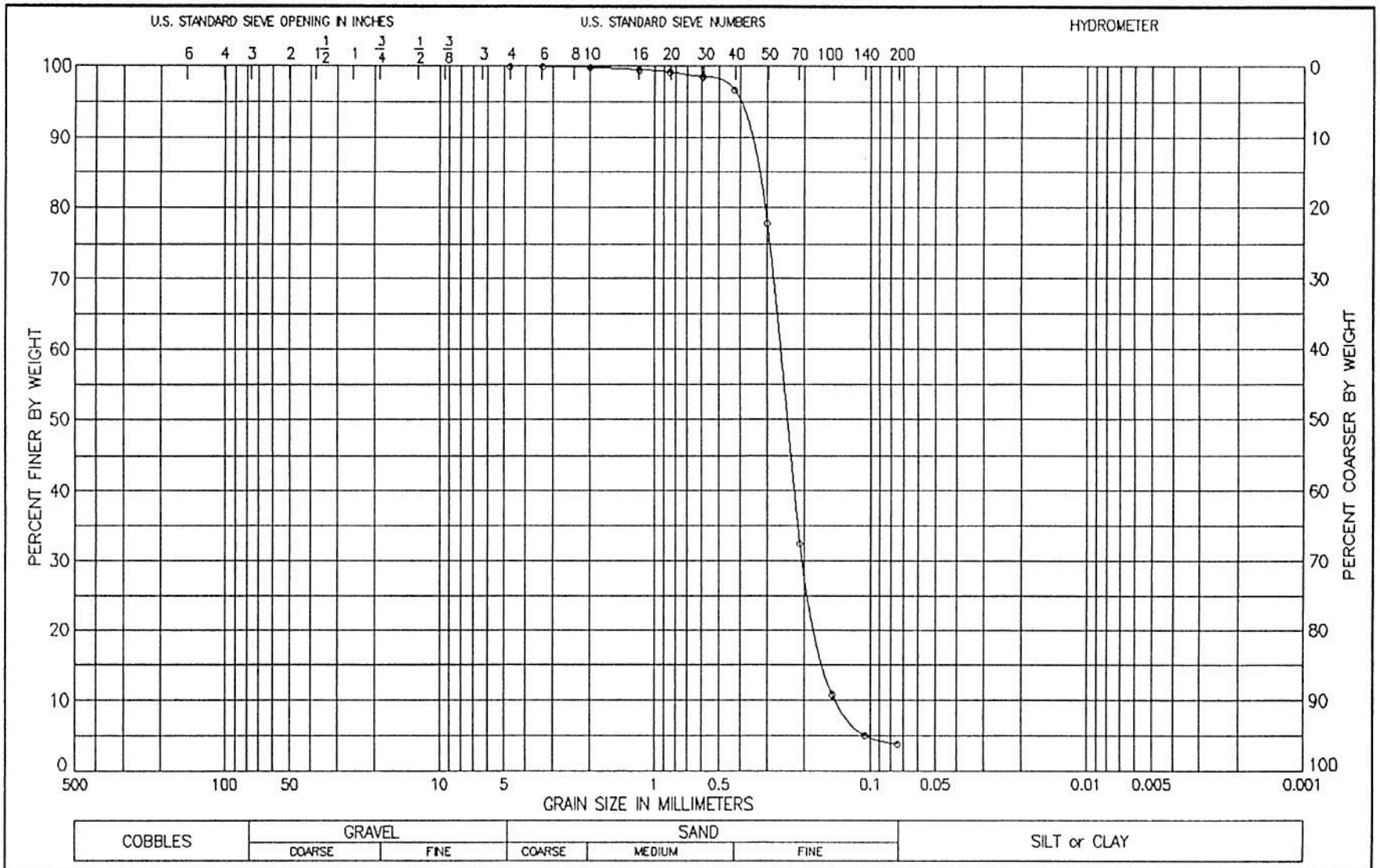
C = _____ T/SF

TEST NO.		1 △	2 □	3 +	
INITAL	WATER CONTENT, %	42.1	42.3	44.1	
	VOID RATIO	1.213	1.233	1.262	
	SATURATION, %	93.6	92.5	94.3	
	DRY DENSITY, PCF	76.1	75.5	74.5	
VOID RATIO AFTER CONSOL					
FIFTY PERCENT CONSOL, MIN		< 1	< 1	< 1	
FINAL	WATER CONTENT, %	45.1	44.5	42.9	
	VOID RATIO				
	SATURATION, %				
NORMAL STRESS, TSF		1.00	2.00	4.00	
MAXIMUM SHEAR STRESS, TSF		.74	1.03	1.33	
TIME TO FAILURE, MIN		94	221	221	
RATE OF STRAIN, IN/MIN		.00016	.00016	.00016	
ULTIMATE SHEAR STRESS, TSF		.41	.55	.39	

TYPE SPECIMEN UNDISTURBED	3.00 IN. SQUARE	.628 IN. THICK
CLASSIFICATION CLAY (CH), LIGHT BROWN		

LL 64	PL 23	PI 41		GS 2.70 (EST)

REMARKS;	PROJECT DARLINGTON RESERVOIR	
	BORING NO. DD-9U	SAMPLE 11D2
	DEPTH/ELEV 34.3	TECH. US
	LABORATORY USAE WES - STF/GL	DATE 10 DEC 96
	DIRECT SHEAR TEST REPORT	



LL	PL	PI	GS	NAT W,%	ORG,%

CLASSIFICATION
SAND (SP), BROWN

PROJECT DARLINGTON RESERVOIR

BORING NO. DD-9U SAMPLE NO. 3B

DEPTH/ELEV 8.7 DATE 29 NOV 96

GRADATION CURVE | LABORATORY USAE WES – STF/GL

SIEVE ANALYSIS

PROJECT: DARLINGTON RESERVOIR

BORING: DD-9U SAMPLE: 3B DF: PR0397 .DAT
DEPTH: 8.7 DATE: 29 NOV 96

NO-LIMITS-RAN GS: .00 WC: .00
CLASSIFICATION: 108
SAND (SP), BROWN

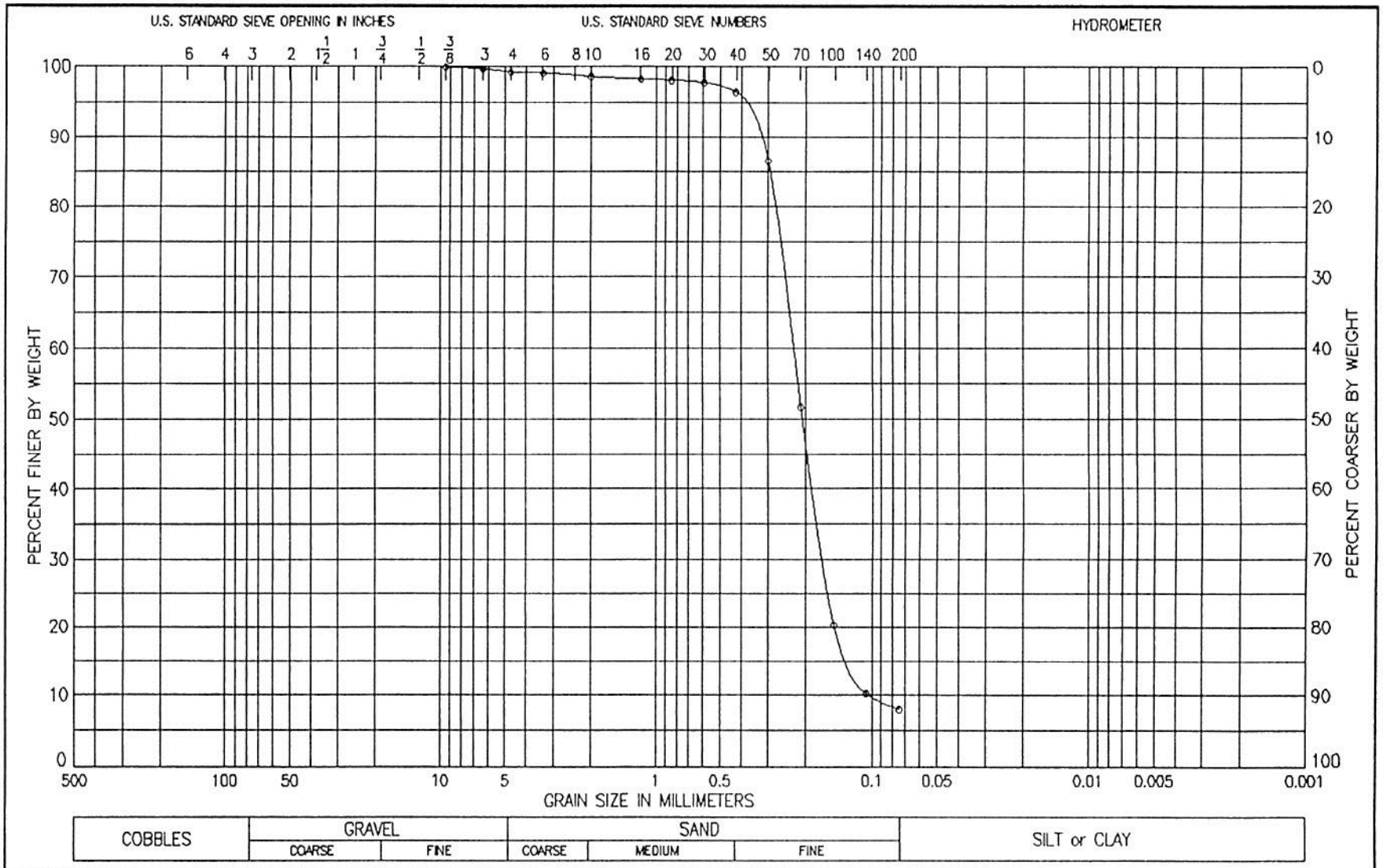
TOTAL WEIGHT OF SAMPLE: .0 gms.
PARTIAL WEIGHT AFTER SPLIT: 64.6 gms.

WEIGHTS gm.	SIEVE SIZE or NUMBER	OPENING mm	PERCENT FINER	PERCENT COARSER
.0	No 4	4.750	100.0	.0
.0	No 6	3.350	100.0	.0
.1	No 10	2.000	99.8	.2
.3	No 16	1.180	99.5	.5
.5	No 20	.850	99.2	.8
.9	No 30	.600	98.6	1.4
2.1	No 40	.425	96.7	3.3
14.3	No 50	.300	77.9	22.1
43.6	No 70	.212	32.5	67.5
57.6	No 100	.150	10.8	89.2
61.3	No 140	.106	5.1	94.9
62.1	No 200	.075	3.9	96.1

PERCENT GRAVEL = .0
PERCENT SAND = 96.1
PERCENT FINES = 3.9

D60 = .27
D30 = .20
D10 = .14
CU = 1.85
CC = 1.10

EDE



LL	PL	PI	GS	NAT W,%	ORG,%

CLASSIFICATION
SILTY SAND (SP-SM), LIGHT BROWN VISUAL

GRADATION CURVE | LABORATORY USAE WES -- STF/GL

PROJECT DARLINGTON RESERVOIR

BORING NO. DD-9U SAMPLE NO. 3C

DEPTH/ELEV 8.7 DATE 29 NOV 96

SIEVE ANALYSIS

PROJECT: DARLINGTON RESERVOIR

BORING: DD-9U SAMPLE: 3C DF: PR0397 .DAT
DEPTH: 8.7 DATE: 29 NOV 96

NO-LIMITS-RAN GS: .00 WC: .00
CLASSIFICATION: 118
SILTY SAND (SP-SM), LIGHT BROWN VISUAL

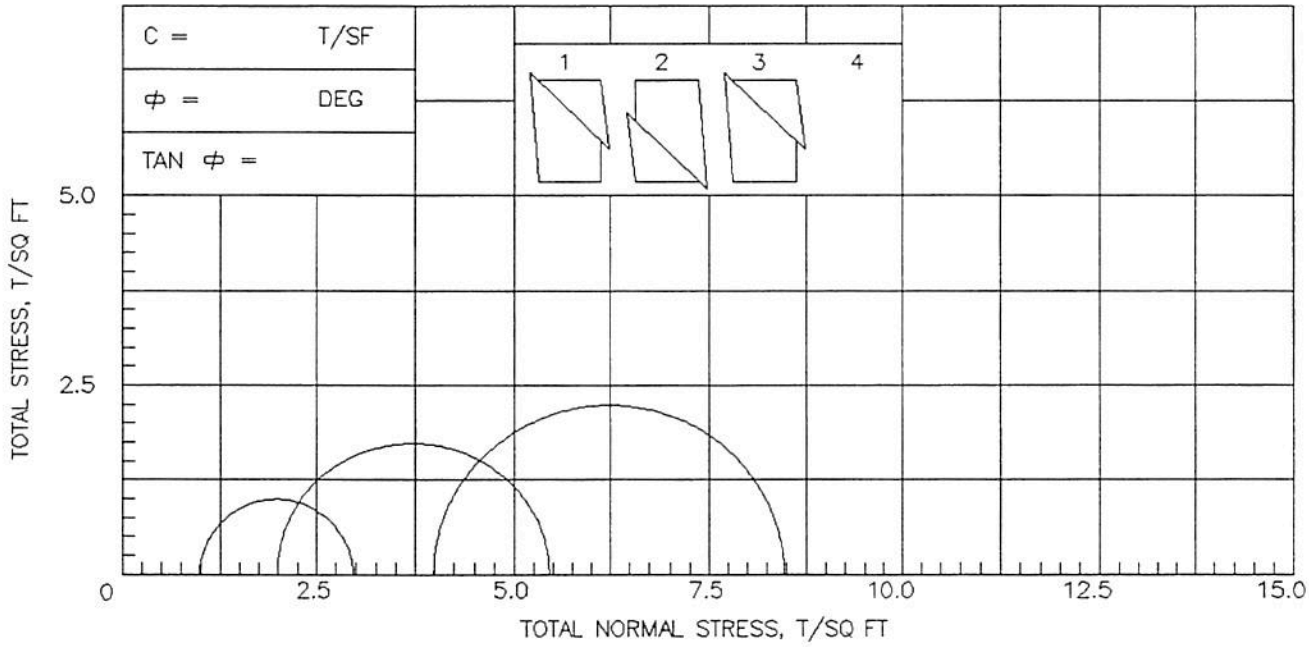
TOTAL WEIGHT OF SAMPLE: 578.0 gms.
PARTIAL WEIGHT AFTER SPLIT: 60.7 gms.

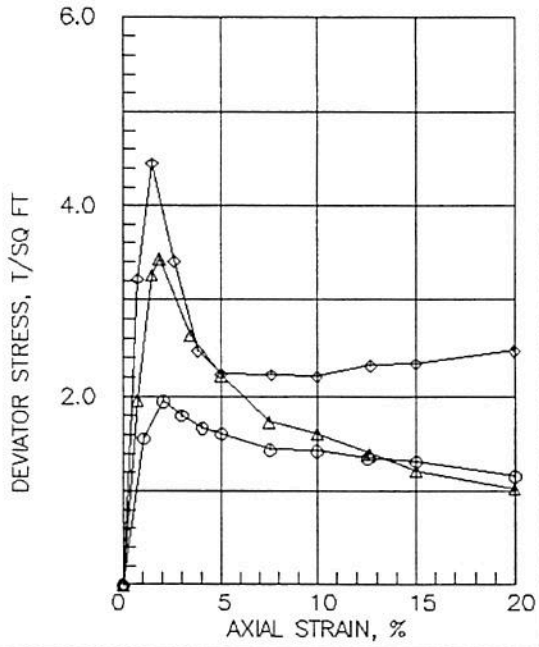
WEIGHTS gm.	SIEVE SIZE or NUMBER	OPENING mm	PERCENT FINER	PERCENT COARSER
.0	3/8 in	9.500	100.0	.0
1.7	No 3	6.350	99.7	.3
2.1	No 4	4.750	99.3	.7
.1	No 6	3.350	99.2	.8
.4	No 10	2.000	98.7	1.3
.6	No 16	1.180	98.4	1.6
.7	No 20	.850	98.2	1.8
.9	No 30	.600	97.9	2.1
1.7	No 40	.425	96.6	3.4
7.8	No 50	.300	86.6	13.4
29.1	No 70	.212	51.7	48.3
48.3	No 100	.150	20.3	79.7
54.4	No 140	.106	10.3	89.7
55.8	No 200	.075	8.0	92.0

PERCENT GRAVEL = .7
PERCENT SAND = 91.3
PERCENT FINES = 8.0

D60 = .23
D30 = .17
D10 = .10
CU = 2.29
CC = 1.21

EDE





SPECIMEN NO.		⊙ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	42.1	42.7	42.1	
	DRY DENSITY, PCF	78.5	78.7	79.6	
	SATURATION, %	99.1	100+	100+	
	VOID RATIO	1.148	1.141	1.117	
BEFORE SHEAR	WATER CONTENT, %	44.0	44.0	43.3	
	DRY DENSITY, PCF	79.7	80.2	82.0	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	1.116	1.101	1.057	
	BACK PRESS., TSF	5.04	5.04	5.04	
MIN. PRIN. STRESS, TSF		1.00	2.00	4.00	
MAX. DEV. STRESS, TSF		1.96	3.44	4.45	
TIME TO FAILURE, MIN.		131	120	95	
RATE OF STRAIN, %/MIN		.016	.016	.016	
INITIAL DIAMETER, IN.		1.36	1.36	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED–STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), LIGHT BROWN

LL 63	PL 22	PI 41	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	$\bar{R}$ TEST

REMARKS:

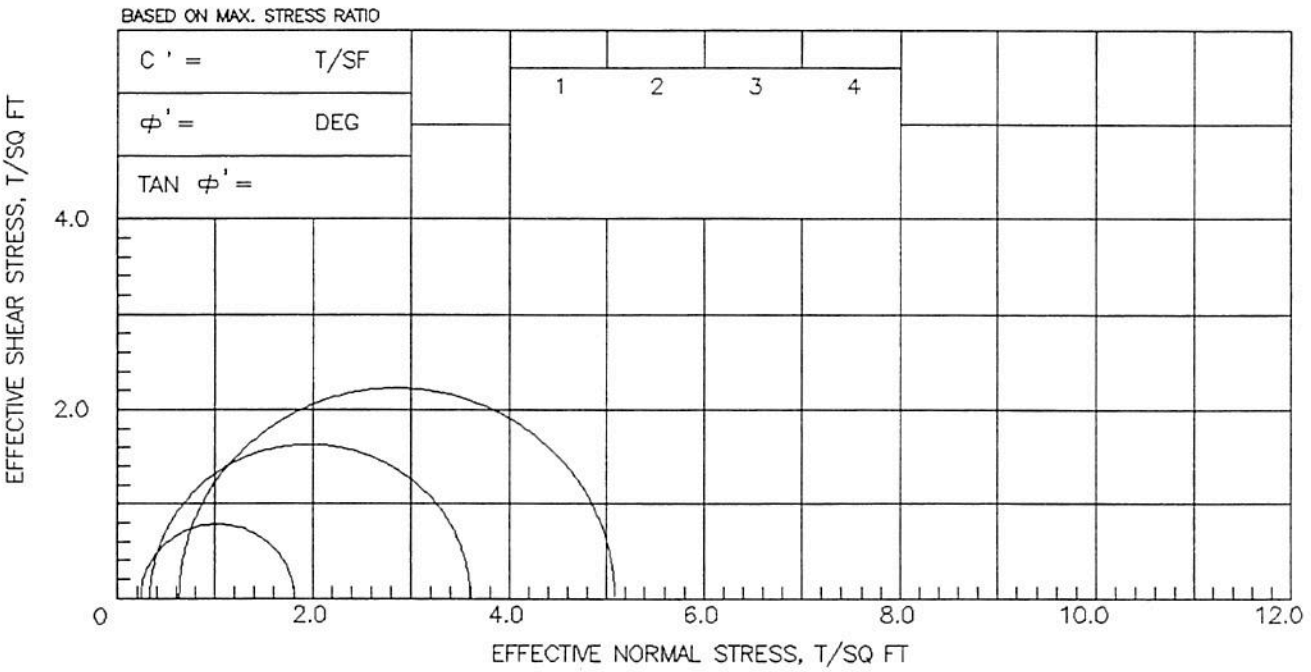
PROJECT DARLINGTON RESERVOIR

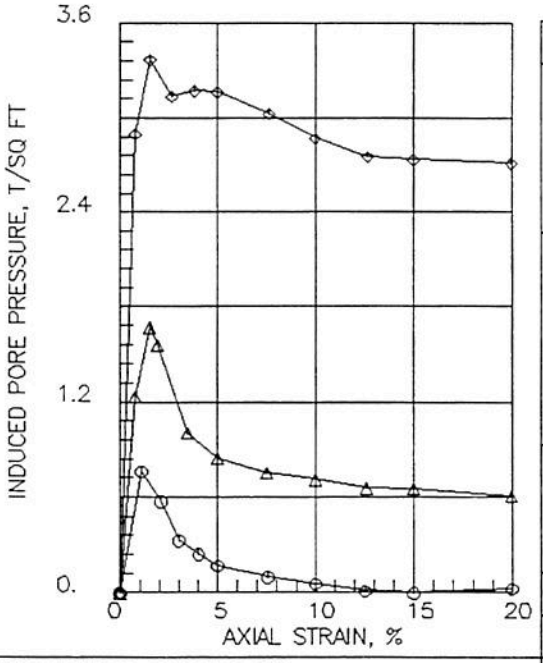
BORING NO. DD–9U SAMPLE NO. 11D1

DEPTH/ELEV 33.9 TECH. JL

LABORATORY USAE WES – STF/GL DATE 14 DEC 96

$\bar{R}$–TRIAXIAL COMPRESSION TEST REPORT





SPECIMEN NO.		⊙ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	42.1	42.7	42.1	
	DRY DENSITY, PCF	78.5	78.7	79.6	
	SATURATION, %	99.1	100+	100+	
	VOID RATIO	1.148	1.141	1.117	
BEFORE SHEAR	WATER CONTENT, %	44.0	44.0	43.3	
	DRY DENSITY, PCF	79.7	80.2	82.0	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	1.116	1.101	1.057	
	BACK PRESS., TSF	5.04	5.04	5.04	
@ r_1'/r_3' MAX	MIN PRIN EFF STRESS, TSF	.24	.33	.63	
	DEV. STRESS, TSF	1.57	3.27	4.45	
	TIME TO FAILURE, MIN.	67	94	95	
RATE OF STRAIN, %/MIN		.016	.016	.016	
INITIAL DIAMETER, IN.		1.36	1.36	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED–STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), LIGHT BROWN

LL 63	PL 22	PI 41	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	$\bar{R}$ TEST

REMARKS:

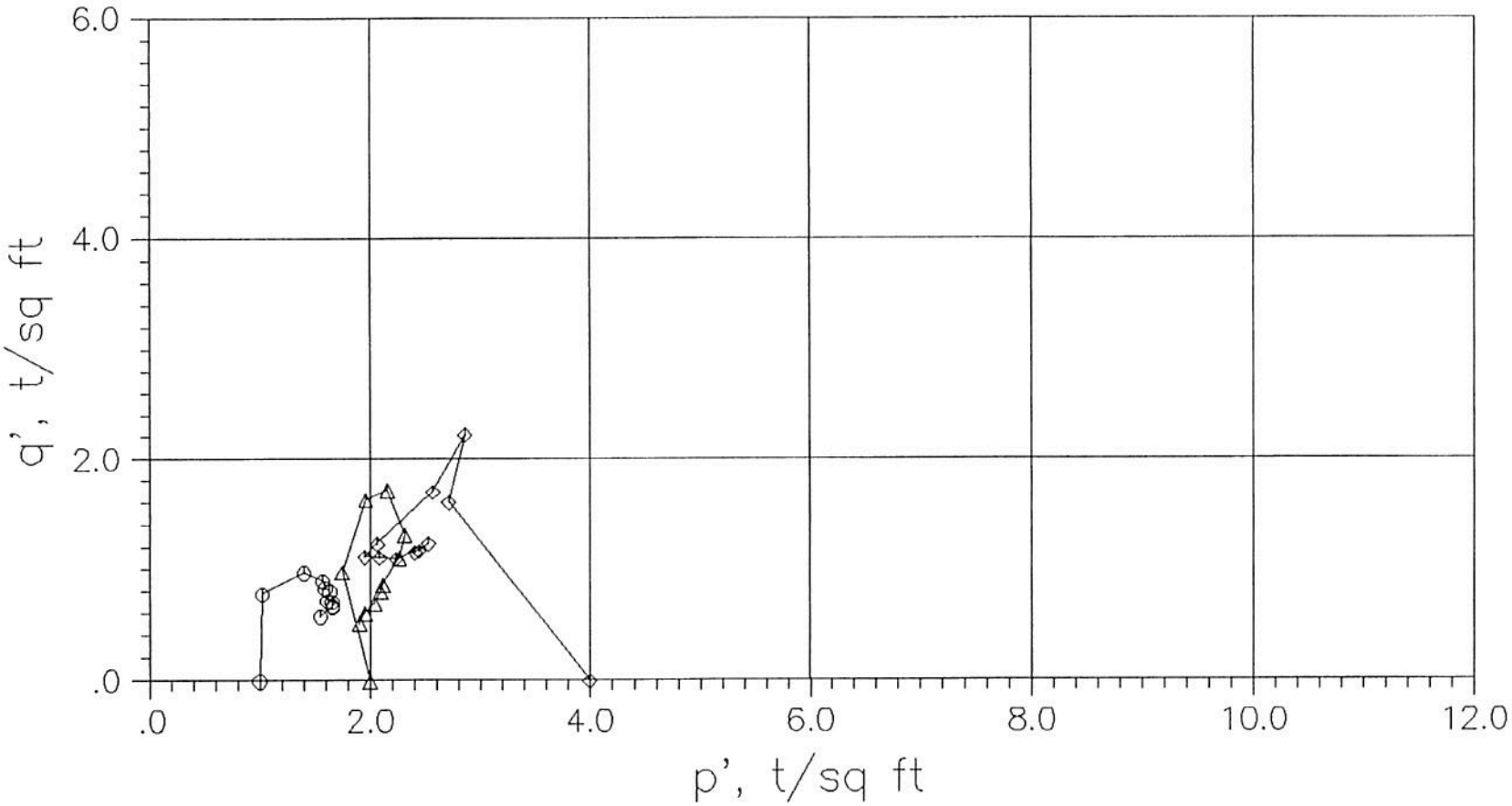
PROJECT DARLINGTON RESERVOIR

BORING NO. DD–9U SAMPLE NO. 11D1

DEPTH/ELEV 33.9 TECH. JL

LABORATORY USAE WES – STF/GL DATE 14 DEC 96

$\bar{R}$–TRIAXIAL COMPRESSION TEST REPORT



PROJECT DARLINGTON RESERVOIR	
BORING DD-9U	SAMPLE NO. 11D1
DEPTH/ELEV 33.9	DATE 14 DEC 96

Stress Paths

LABORATORY USAE WES – STF/GL

R-BAR TRIAXIAL COMPRESSION TEST REPORT

TLE: DARLINGTON RESERVOIR

BOR: DD-9U SAM: 11D1 DEP: 33.9 DAT: 14 DEC 96 TEC: JL
DES: CLAY (CH), LIGHT BROWN

SAMPLE TYPE = UNDISTURBED
GS: 2.70 (ESTIMATED)
LL: 63 PL: 22 PI: 41

WEIGHT OF SOLIDS = 90.04 I. HEIGHT = 3.000" I. DIAMETER = 1.362"

HEIGHT CHANGES: PRE-CONS. = .00300" DURING CONS. = .01700"

VOLUME CHANGE DURING CONS. = .9 CC TIME OF TEST = 1258 MINS.

BACK PRESSURE = 70.00 PSI (5.04 TSF) CHAMBER PRESSURE = 83.89

EFF CONFINING PRESSURE, TSF = 1.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	42.14	44.05
VOID RATIO	1.148	1.116
SATURATION, %	99.14	106.60
DRY DENSITY, PCF	78.49	79.67

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	EFFECTIVE E3 - TSF -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	70.0	.0	.00	.00	1.000	1.000	1.000	1.00	.00
.032	31.8	80.6	1.1	.76	1.57	1.804	.237	7.618	1.02	.78
.062	40.1	78.0	2.1	.58	1.96	2.380	.424	5.614	1.40	.98
.089	37.4	74.6	3.0	.33	1.81	2.477	.669	3.703	1.57	.90
.120	35.1	73.4	4.0	.24	1.68	2.434	.755	3.222	1.59	.84
.149	34.2	72.4	5.0	.17	1.62	2.446	.827	2.957	1.64	.81
.224	31.3	71.4	7.5	.10	1.44	2.342	.899	2.604	1.62	.72
.298	31.9	70.8	10.0	.06	1.43	2.373	.942	2.518	1.66	.72
.374	31.1	70.2	12.6	.01	1.36	2.341	.986	2.375	1.66	.68
.447	31.3	70.0	15.0	.00	1.33	2.326	1.000	2.326	1.66	.66
.596	29.2	70.4	20.0	.03	1.16	2.135	.971	2.198	1.55	.58

RATE OF STRAIN = .016

R-BAR TRIAXIAL COMPRESSION TEST REPORT

LE: DARLINGTON RESERVOIR

BOR: DD-9U SAM: 11D1 DEP: 33.9 DAT: 14 DEC 96 TEC: JL
DES: CLAY (CH), LIGHT BROWN

SAMPLE TYPE = UNDISTURBED
GS: 2.70 (ESTIMATED)
LL: 63 PL: 22 PI: 41

WEIGHT OF SOLIDS = 89.93 I. HEIGHT = 3.000" I. DIAMETER = 1.359"

HEIGHT CHANGES: PRE-CONS. = .00200" DURING CONS. = .03100"

VOLUME CHANGE DURING CONS. = 1.2 CC TIME OF TEST = 1258 MINS.

BACK PRESSURE = 70.00 PSI (5.04 TSF) CHAMBER PRESSURE = 97.78

EFF CONFINING PRESSURE, TSF = 2.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	42.68	44.01
VOID RATIO	1.141	1.101
SATURATION, %	101.01	107.97
DRY DENSITY, PCF	78.74	80.25

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	EFFECTIVE E3 - TSF -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	70.0	.0	.00	.00	2.000	2.000	1.000	2.00	.00
.022	39.7	87.2	.7	1.24	1.97	2.733	.762	3.588	1.75	.99
.044	66.3	93.2	1.5	1.67	3.27	3.597	.330	10.910	1.96	1.63
.056	70.0	91.7	1.9	1.56	3.44	3.874	.438	8.849	2.16	1.72
.102	54.6	84.0	3.4	1.01	2.64	3.630	.992	3.659	2.31	1.32
.148	46.8	81.8	5.0	.85	2.22	3.375	1.150	2.934	2.26	1.11
.222	37.6	80.5	7.5	.76	1.74	2.984	1.244	2.399	2.11	.87
.297	35.9	79.9	10.0	.71	1.62	2.903	1.287	2.256	2.10	.81
.374	32.1	79.2	12.6	.66	1.40	2.741	1.338	2.049	2.04	.70
.445	28.7	79.1	15.0	.66	1.22	2.565	1.345	1.908	1.96	.61
.593	25.8	78.5	20.0	.61	1.03	2.421	1.388	1.744	1.90	.52

RATE OF STRAIN = .016

R-BAR TRIAXIAL COMPRESSION TEST REPORT

TLE: DARLINGTON RESERVOIR

BOR: DD-9U SAM: 11D1 DEP: 33.9 DAT: 14 DEC 96 TEC: JL
DES: CLAY (CH), LIGHT BROWN

SAMPLE TYPE = UNDISTURBED
GS: 2.70 (ESTIMATED)
LL: 63 PL: 22 PI: 41

WEIGHT OF SOLIDS = 91.20 I. HEIGHT = 3.000" I. DIAMETER = 1.361"

HEIGHT CHANGES: PRE-CONS. = .00000" DURING CONS. = .04800"

VOLUME CHANGE DURING CONS. = 2.0 CC TIME OF TEST = 1258 MINS.

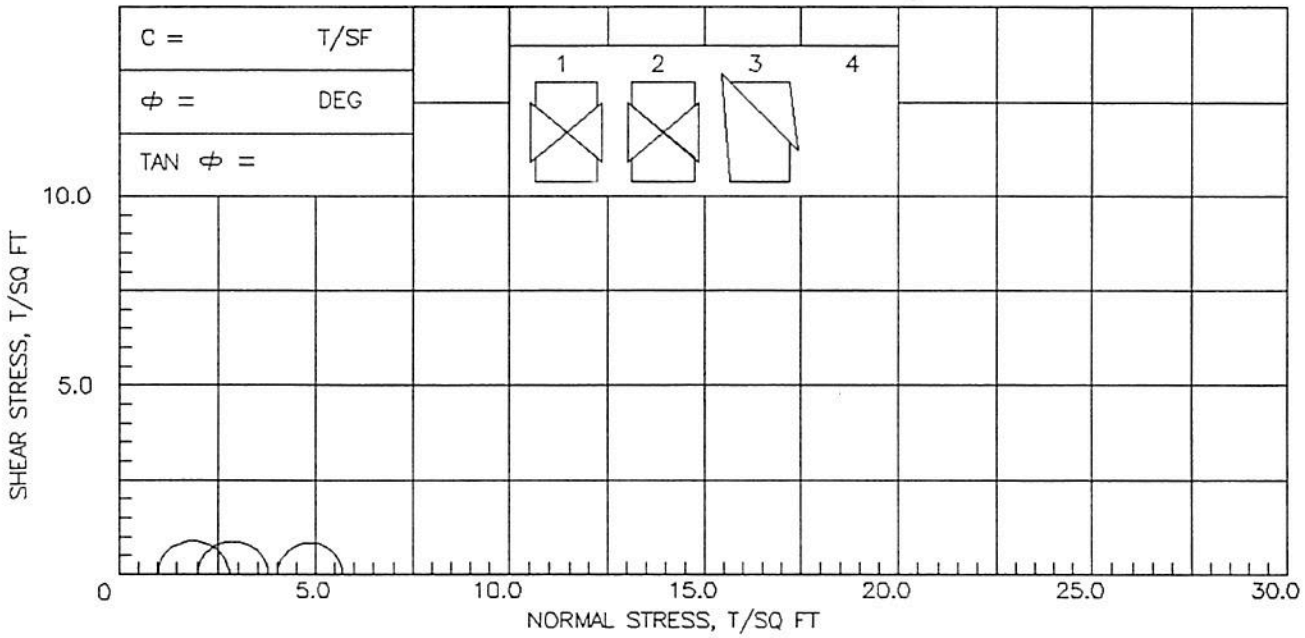
BACK PRESSURE = 70.00 PSI (5.04 TSF) CHAMBER PRESSURE = 125.56

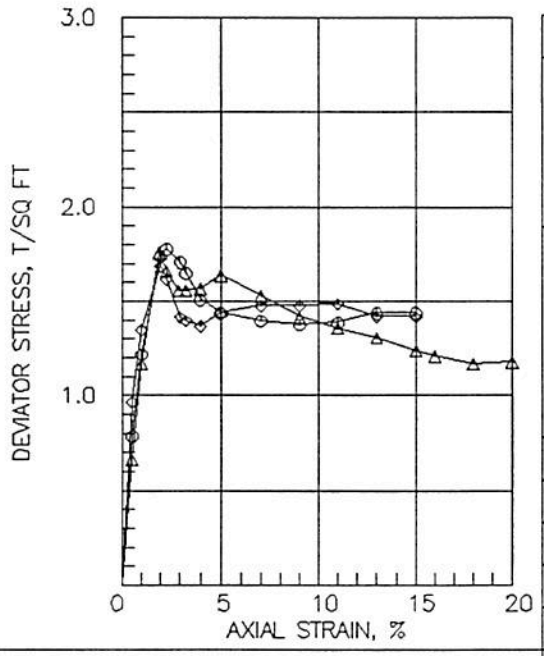
EFF CONFINING PRESSURE, TSF = 4.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	42.08	43.32
VOID RATIO	1.117	1.057
SATURATION, %	101.70	110.70
DRY DENSITY, PCF	79.61	81.96

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	EFFECTIVE E3 - TSF -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	70.0	.0	.00	.00	4.000	4.000	1.000	4.00	.00
.022	64.8	110.3	.7	2.90	3.22	4.323	1.099	3.935	2.71	1.61
.044	90.2	116.8	1.5	3.37	4.45	5.085	.631	8.063	2.86	2.23
.078	69.9	113.6	2.6	3.14	3.41	4.273	.861	4.962	2.57	1.71
.112	51.4	114.1	3.8	3.17	2.48	3.304	.825	4.005	2.06	1.24
.148	47.1	114.0	5.0	3.17	2.24	3.075	.832	3.695	1.95	1.12
.225	48.2	112.1	7.6	3.03	2.23	3.201	.969	3.303	2.09	1.12
.295	49.0	109.9	10.0	2.87	2.21	3.338	1.127	2.961	2.23	1.11
.374	53.1	108.3	12.7	2.76	2.32	3.567	1.243	2.871	2.40	1.16
.443	55.1	108.0	15.0	2.74	2.35	3.612	1.264	2.857	2.44	1.17
.590	61.9	107.7	20.0	2.71	2.48	3.769	1.286	2.931	2.53	1.24

RATE OF STRAIN = .016





SPECIMEN NO.		○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	38.2	38.4	38.3	
	DRY DENSITY, PCF	83.9	83.2	83.4	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	1.008	1.027	1.021	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		1.00	2.00	4.00	
MAX. DEV. STRESS, TSF		1.78	1.76	1.69	
TIME TO FAILURE, MIN.		4	23	24	
STRAIN RATE INCR., %			4	4	
INITIAL DIAMETER, IN.		1.35	1.36	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), LIGHT GRAY

LL 58	PL 17	PI 41	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:

RESIDUAL STRENGTH AT 25%

SPEC. 1 = 1.36 TSF

SPEC. 2 = 1.15 TSF

SPEC. 3 = 1.26 TSF

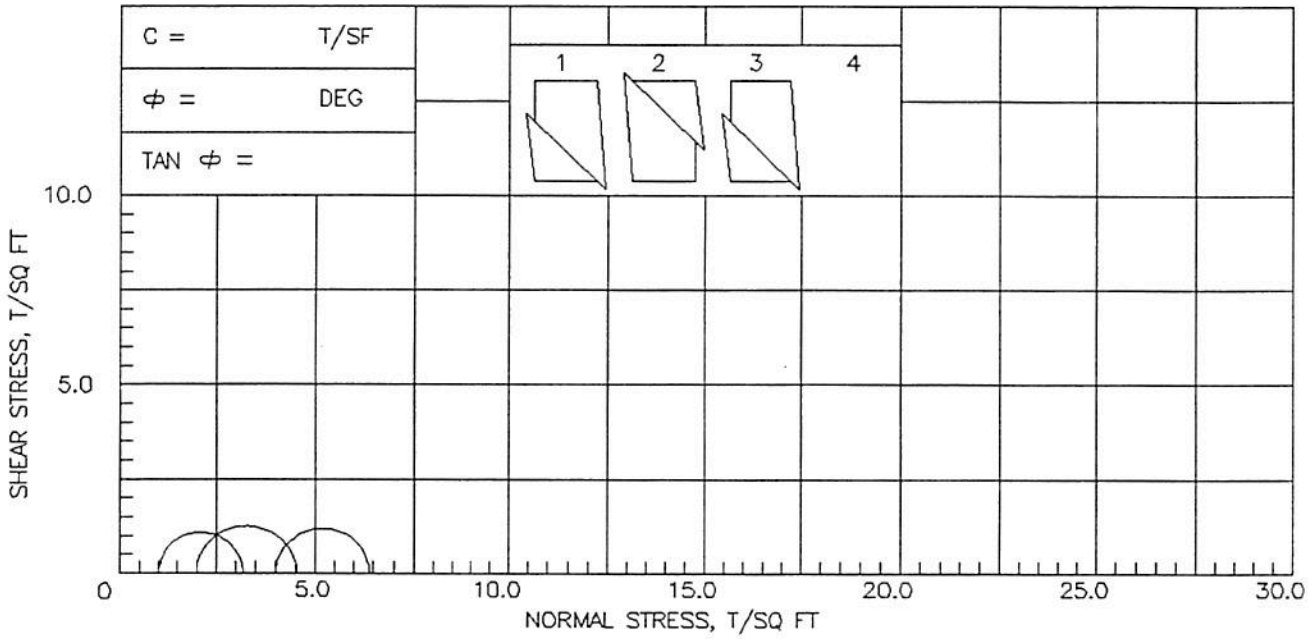
PROJECT DARLINGTON RESERVOIR

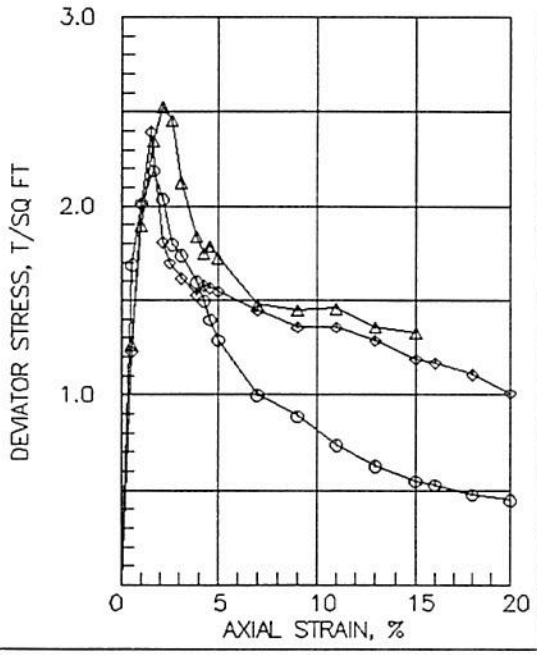
BORING NO. DD-10U | SAMPLE NO. 9C

DEPTH/ELEV 25.0 | TECH. JAL

LABORATORY USAE WES - STF/GL | DATE 27 NOV 96

TRIAXIAL COMPRESSION TEST REPORT





SPECIMEN NO.		⊙ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	45.2	45.0	44.6	
	DRY DENSITY, PCF	77.1	77.4	77.1	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	1.186	1.178	1.187	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		1.00	2.00	4.00	
MAX. DEV. STRESS, TSF		2.19	2.53	2.40	
TIME TO FAILURE, MIN.		3	27	18	
STRAIN RATE INCR., %			4	3	
INITIAL DIAMETER, IN.		1.33	1.33	1.34	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED–STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), LIGHT GRAY; IRON OXIDE PARTICLES

LL 69	PL 22	PI 47	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:

RESIDUAL STRENGTH AT 25%

SPEC. 1 = 0.39 TSF

SPEC. 2 = 1.27 TSF

SPEC. 3 = 0.80 TSF

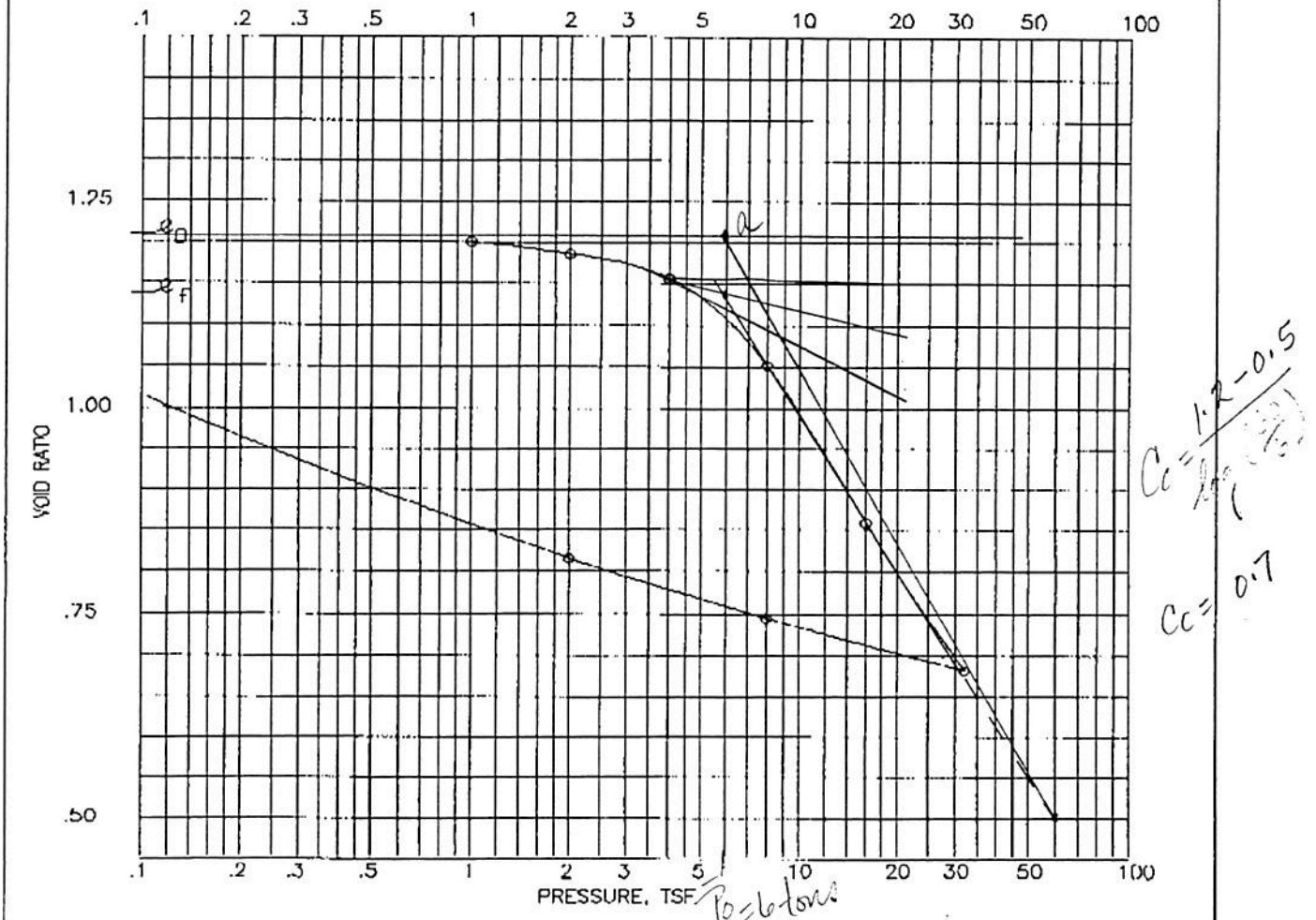
PROJECT DARLINGTON RESERVOIR

BORING NO. DD–10U SAMPLE NO. 10B

DEPTH/ELEV 28.3 TECH. GM

LABORATORY USAE WES – STF/GL DATE 2 DEC 96

TRIAXIAL COMPRESSION TEST REPORT



	BEFORE TEST	AFTER TEST
OVERBURDEN PRESSURE, TSF	WATER CONTENT, %	44.3	42.3
PRECONSOL. PRESSURE, TSF	DRY DENSITY, PCF	76.4	78.9
COMPRESSION INDEX	SATURATION, %	99.0	100 +
TYPE SPECIMEN: UNDISTURBED	VOID RATIO	1.207	1.136
DIA. IN 4.44 / HT. IN 1.121	BACK PRESSURE, TSF		

CLASSIFICATION: CLAY (CH), LIGHT BROWN

LL | PL | PI

GS 2.70 (EST) | D_{10}

PROJECT: DARLINGTON RESERVOIR

REMARKS:

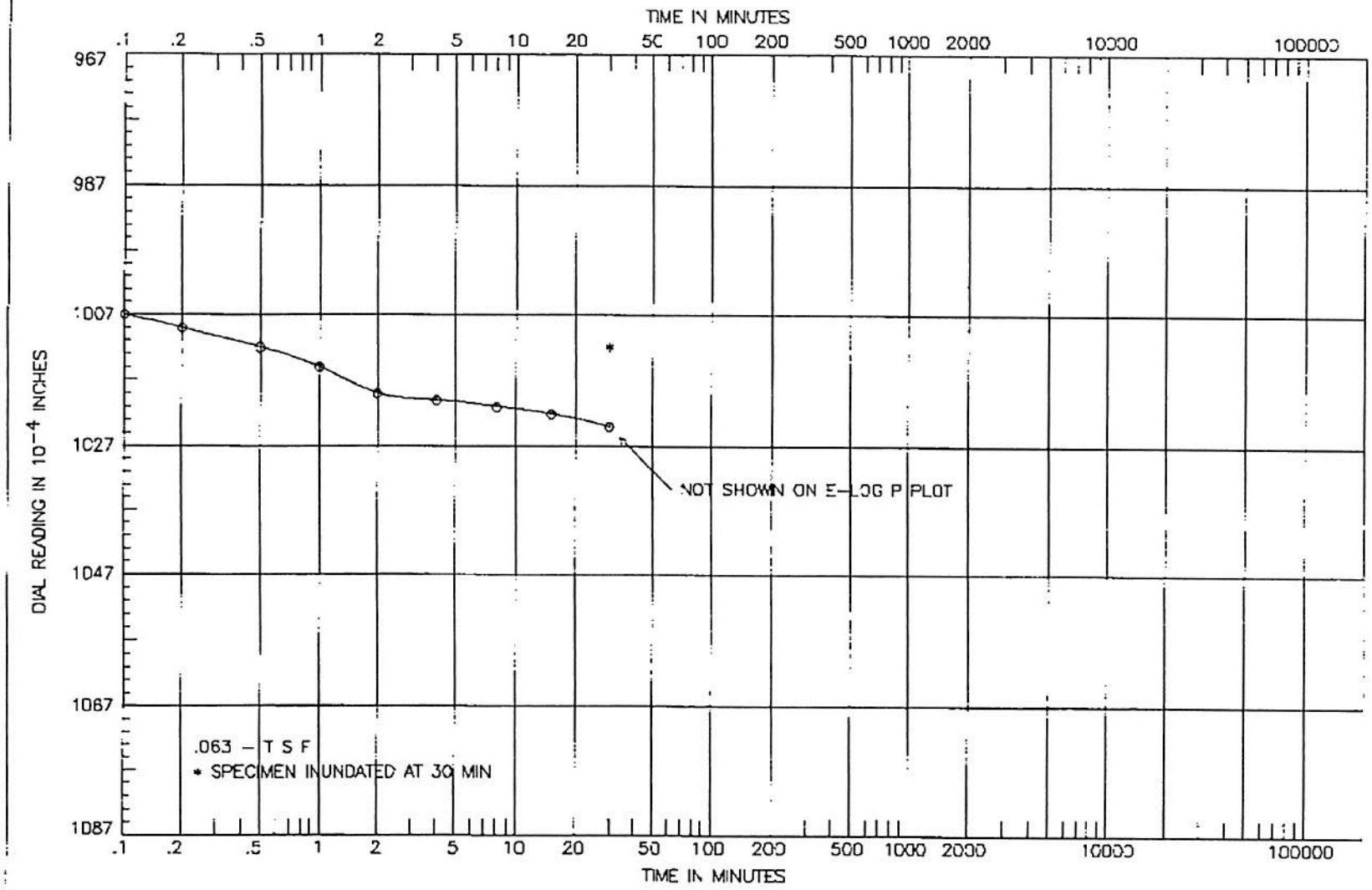
BORING NO. DD-10U | SAMPLE NO. 10C

DEPTH/ELEV 29.1 | TECH. JL

LABORATORY USAE WES – STF/GL | DATE 22 NOV 96

CONSOLIDATION TEST REPORT

SHEET 1 OF 11

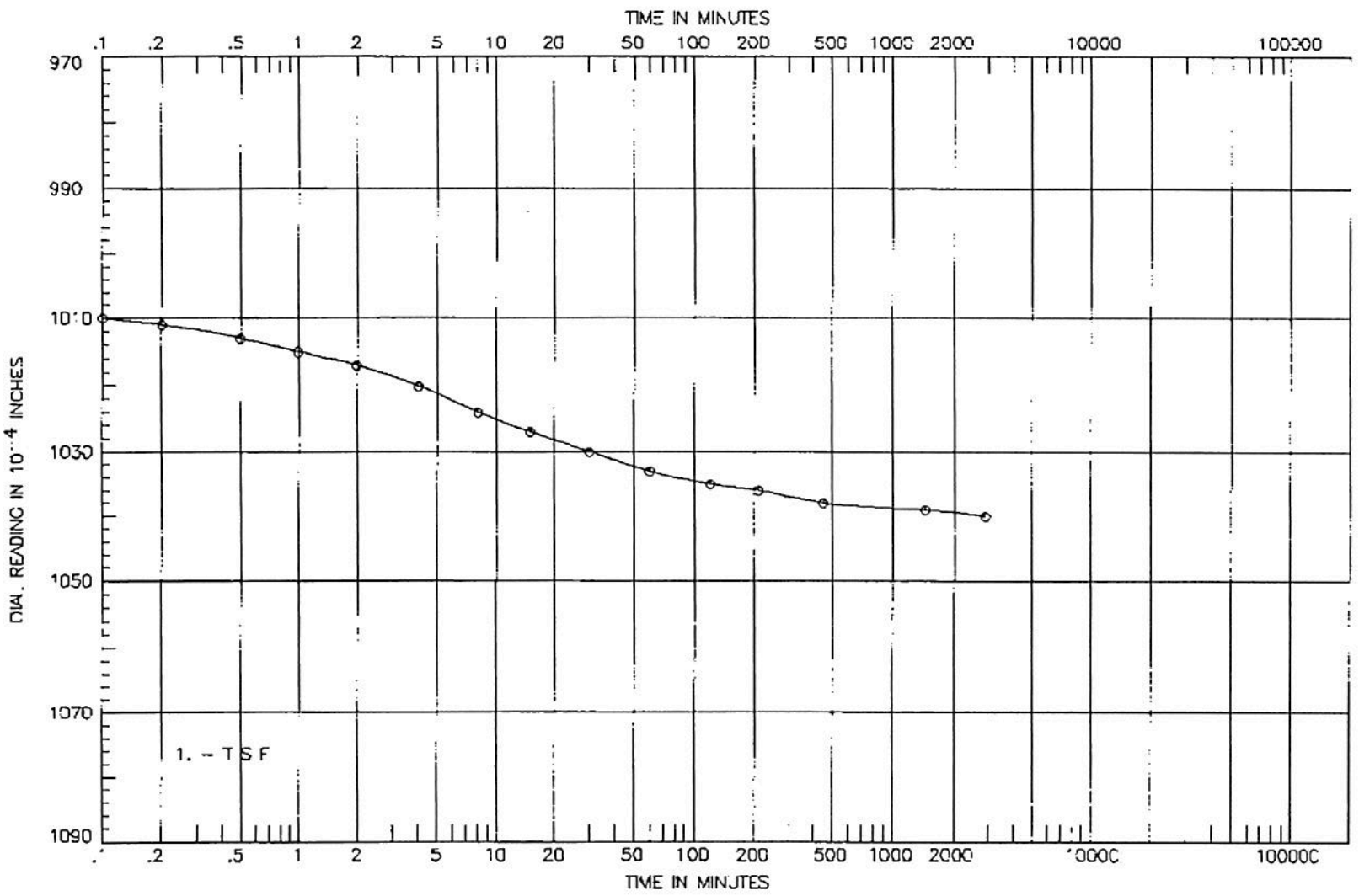


PROJECT DARLINGTON RESERVOIR	
BORING DD-10C	SAMPLE NO. 10C
DEPTH/ELEV 29.1	DATE 22 NOV 96

CONSOLIDATION TEST TIME CURVES

LABORATORY USAE WES - STF/GL

SHEET 2 OF 11



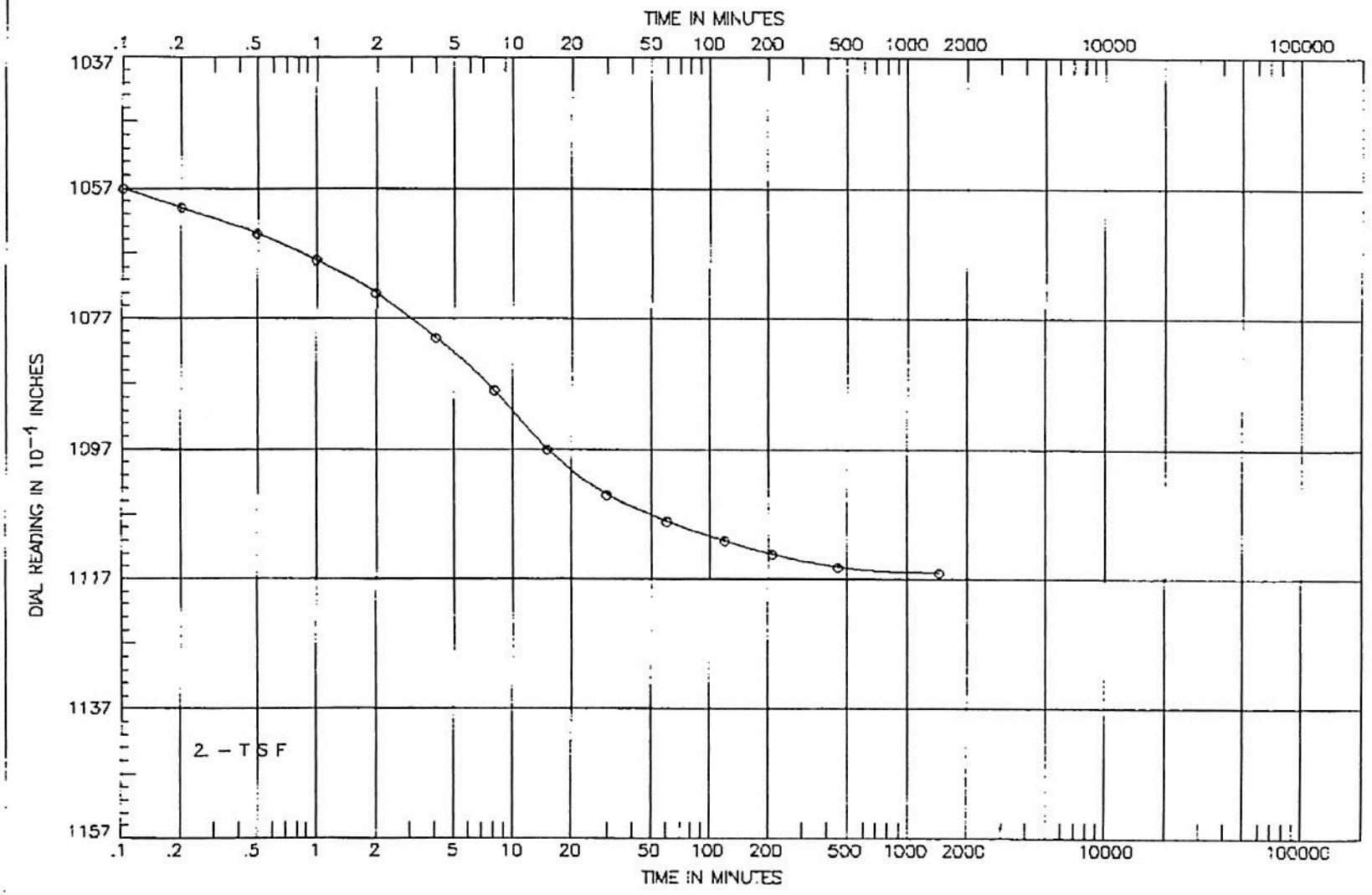
PROJECT DARLINGTON RESERVOIR	
BORING DD-10U	SAMPLE NO. 10C
DEPTH/ELEV 29.1	DATE 22 NOV 96

CONSOLIDATION TEST TIME CURVES

LABORATORY USAE WES - STF/GL

SHEET 3 OF 11

12/13 '96 11:11 ID:CEWES-GS 00 FAX:601-634-4219 PAGE 15

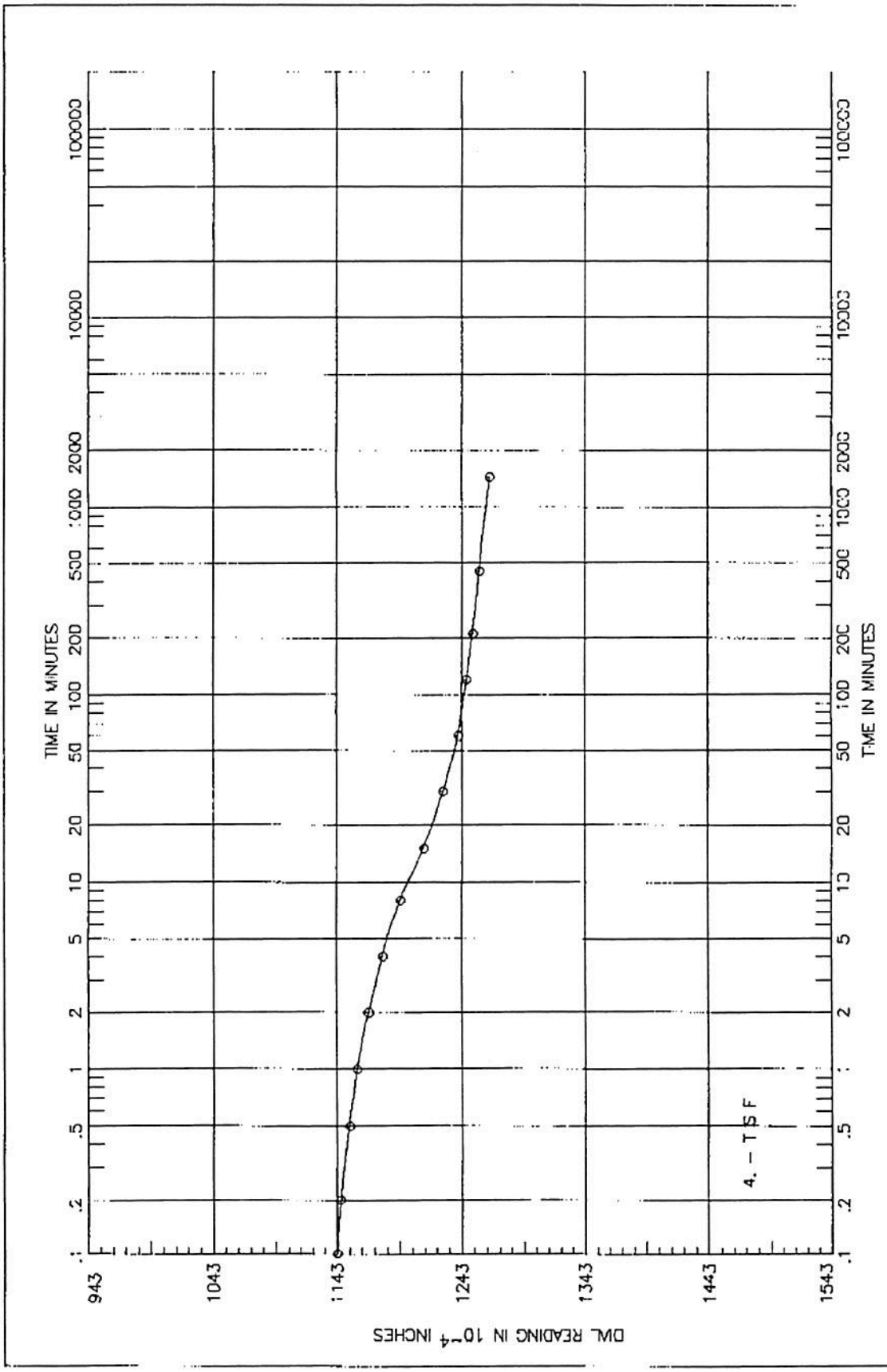


PROJECT DARLINGTON RESERVOIR	
BORING DD-10J	SAMPLE NO. 10C
DEPTH/ELEV 29.1	DATE 22 NOV 96

CONSOLIDATION TEST TIME CURVES

LABORATORY USAE WES - STF/GL

SHEET 4 OF 11

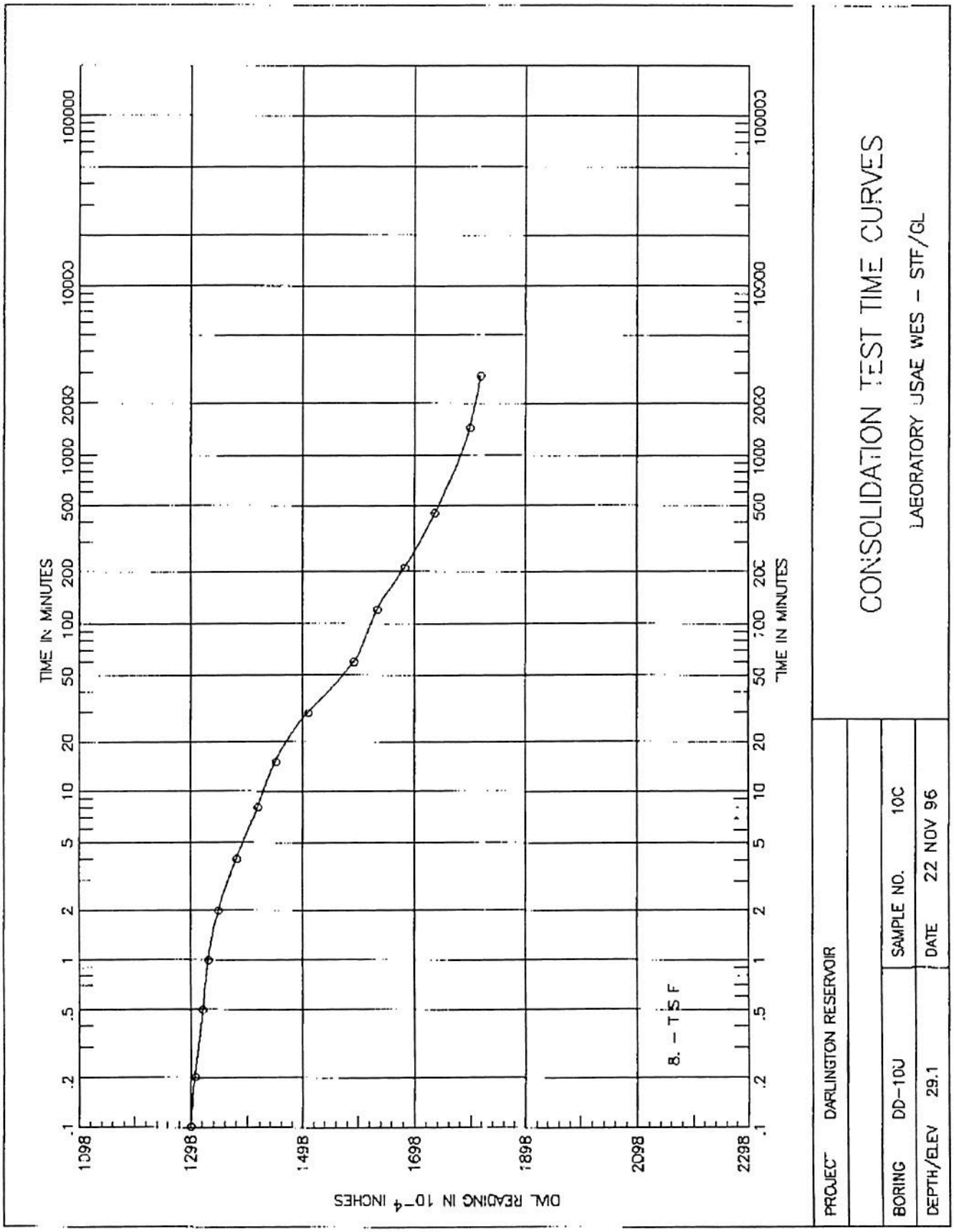


CONSOLIDATION TEST TIME CURVES

LABORATORY USAE WES – STF/GL

PROJECT	DARLINGTON RESERVOIR
BORING	DD-10U
SAMPLE NO.	10C
DEPTH/ELEV	29.1
DATE	22 NOV 96

SHEET 5 OF 11

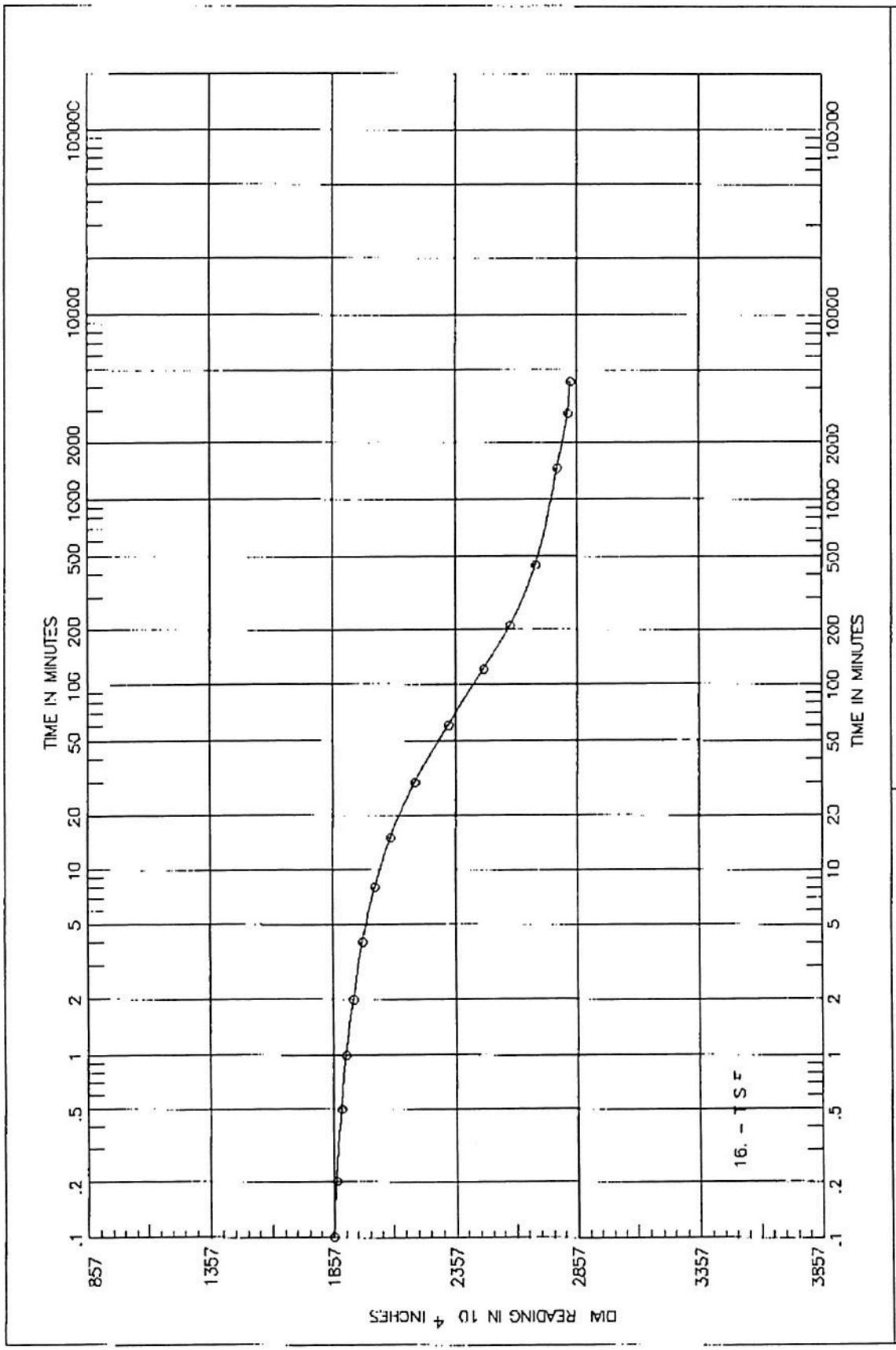


CONSOLIDATION TEST TIME CURVES

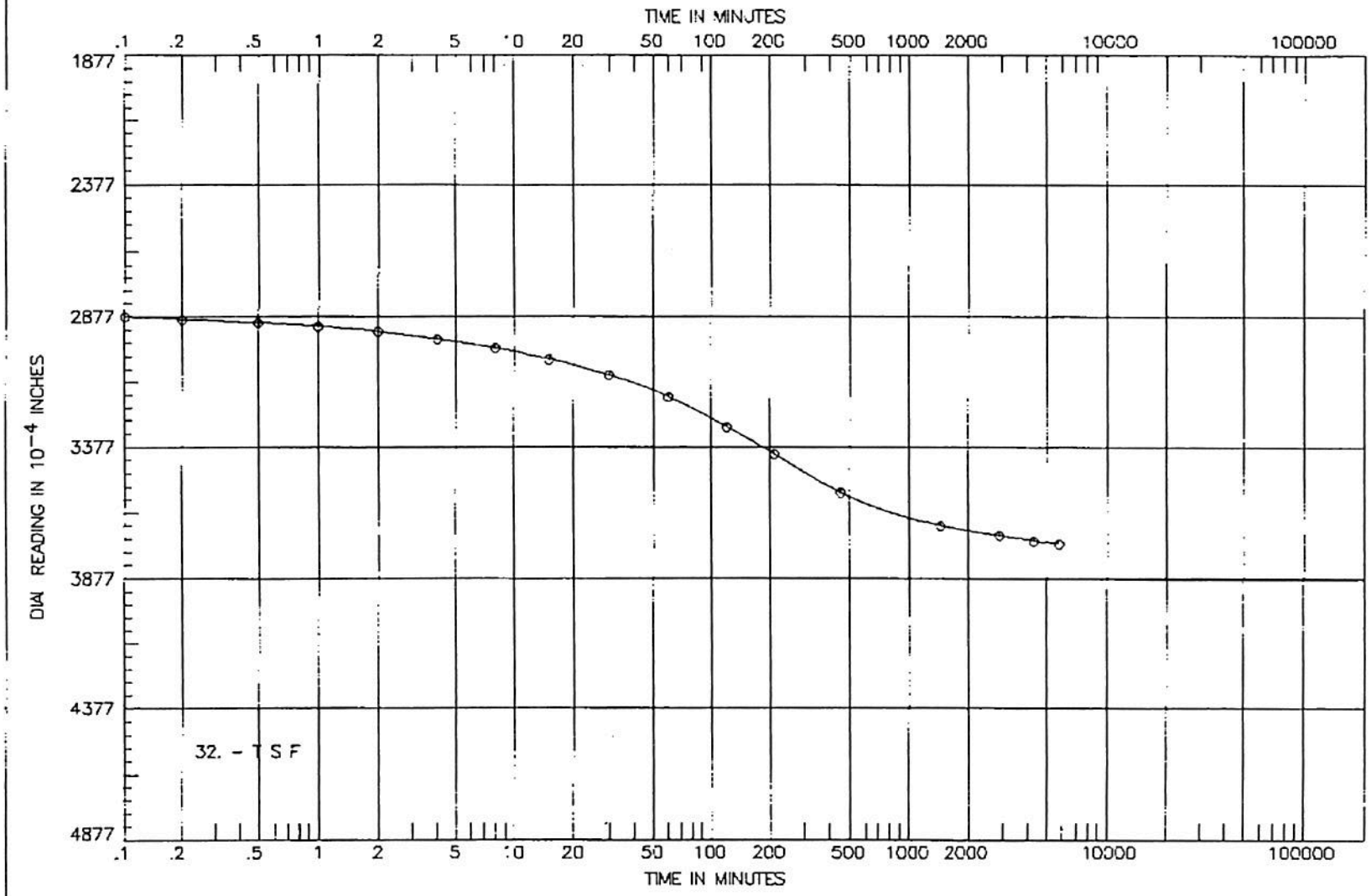
LAEORATORY USAE WES – STF/GL

PROJECT	DARLINGTON RESERVOIR
BORING	DD-10U
SAMPLE NO.	10C
DEPTH/ELEV	29.1
DATE	22 NOV 96

SHEET 6 OF 11


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
.1 .2 .5 1 2 5 10 20 50 100 200 500 1000 2000 10000 100000
857 1357 1857 2357 2857 3357 3857
DIAL READING IN 10 -4 INCHES
PROJECT DARLINGTON RESERVOIR
BORING DD-10U
SAMPLE NO. 10C
DEPT-/ELEV 29.1
DATE 22 NOV 96

SHEET 7 OF 11

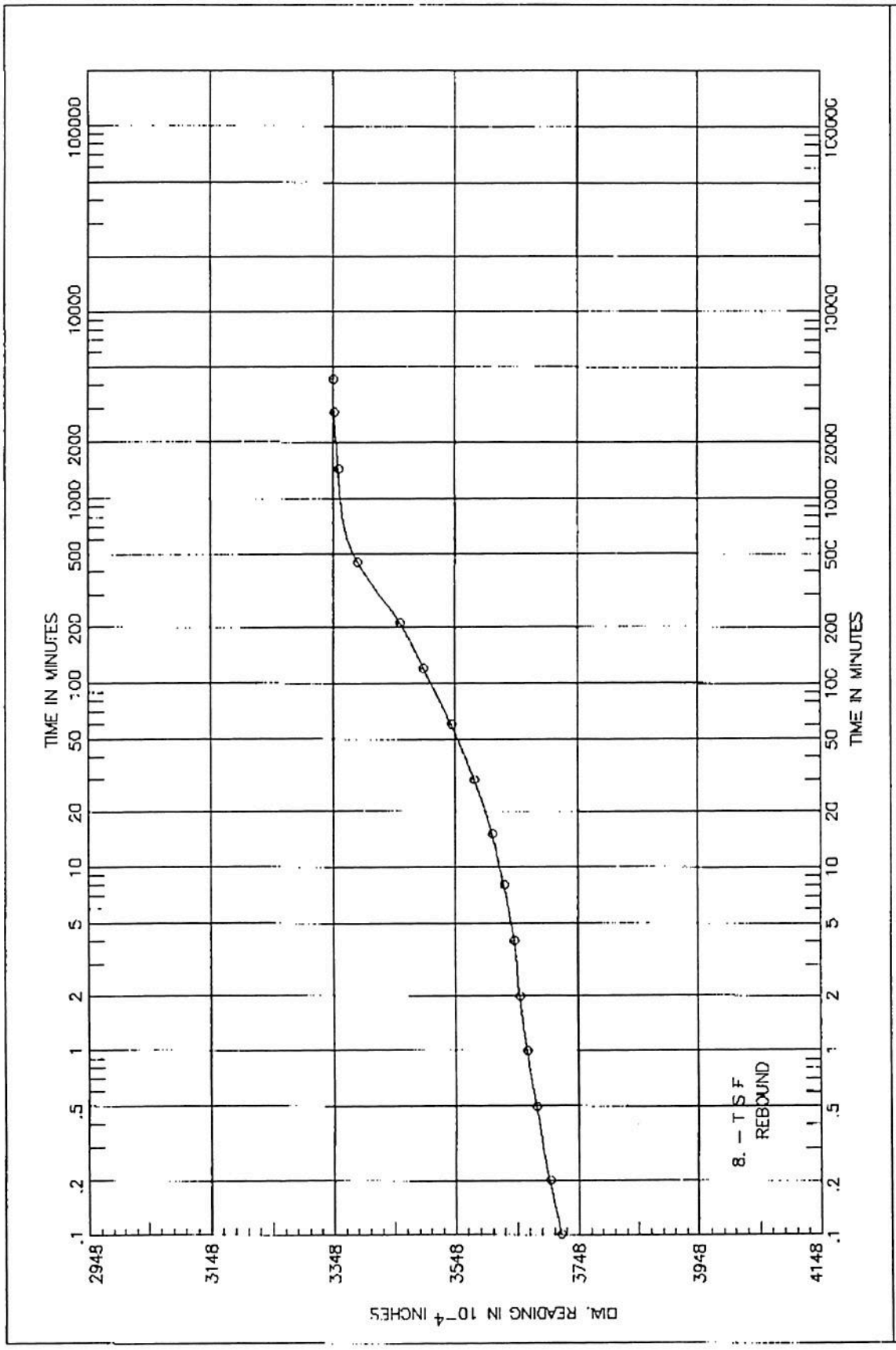


PROJECT DARLINGTON RESERVOIR	
BORING DD-10U	SAMPLE NO. 10C
DEPTH/ELEV 29.1	DATE 22 NOV 96

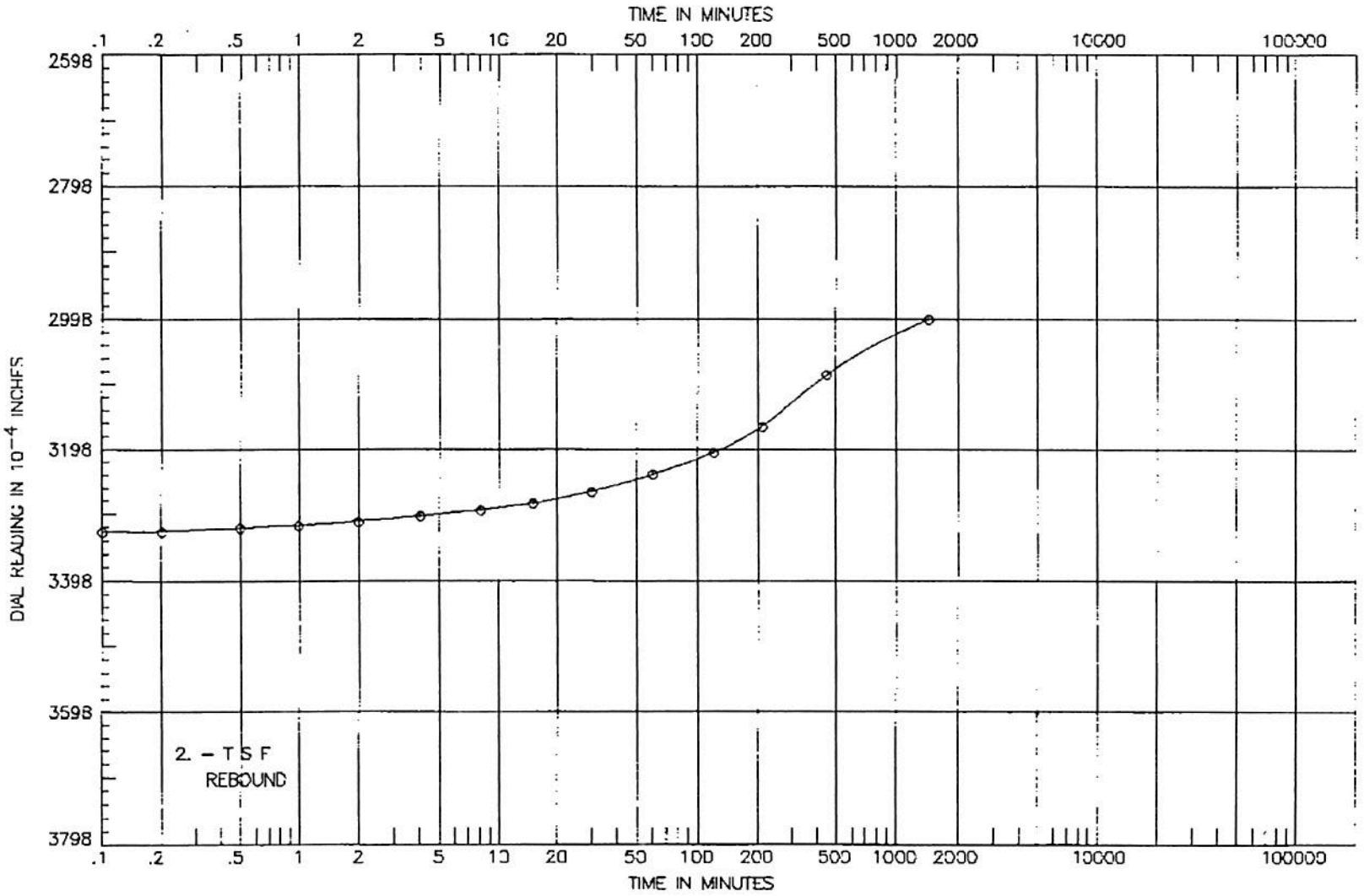
CONSOLIDATION TEST TIME CURVES

LABORATORY USAE WES - STF/GL

SHEET 8 OF 11


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
PROJECT DARLINGTON RESERVOIR
BORING DD-10U
SAMPLE NO. 10C
DEPTH/ELEV 29.
DATE 22 NOV 96
8. - T S F
REBOUND
TIME IN MINUTES
TIME IN MINUTES
DIAL READING IN 10⁻⁴ INCHES
2948
3148
3348
3548
3748
3948
4148
.1
.2
.5
1
2
5
10
20
50
100
200
500
1000
2000
5000
10000
100000

SHEET 9 OF 11

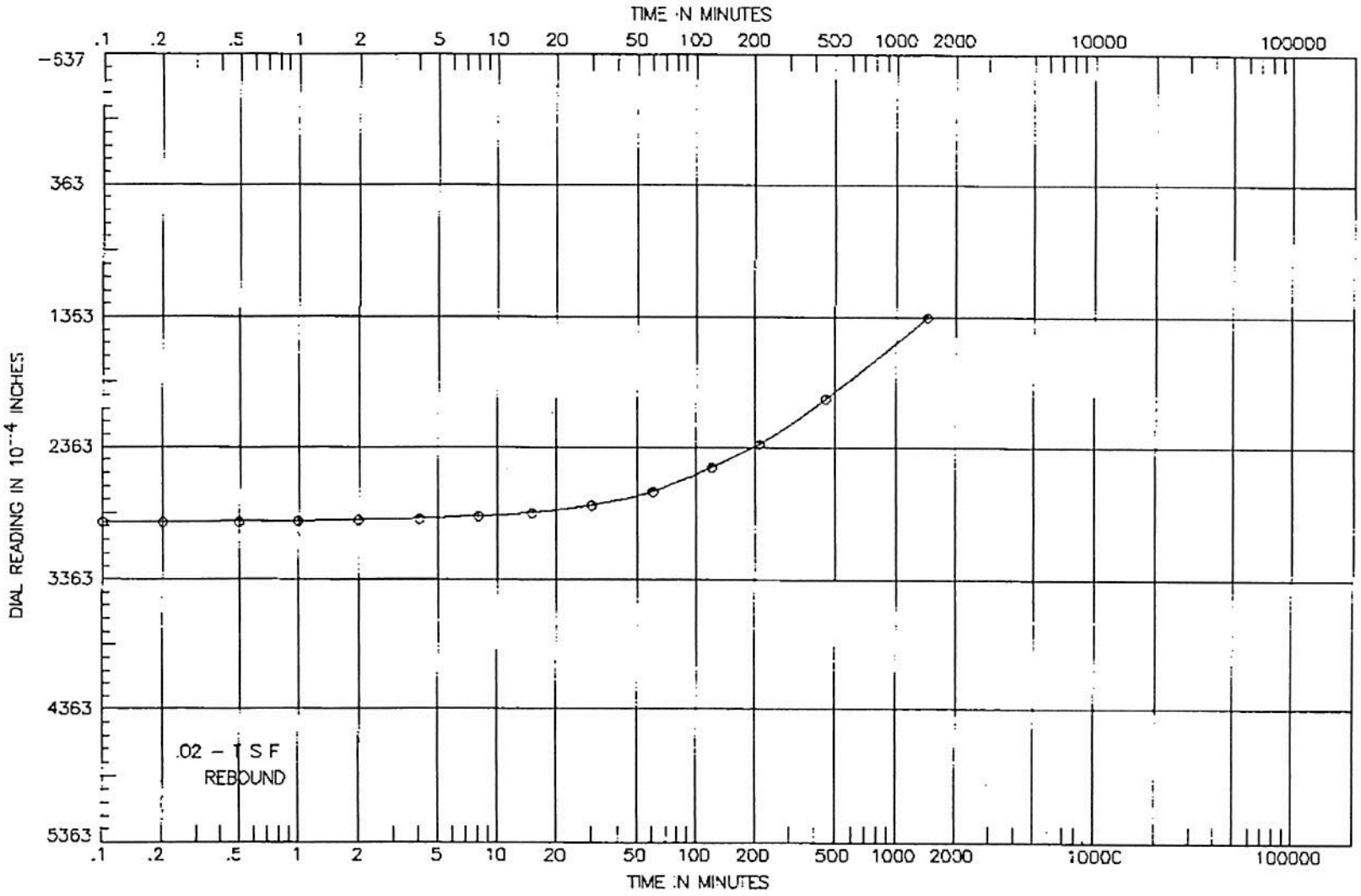


PROJECT DARLINGTON RESERVOIR	
BORING DD-10U	SAMPLE NO. 10C
DEPTH/ELEV 29.1	DATE 22 NOV 96

CONSOLIDATION TEST TIME CURVES

LABORATORY USAE WES - STF/GL

SHEET 10 OF 11

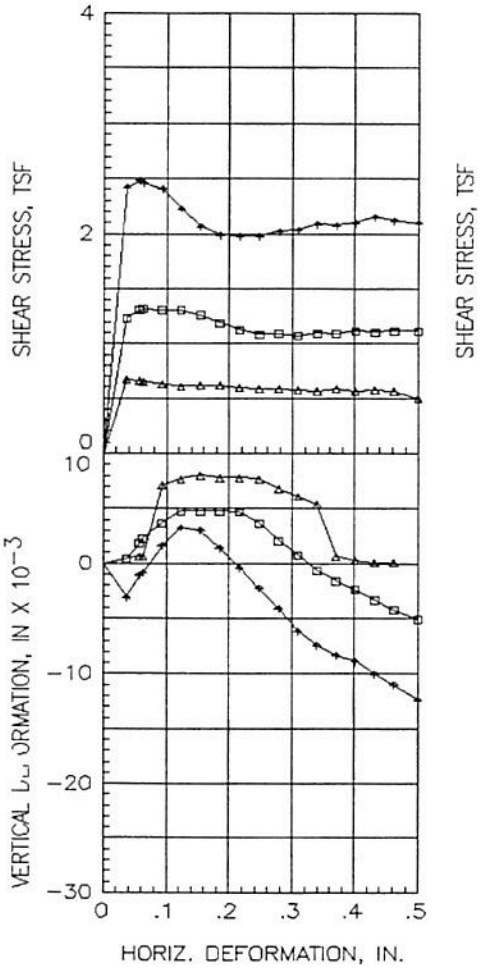


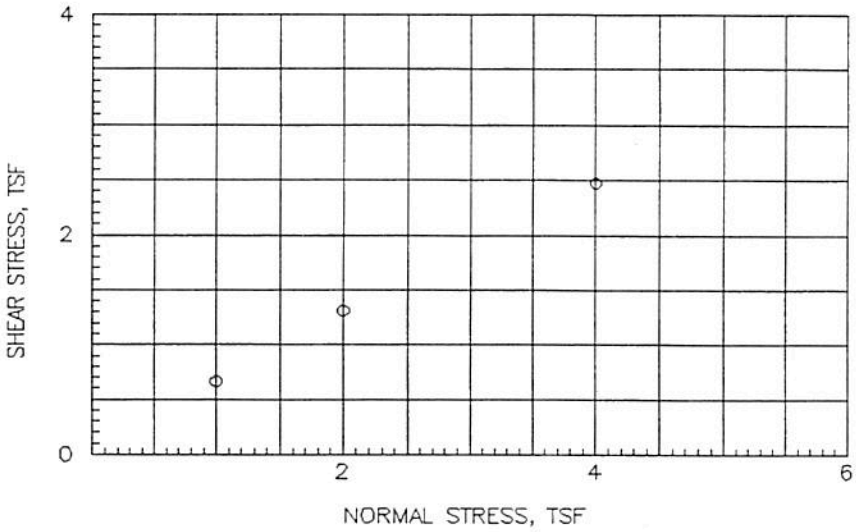
PROJECT DARLINGTON RESERVOIR	
BORING DD-10U	SAMPLE NO. 10C
DEPTH/ELEV 29.1	DATE 22 NOV 96

CONSOLIDATION TEST TIME CURVES

LABORATORY USAE WES - STF/GL

SHEET 11 OF 11





ϕ = _____ DEG

TAN ϕ = _____

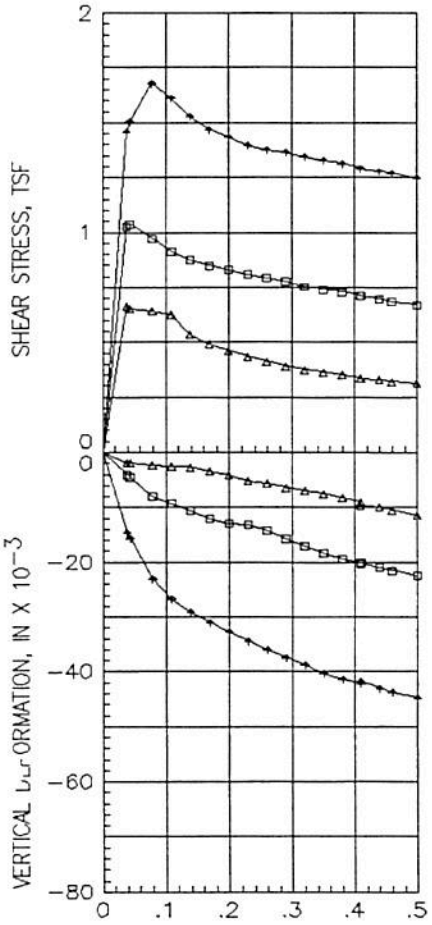
C = _____ T/SF

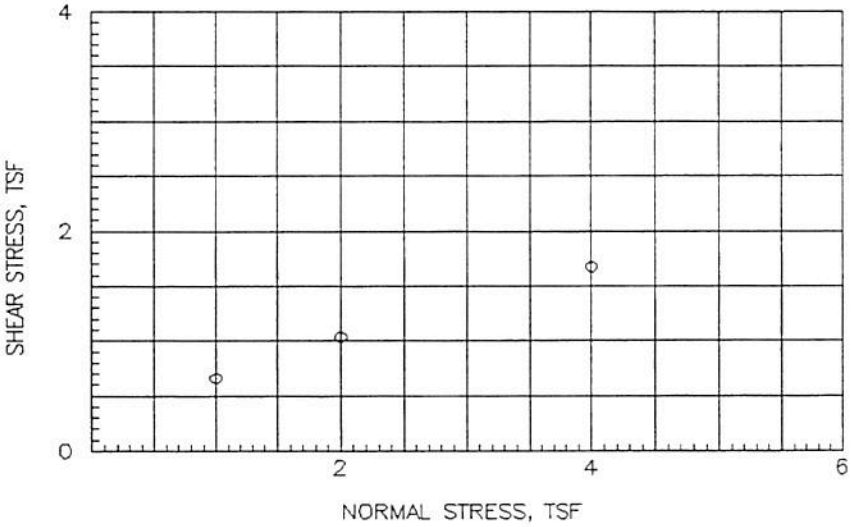
TEST NO.		1 △	2 □	3 +	
INITAL	WATER CONTENT, %	.1	.1	.1	
	VOID RATIO	.566	.566	.566	
	SATURATION, %	.7	.7	.7	
	DRY DENSITY, PCF	105.6	105.6	105.6	
VOID RATIO AFTER CONSOL					
FIFTY PERCENT CONSOL, MIN		< 1	< 1	< 1	
FINAL	WATER CONTENT, %	23.0	22.5	21.9	
	VOID RATIO				
	SATURATION, %				
NORMAL STRESS, TSF		1.00	2.00	4.00	
MAXIMUM SHEAR STRESS, TSF		.67	1.32	2.48	
TIME TO FAILURE, MIN		1	2	2	
RATE OF STRAIN, IN/MIN		.03001	.03001	.03001	
ULTIMATE SHEAR STRESS, TSF		.49	1.07	1.98	

TYPE SPECIMEN UNDISTURBED	3.00 IN. SQUARE	.500 IN. THICK
CLASSIFICATION SAND (SP), BROWN		

LL	PL	PI		GS 2.65 (EST)

REMARKS;	PROJECT DARLINGTON RESERVOIR	
	BORING NO. DD-10U	SAMPLE 1B
	DEPTH/ELEV 1.3	TECH. US
	LABORATORY USAE WES - STF/GL	DATE 19 DEC 96
	DIRECT SHEAR TEST REPORT	





Φ = _____ DEG

TAN Φ = _____

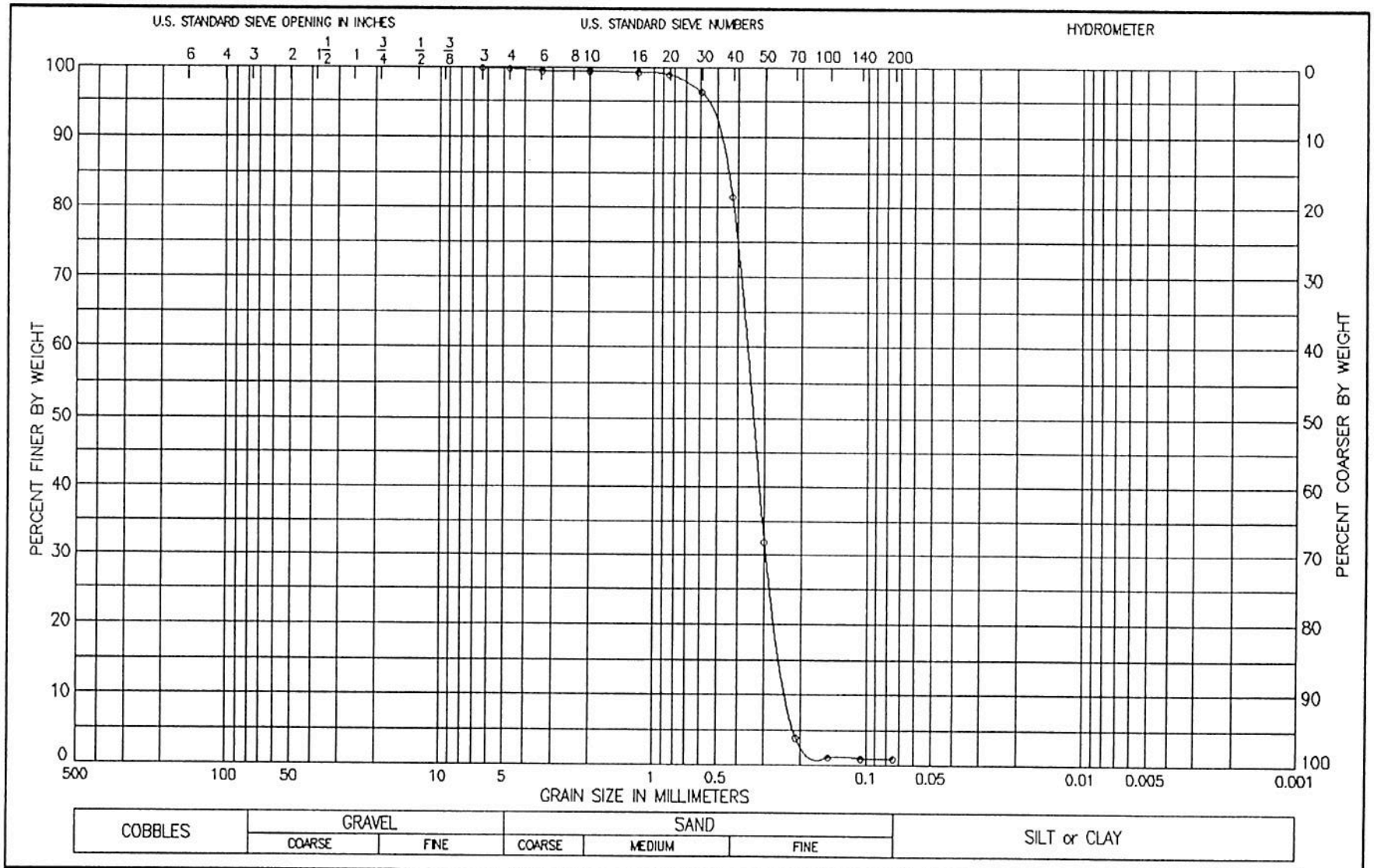
C = _____ T/SF

	TEST NO.	1 △	2 □	3 +	
INITAL	WATER CONTENT, %	40.1	38.9	38.4	
	VOID RATIO	1.152	1.118	1.101	
	SATURATION, %	93.9	94.0	94.1	
	DRY DENSITY, PCF	78.3	79.5	80.2	
VOID RATIO AFTER CONSOL					
FIFTY PERCENT CONSOL, MIN		< 1	< 1	< 1	
FINAL	WATER CONTENT, %	42.2	39.5	34.8	
	VOID RATIO				
	SATURATION, %				
NORMAL STRESS, TSF		1.00	2.00	4.00	
MAXIMUM SHEAR STRESS, TSF		.67	1.04	1.68	
TIME TO FAILURE, MIN		210	238	430	
RATE OF STRAIN, IN/MIN		.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, TSF		.31	.67	1.25	

TYPE SPECIMEN UNDISTURBED	3.00 IN. SQUARE	.628 IN. THICK
CLASSIFICATION CLAY (CH), LIGHT BROWN		

LL 77	PL 18	PI 59		GS 2.70 (EST)

REMARKS;	PROJECT DARLINGTON RESERVOIR	
	BORING NO. DD-10U	SAMPLE 10D2
	DEPTH/ELEV 30.1	TECH. US
	LABORATORY USAE WES – STF/GL	DATE 11 DEC 96
	DIRECT SHEAR TEST REPORT	



LL	PL	PI	GS	NAT W,%	ORG,%

CLASSIFICATION
SAND (SP), BROWN

GRADATION CURVE

LABORATORY USAE WES – STF/GL

PROJECT DARLINGTON RESERVOIR

BORING NO. DD-10U SAMPLE NO. 1B

DEPTH/ELEV 1.3 DATE 29 NOV 96

SIEVE ANALYSIS

PROJECT: DARLINGTON RESERVOIR

BORING: DD-10U SAMPLE: 1B DF: PR0397 .DAT
DEPTH: 1.3 DATE: 29 NOV 96

NO-LIMITS-RAN GS: .00 WC: .00
CLASSIFICATION: 130
SAND (SP), BROWN

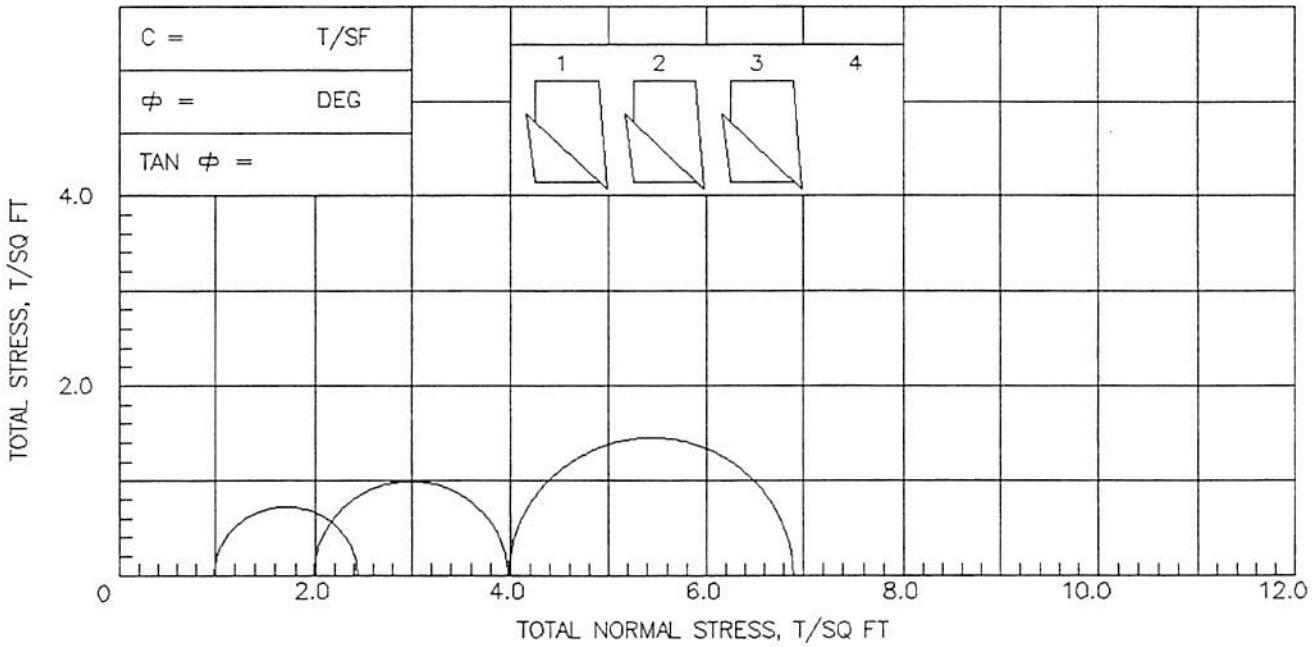
TOTAL WEIGHT OF SAMPLE: 631.4 gms.
PARTIAL WEIGHT AFTER SPLIT: 60.4 gms.

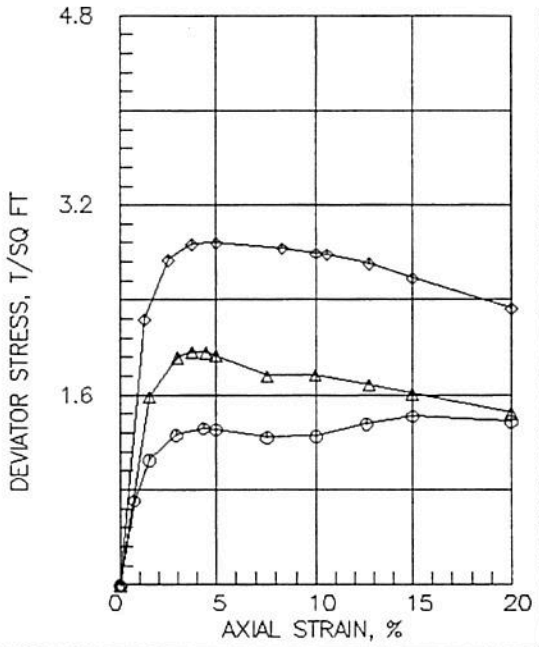
WEIGHTS gm.	SIEVE SIZE or NUMBER	OPENING mm	PERCENT FINER	PERCENT COARSER
.0	No 3	6.350	100.0	.0
.7	No 4	4.750	99.9	.1
.2	No 6	3.350	99.6	.4
.2	No 10	2.000	99.6	.4
.3	No 16	1.180	99.4	.6
.5	No 20	.850	99.1	.9
2.1	No 30	.600	96.4	3.6
11.1	No 40	.425	81.5	18.5
41.1	No 50	.300	31.9	68.1
58.0	No 70	.212	4.0	96.0
59.7	No 100	.150	1.2	98.8
59.8	No 140	.106	1.0	99.0
59.8	No 200	.075	1.0	99.0

PERCENT GRAVEL = .1
PERCENT SAND = 98.9
PERCENT FINES = 1.0

D60 = .37
D30 = .29
D10 = .23
CU = 1.61
CC = 1.01

EDE





SPECIMEN NO.		⊙ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	38.5	38.1	38.3	
	DRY DENSITY, PCF	83.1	82.8	83.1	
	SATURATION, %	100+	99.4	100+	
	VOID RATIO	1.027	1.037	1.030	
BEFORE SHEAR	WATER CONTENT, %	40.6	38.7	36.2	
	DRY DENSITY, PCF	85.7	86.7	90.3	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	.966	.945	.867	
	BACK PRESS., TSF	5.04	5.04	5.04	
MIN. PRIN. STRESS, TSF		1.00	2.00	4.00	
MAX. DEV. STRESS, TSF		1.43	1.97	2.88	
TIME TO FAILURE, MIN.		954	241	327	
RATE OF STRAIN, %/MIN		.016	.016	.016	
INITIAL DIAMETER, IN.		1.35	1.36	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED–STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), LIGHT BROWN; SILT LENSES

LL 54	PL 16	PI 38	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	$\bar{R}$ TEST

REMARKS:

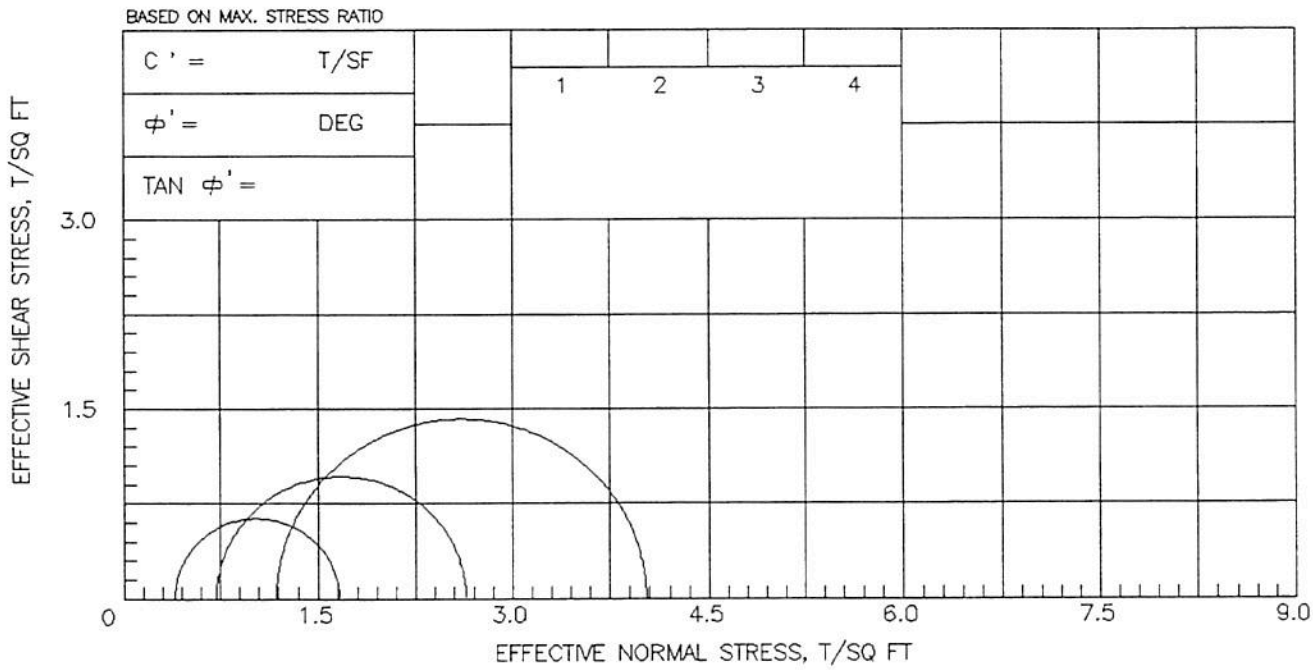
PROJECT DARLINGTON RESERVOIR

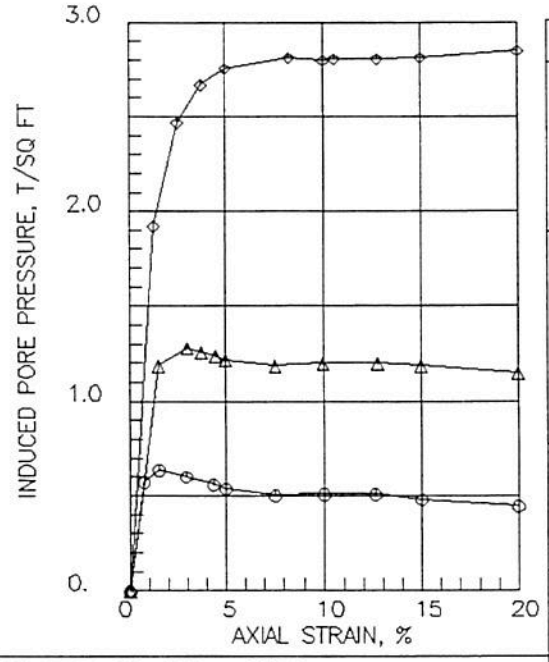
BORING NO. DD–10U SAMPLE NO. 10D1

DEPTH/ELEV 29.8 TECH. JL

LABORATORY USAE WES – STF/GL DATE 14 DEC 96

$\bar{R}$–TRIAXIAL COMPRESSION TEST REPORT





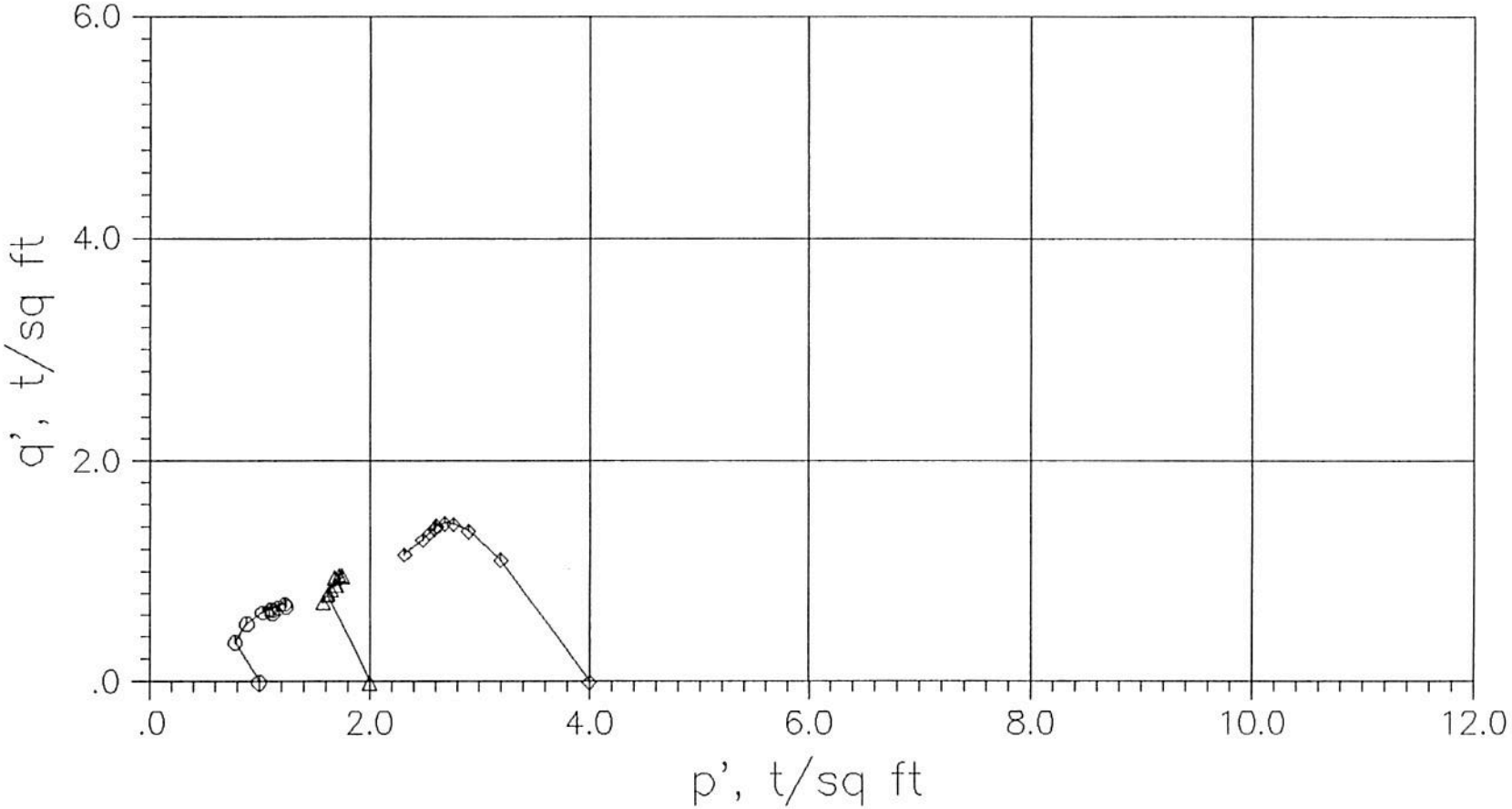
SPECIMEN NO.		⊙ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	38.5	38.1	38.3	
	DRY DENSITY, PCF	83.1	82.8	83.1	
	SATURATION, %	100+	99.4	100+	
	VOID RATIO	1.027	1.037	1.030	
BEFORE SHEAR	WATER CONTENT, %	40.6	38.7	36.2	
	DRY DENSITY, PCF	85.7	86.7	90.3	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	.966	.945	.867	
	BACK PRESS., TSF	5.04	5.04	5.04	
@ $(\sigma_1'/\sigma_3')_{MAX}$	MIN PRIN EFF STRESS, TSF	.40	.72	1.19	
	DEV. STRESS, TSF	1.27	1.92	2.84	
	TIME TO FAILURE, MIN.	186	192	542	
RATE OF STRAIN, %/MIN		.016	.016	.016	
INITIAL DIAMETER, IN.		1.35	1.36	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED–STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), LIGHT BROWN; SILT LENSES

LL 54	PL 16	PI 38	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	$\bar{R}$ TEST

REMARKS:	PROJECT DARLINGTON RESERVOIR	
	BORING NO. DD–10U	SAMPLE NO. 10D1
	DEPTH/ELEV 29.8	TECH. JL
	LABORATORY USAE WES – STF/GL	DATE 14 DEC 96
	$\bar{R}$–TRIAXIAL COMPRESSION TEST REPORT	



PROJECT DARLINGTON RESERVOIR	
BORING DD-10U	SAMPLE NO. 10D1
DEPTH/ELEV 29.8	DATE 14 DEC 96

Stress Paths

LABORATORY USAE WES – STF/GL

R-BAR TRIAXIAL COMPRESSION TEST REPORT

LE: DARLINGTON RESERVOIR

BOR: DD-10U SAM: 10D1 DEP: 29.8 DAT: 14 DEC 96 TEC: JL
DES: CLAY (CH), LIGHT BROWN; SILT LENSES

SAMPLE TYPE = UNDISTURBED
GS: 2.70 (ESTIMATED)
LL: 54 PL: 16 PI: 38

WEIGHT OF SOLIDS = 94.26 I. HEIGHT = 3.000" I. DIAMETER = 1.354"

HEIGHT CHANGES: PRE-CONS. = .00200" DURING CONS. = .03200"

VOLUME CHANGE DURING CONS. = 2.0 CC TIME OF TEST = 1258 MINS.

BACK PRESSURE = 70.00 PSI (5.04 TSF) CHAMBER PRESSURE = 83.89

EFF CONFINING PRESSURE, TSF = 1.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	38.54	40.59
VOID RATIO	1.027	.966
SATURATION, %	101.28	113.43
^RY DENSITY, PCF	83.14	85.73

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	EFFECTIVE E3 - TSF -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	70.0	.0	.00	.00	1.000	1.000	1.000	1.00	.00
.022	14.1	78.0	.7	.58	.71	1.137	.424	2.682	.78	.36
.045	21.0	78.9	1.5	.64	1.05	1.413	.359	3.935	.89	.53
.087	25.6	78.4	2.9	.60	1.27	1.662	.395	4.205	1.03	.63
.129	27.2	77.8	4.3	.56	1.33	1.765	.438	4.025	1.10	.66
.148	27.1	77.5	5.0	.54	1.31	1.773	.460	3.853	1.12	.66
.224	26.5	77.0	7.6	.50	1.25	1.745	.496	3.518	1.12	.62
.297	27.4	77.1	10.0	.51	1.26	1.746	.489	3.571	1.12	.63
.374	30.6	77.1	12.6	.51	1.36	1.852	.489	3.789	1.17	.68
.445	32.9	76.7	15.0	.48	1.43	1.943	.518	3.754	1.23	.71
.593	33.8	76.2	20.0	.45	1.38	1.932	.554	3.490	1.24	.69

RATE OF STRAIN = .016

R-BAR TRIAXIAL COMPRESSION TEST REPORT

LE: DARLINGTON RESERVOIR

BOR: DD-10U SAM: 10D1 DEP: 29.8 DAT: 14 DEC 96 TEC: JL
DES: CLAY (CH), LIGHT BROWN; SILT LENSES

SAMPLE TYPE = UNDISTURBED
GS: 2.70 (ESTIMATED)
LL: 54 PL: 16 PI: 38

WEIGHT OF SOLIDS = 94.53 I. HEIGHT = 3.000" I. DIAMETER = 1.359"

HEIGHT CHANGES: PRE-CONS. = .00700" DURING CONS. = .05900"

VOLUME CHANGE DURING CONS. = 2.7 CC TIME OF TEST = 1258 MINS.

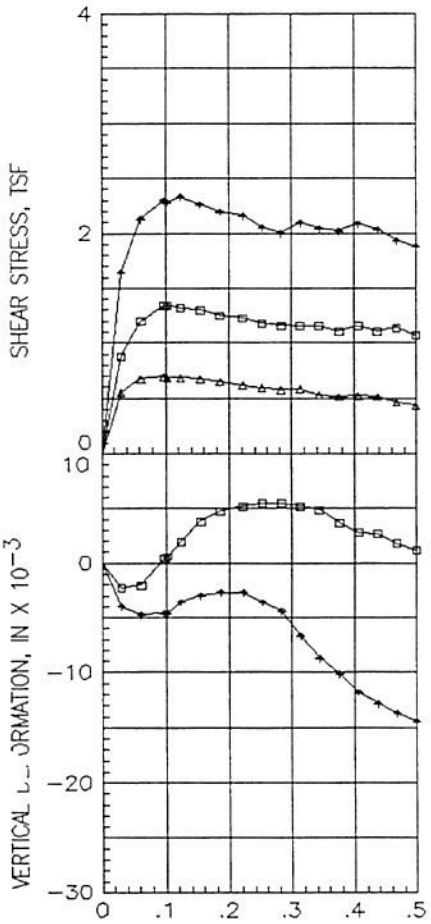
BACK PRESSURE = 70.00 PSI (5.04 TSF) CHAMBER PRESSURE = 97.78

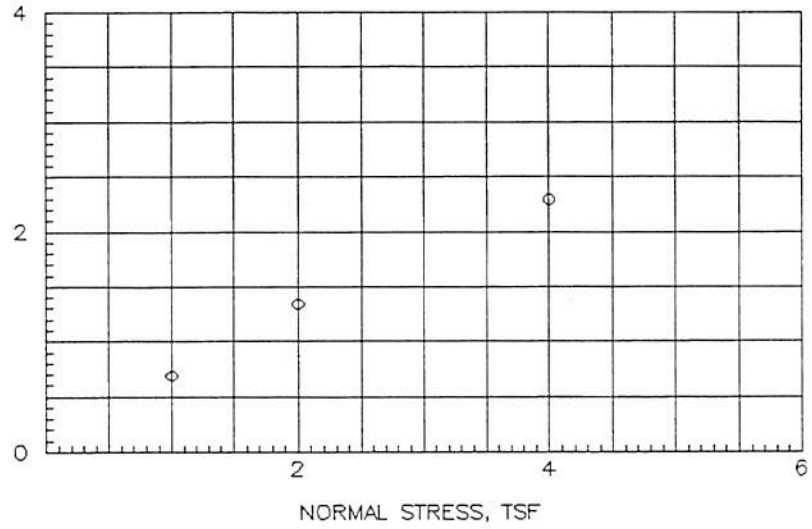
EFF CONFINING PRESSURE, TSF = 2.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	38.15	38.72
VOID RATIO	1.037	.945
SATURATION, %	99.36	110.59
`RY DENSITY, PCF	82.76	86.65

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF	E3 -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	70.0	.0	.00	.00	2.000	2.000	1.000	2.00	.00
.044	31.8	86.5	1.5	1.19	1.59	2.404	.812	2.960	1.61	.80
.088	39.0	87.8	3.0	1.28	1.92	2.641	.719	3.676	1.68	.96
.110	40.2	87.5	3.7	1.26	1.97	2.706	.740	3.657	1.72	.98
.132	40.5	87.2	4.5	1.24	1.97	2.727	.762	3.581	1.74	.98
.147	40.1	86.9	5.0	1.22	1.94	2.719	.783	3.471	1.75	.97
.222	37.6	86.5	7.6	1.19	1.77	2.578	.812	3.175	1.70	.88
.293	38.8	86.7	10.0	1.20	1.77	2.573	.798	3.225	1.69	.89
.374	38.2	86.7	12.7	1.20	1.69	2.492	.798	3.123	1.64	.85
.440	37.4	86.5	15.0	1.19	1.62	2.428	.812	2.989	1.62	.81
.587	35.7	86.0	20.0	1.15	1.45	2.299	.848	2.711	1.57	.73

RATE OF STRAIN = .016





ϕ = _____ DEG

TAN ϕ = _____

C = _____ T/SF

TEST NO.		1 △	2 □	3 +	
INITAL	WATER CONTENT, %	.1	.1	.1	
	VOID RATIO	.566	.566	.566	
	SATURATION, %	.7	.7	.7	
	DRY DENSITY, PCF	105.6	105.6	105.6	
VOID RATIO AFTER CONSOL					
FIFTY PERCENT CONSOL, MIN		< 1	< 1	< 1	
FINAL	WATER CONTENT, %	16.7	16.8	16.3	
	VOID RATIO				
	SATURATION, %				
NORMAL STRESS, TSF		1.00	2.00	4.00	
MAXIMUM SHEAR STRESS, TSF		.70	1.35	2.30	
TIME TO FAILURE, MIN		3	3	3	
RATE OF STRAIN, IN/MIN		.02999	.02999	.02999	
ULTIMATE SHEAR STRESS, TSF		.43	1.06	1.87	

TYPE SPECIMEN UNDISTURBED	3.00 IN. SQUARE	.500 IN. THICK
CLASSIFICATION SAND (SP), LIGHT BROWN		

LL	PL	PI		GS 2.65 (EST)

REMARKS;	PROJECT DARLINGTON RESERVOIR	
	BORING NO. DD-11UA	SAMPLE 9
	DEPTH/ELEV 20.5	TECH. US
	LABORATORY USAE WES - STF/GL	DATE 19 DEC 96
	DIRECT SHEAR TEST REPORT	

R-BAR TRIAXIAL COMPRESSION TEST REPORT

LE: DARLINGTON RESERVOIR

BOR: DD-10U SAM: 10D1 DEP: 29.8 DAT: 14 DEC 96 TEC: JL
DES: CLAY (CH), LIGHT BROWN; SILT LENSES

SAMPLE TYPE = UNDISTURBED
GS: 2.70 (ESTIMATED)
LL: 54 PL: 16 PI: 38

WEIGHT OF SOLIDS = 94.86 I. HEIGHT = 3.000" I. DIAMETER = 1.359"

HEIGHT CHANGES: PRE-CONS. = .00700" DURING CONS. = .10400"

VOLUME CHANGE DURING CONS. = 5.2 CC TIME OF TEST = 1258 MINS.

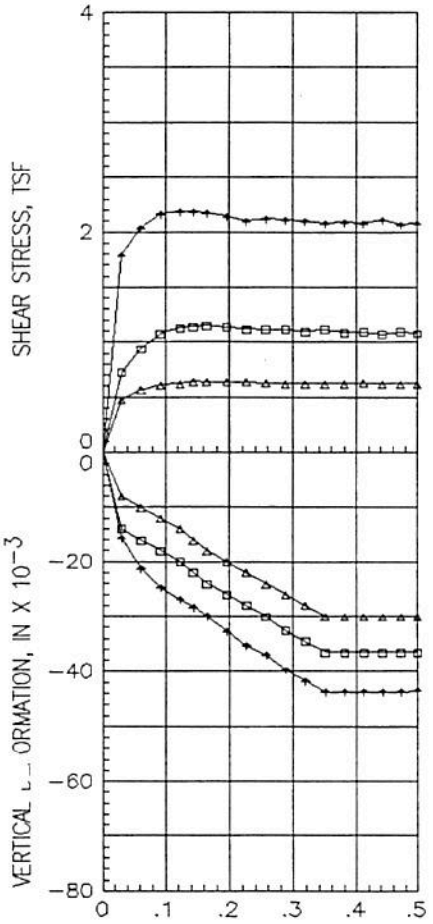
BACK PRESSURE = 70.00 PSI (5.04 TSF) CHAMBER PRESSURE = 125.56

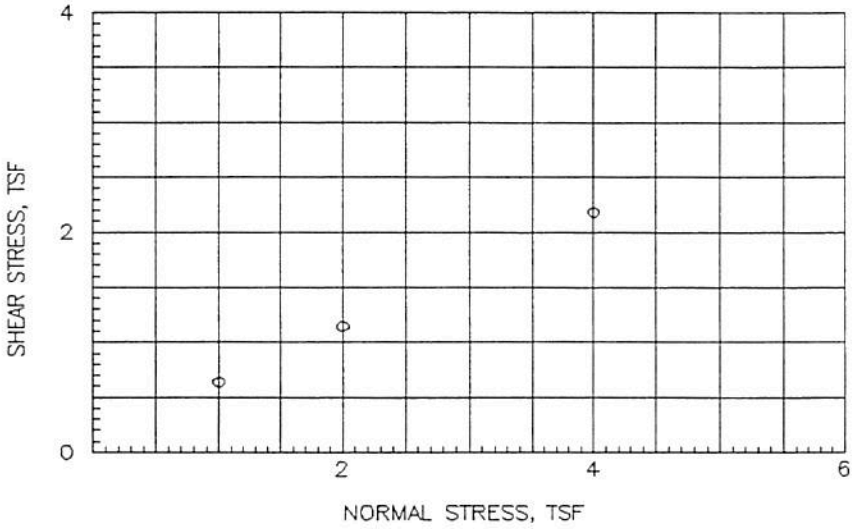
EFF CONFINING PRESSURE, TSF = 4.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	38.30	36.20
VOID RATIO	1.030	.867
SATURATION, %	100.44	112.68
RY DENSITY, PCF	83.05	90.27

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	E3	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	70.0	.0	.00	.00	4.000	4.000	1.000	4.00	.00
.036	43.6	96.7	1.2	1.92	2.24	4.314	2.078	2.076	3.20	1.12
.072	54.1	104.3	2.5	2.47	2.74	4.271	1.531	2.790	2.90	1.37
.108	57.4	107.1	3.7	2.67	2.87	4.199	1.329	3.160	2.76	1.44
.145	58.4	108.3	5.0	2.76	2.88	4.124	1.243	3.319	2.68	1.44
.240	59.6	109.1	8.3	2.81	2.84	4.024	1.185	3.396	2.60	1.42
.289	59.8	108.9	10.0	2.80	2.80	3.995	1.199	3.331	2.60	1.40
.305	60.0	109.0	10.6	2.81	2.79	3.980	1.192	3.338	2.59	1.39
.369	59.8	109.0	12.8	2.81	2.71	3.902	1.192	3.273	2.55	1.35
.433	58.6	109.1	15.0	2.81	2.59	3.773	1.185	3.184	2.48	1.29
.578	56.1	109.6	20.0	2.85	2.33	3.480	1.149	3.029	2.31	1.17

RATE OF STRAIN = .016





Φ = _____ DEG

TAN Φ = _____

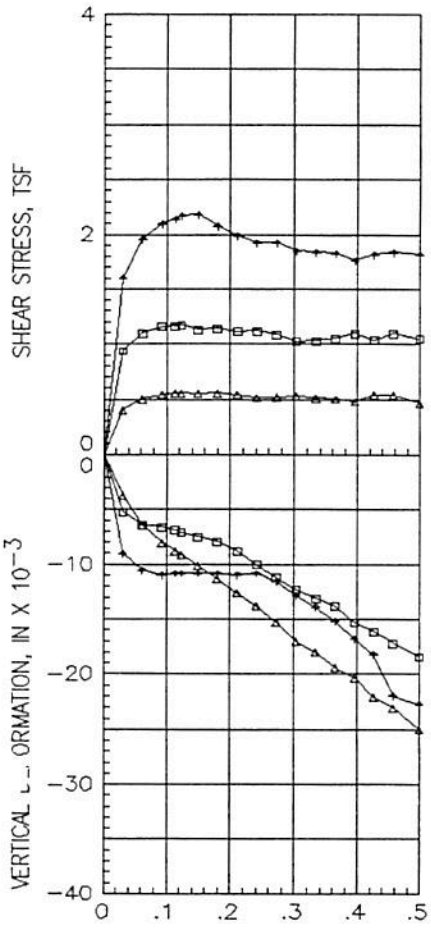
C = _____ T/SF

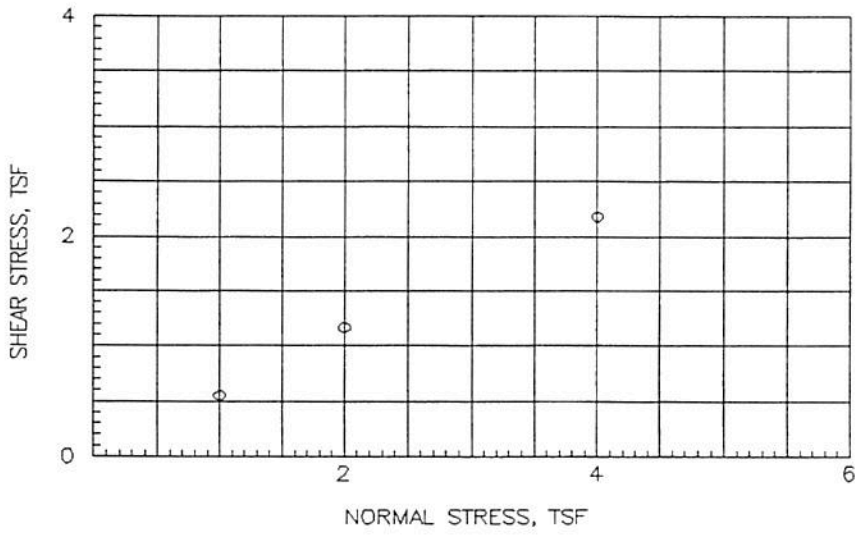
TEST NO.		1 △	2 □	3 +	
INITAL	WATER CONTENT, %	.1	.1	.1	
	VOID RATIO	.914	.933	.981	
	SATURATION, %	.4	.4	.4	
	DRY DENSITY, PCF	87.0	86.2	84.1	
VOID RATIO AFTER CONSOL					
FIFTY PERCENT CONSOL, MIN		< 1	< 1	< 1	
FINAL	WATER CONTENT, %	18.9	19.4	17.7	
	VOID RATIO				
	SATURATION, %				
NORMAL STRESS, TSF		1.00	2.00	4.00	
MAXIMUM SHEAR STRESS, TSF		.65	1.15	2.19	
TIME TO FAILURE, MIN		5	6	4	
RATE OF STRAIN, IN/MIN		.02799	.02799	.02799	
ULTIMATE SHEAR STRESS, TSF		.62	1.07	2.07	

TYPE SPECIMEN UNDISTURBED	3.00 IN. SQUARE	.500 IN. THICK
CLASSIFICATION SILTY SAND (SP-SM), REDDISH BROWN		

LL	PL	PI		GS 2.67 (EST)

REMARKS;	PROJECT DARLINGTON RESERVOIR	
	BORING NO. DD-11UA	SAMPLE 18
	DEPTH/ELEV 45.5	TECH. US
	LABORATORY USAE WES - STF/GL	DATE 19 DEC 96
	DIRECT SHEAR TEST REPORT	





Φ = _____ DEG

TAN Φ = _____

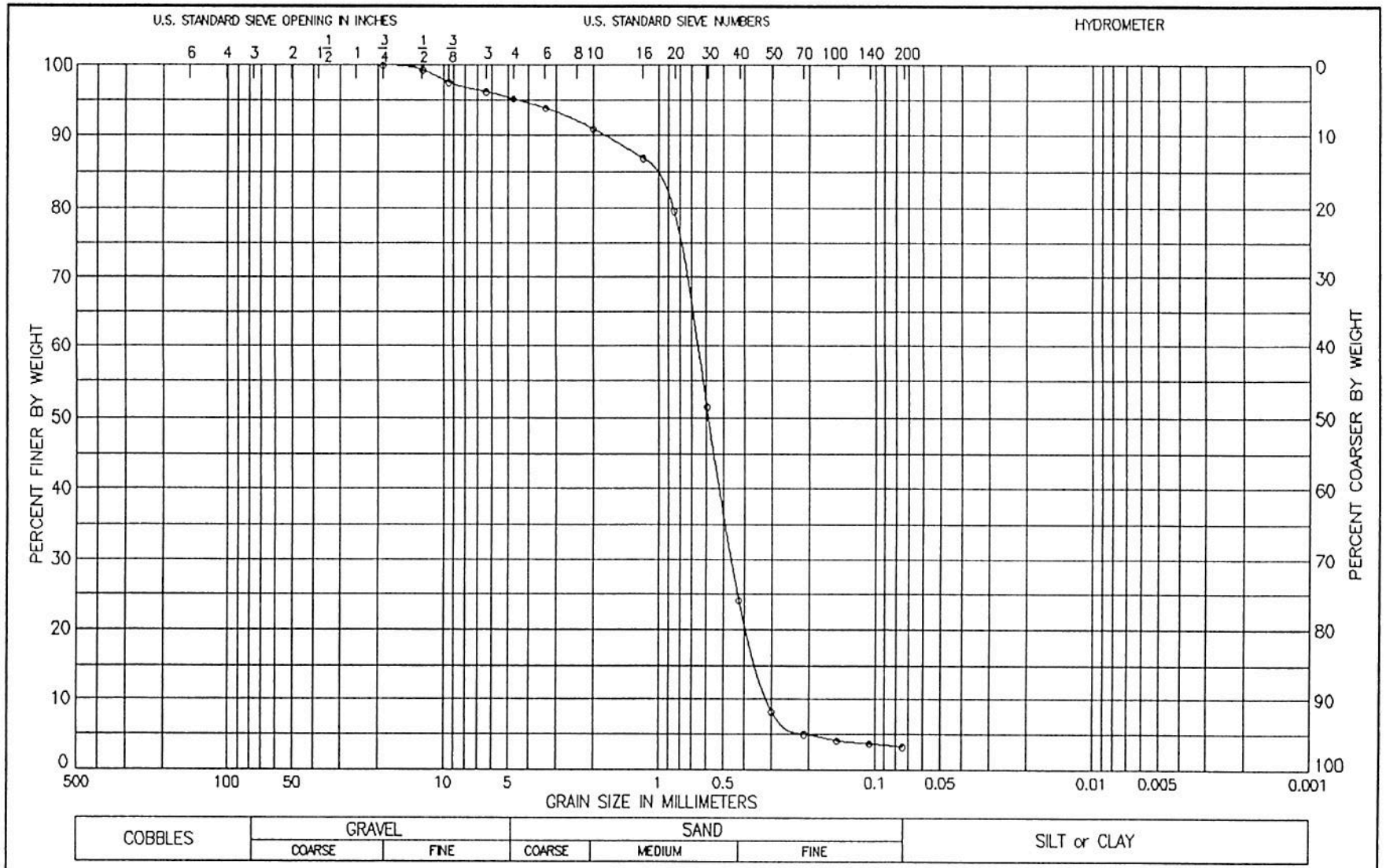
C = _____ T/SF

	TEST NO.	1 △	2 □	3 +	
INITAL	WATER CONTENT, %	.1	.1	.1	
	VOID RATIO	.643	.643	.643	
	SATURATION, %	.6	.6	.6	
	DRY DENSITY, PCF	101.4	101.4	101.4	
VOID RATIO AFTER CONSOL					
FIFTY PERCENT CONSOL, MIN		< 1	< 1	< 1	
FINAL	WATER CONTENT, %	16.5	15.2	15.4	
	VOID RATIO				
	SATURATION, %				
NORMAL STRESS, TSF		1.00	2.00	4.00	
MAXIMUM SHEAR STRESS, TSF		.56	1.17	2.19	
TIME TO FAILURE, MIN		4	4	5	
RATE OF STRAIN, IN/MIN		.02999	.02999	.02999	
ULTIMATE SHEAR STRESS, TSF		.47	1.03	1.76	

TYPE SPECIMEN UNDISTURBED	3.00 IN. SQUARE	.500 IN. THICK
CLASSIFICATION SILTY SAND (SP-SM), BROWN		

LL	PL	PI		GS 2.67 (EST)

REMARKS;	PROJECT DARLINGTON RESERVOIR	
	BORING NO. DD-11UA	SAMPLE 23
	DEPTH/ELEV 58.0	TECH. US
	LABORATORY USAE WES - STF/GL	DATE 20 DEC 96
	DIRECT SHEAR TEST REPORT	



LL	PL	PI	GS	NAT W,%	ORG,%

CLASSIFICATION
SAND (SP), LIGHT BROWN; TRACE OF GRAVEL

GRADATION CURVE | LABORATORY USAE WES – STF/GL

PROJECT DARLINGTON RESERVOIR

BORING NO. DD-11UA SAMPLE NO. 9

DEPTH/ELEV 20.5 DATE 29 NOV 96

SIEVE ANALYSIS

PROJECT: DARLINGTON RESERVOIR

BORING: DD-11UA SAMPLE: 9 DF: PR0397 .DAT
DEPTH: 20.5 DATE: 29 NOV 96

NO-LIMITS-RAN GS: .00 WC: .00
CLASSIFICATION: 142
SAND (SP), LIGHT BROWN; TRACE OF GRAVEL

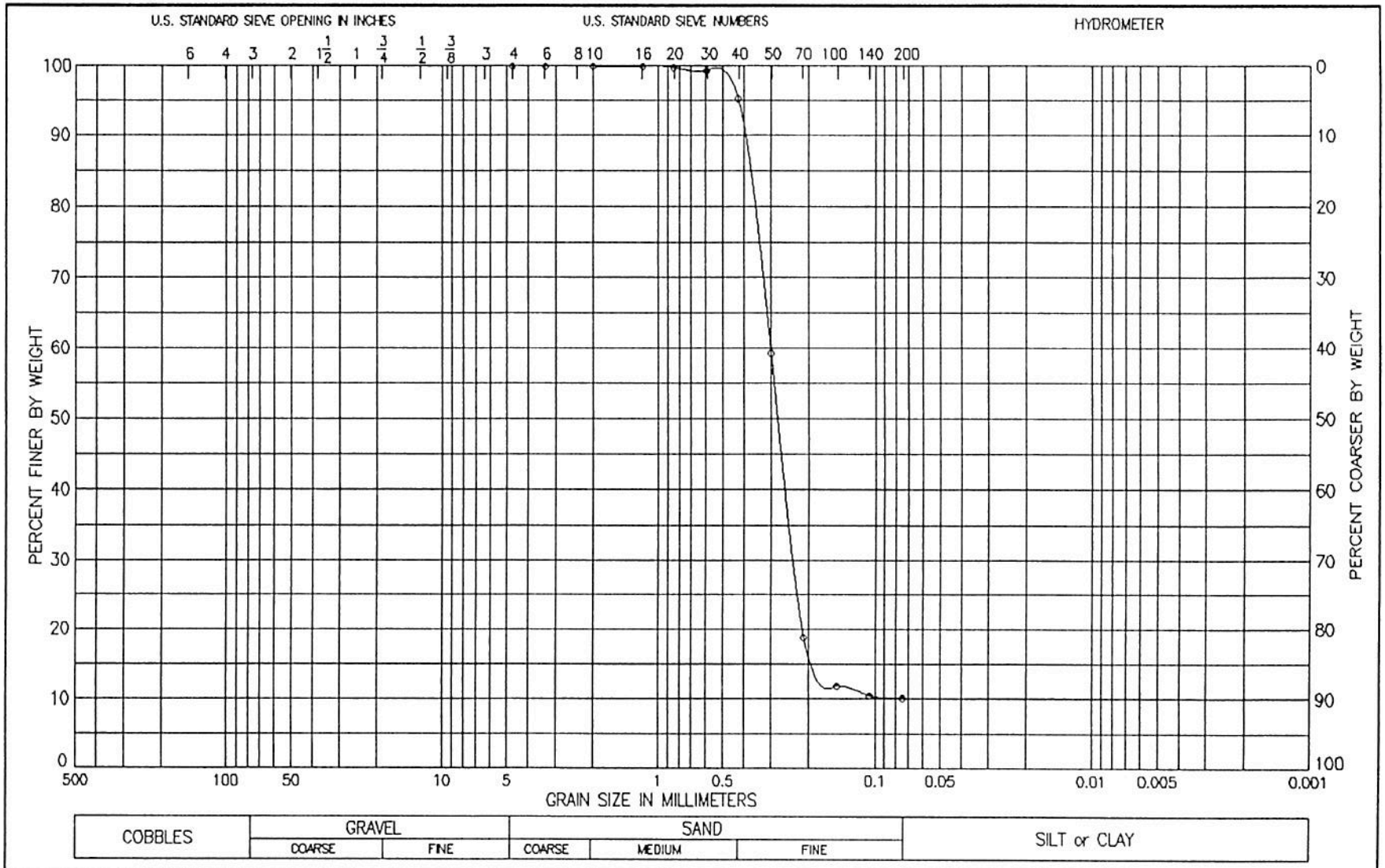
TOTAL WEIGHT OF SAMPLE: 628.7 gms.
PARTIAL WEIGHT AFTER SPLIT: 64.0 gms.

WEIGHTS gm.	SIEVE SIZE or NUMBER	OPENING mm	PERCENT FINER	PERCENT COARSER
.0	3/4 in	19.100	100.0	.0
3.8	1/2 in	12.500	99.4	.6
10.9	3/8 in	9.500	97.7	2.3
8.7	No 3	6.350	96.3	3.7
6.6	No 4	4.750	95.2	4.8
.9	No 6	3.350	93.9	6.1
2.9	No 10	2.000	90.9	9.1
5.6	No 16	1.180	86.9	13.1
10.5	No 20	.850	79.6	20.4
29.4	No 30	.600	51.5	48.5
47.8	No 40	.425	24.1	75.9
58.5	No 50	.300	8.2	91.8
60.6	No 70	.212	5.1	94.9
61.2	No 100	.150	4.2	95.8
61.5	No 140	.106	3.7	96.3
61.8	No 200	.075	3.3	96.7

PERCENT GRAVEL = 4.8
PERCENT SAND = 92.0
PERCENT FINES = 3.3

D60 = .68
D30 = .46
D10 = .31
CU = 2.15
CC = 1.01

EDE



LL	PL	PI	GS	NAT W,%	ORG,%

CLASSIFICATION
SILTY SAND (SP–SM), REDDISH BROWN VISUAL

GRADATION CURVE | LABORATORY USAE WES – STF/GL

PROJECT DARLINGTON RESERVOIR

BORING NO. DD–11UA SAMPLE NO. 18

DEPTH/ELEV 45.5 DATE 29 NOV 96

SIEVE ANALYSIS

PROJECT: DARLINGTON RESERVOIR

BORING: DD-11UA SAMPLE: 18 DF: PR0397 .DAT
DEPTH: 45.5 DATE: 29 NOV 96

NO-LIMITS-RAN GS: .00 WC: .00
CLASSIFICATION: 154
SILTY SAND (SP-SM), REDDISH BROWN VISUAL

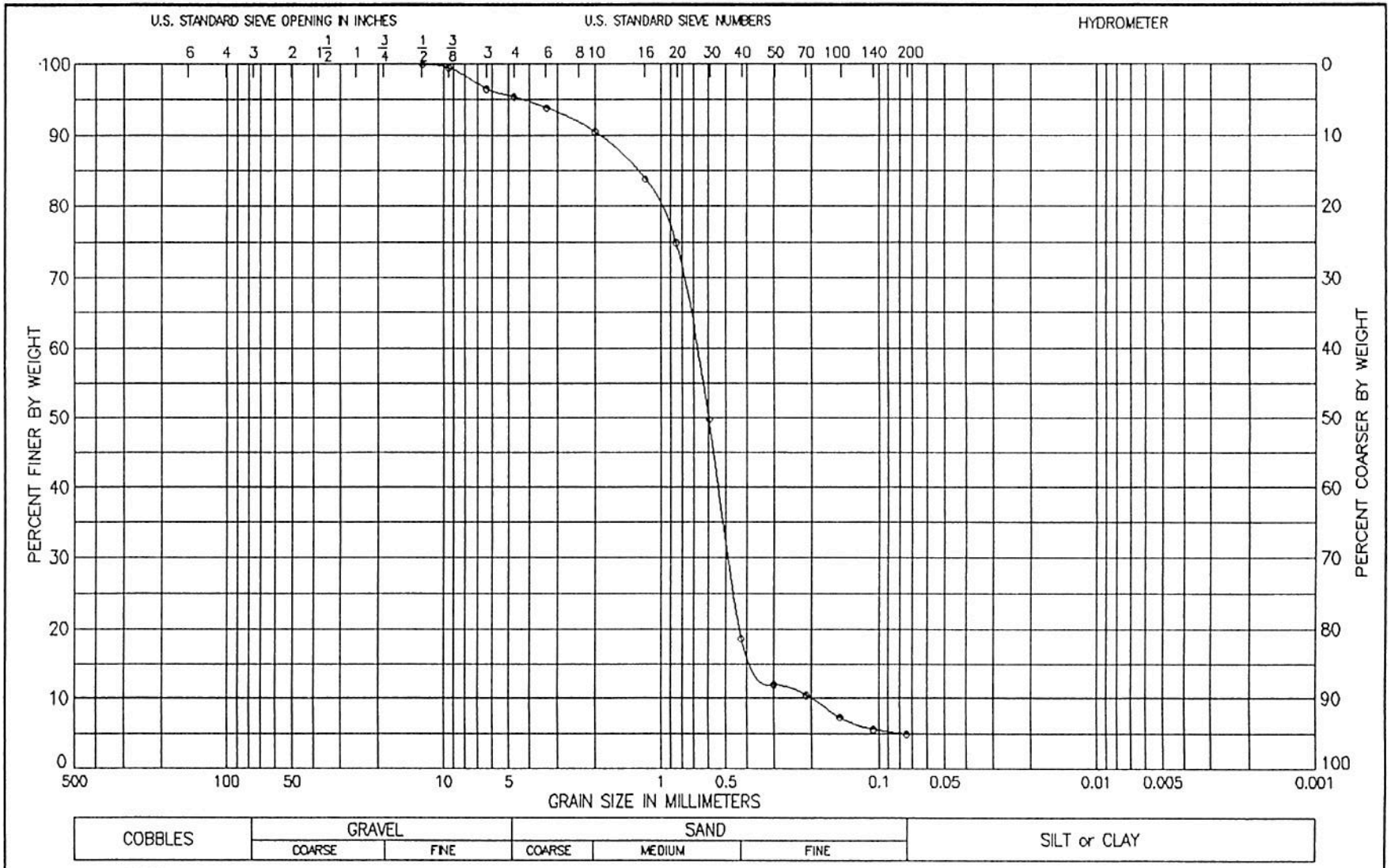
TOTAL WEIGHT OF SAMPLE: .0 gms.
PARTIAL WEIGHT AFTER SPLIT: 58.0 gms.

WEIGHTS gm.	SIEVE SIZE or NUMBER	OPENING mm	PERCENT FINER	PERCENT COARSER
.0	No 4	4.750	100.0	.0
.0	No 6	3.350	100.0	.0
.0	No 10	2.000	100.0	.0
.0	No 16	1.180	100.0	.0
.1	No 20	.850	99.8	.2
.4	No 30	.600	99.3	.7
2.7	No 40	.425	95.3	4.7
23.6	No 50	.300	59.3	40.7
47.1	No 70	.212	18.8	81.2
51.1	No 100	.150	11.9	88.1
51.9	No 140	.106	10.5	89.5
52.1	No 200	.075	10.2	89.8

PERCENT GRAVEL = .0
PERCENT SAND = 89.8
PERCENT FINES = 10.2

D60 = .30
D30 = .24
D10 = .06
CU = 5.08
CC = 3.10

EDE



LL	PL	PI	GS	NAT W,%	ORG,%

CLASSIFICATION
SILTY SAND (SP-SM), BROWN; TRACE OF GRAVEL VISUAL

GRADATION CURVE | LABORATORY USAE WES – STF/GL

PROJECT DARLINGTON RESERVOIR

BORING NO. DD-11UA SAMPLE NO. 23

DEPTH/ELEV 58.0 DATE 29 NOV 96

SIEVE ANALYSIS

PROJECT: DARLINGTON RESERVOIR

BORING: DD-11UA SAMPLE: 23 DF: PR0397 .DAT
DEPTH: 58.0 DATE: 29 NOV 96

NO-LIMITS-RAN GS: .00 WC: .00
CLASSIFICATION: 164
SILTY SAND (SP-SM), BROWN; TRACE OF GRAVEL VISUAL

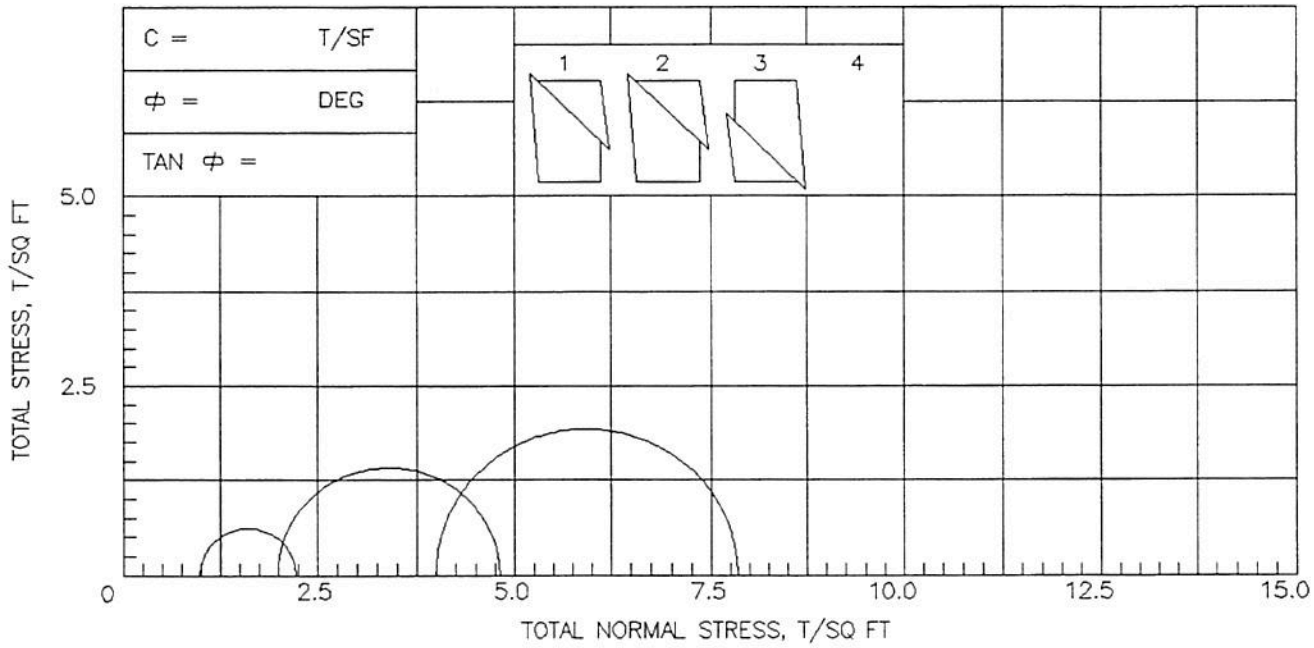
TOTAL WEIGHT OF SAMPLE: 477.6 gms.
PARTIAL WEIGHT AFTER SPLIT: 68.5 gms.

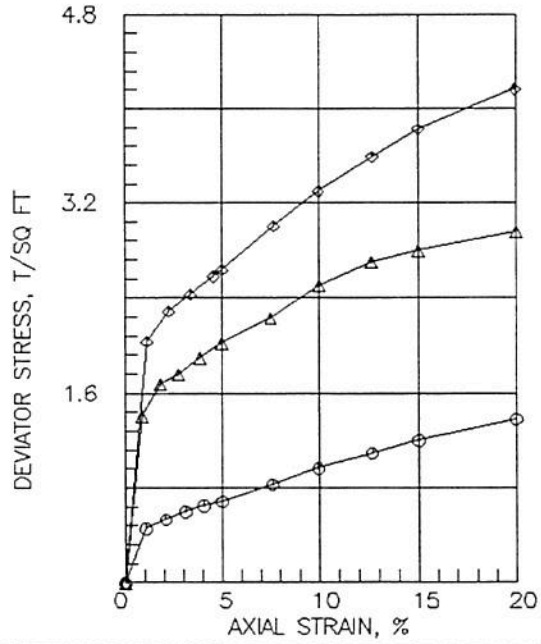
WEIGHTS gm.	SIEVE SIZE or NUMBER	OPENING mm	PERCENT FINER	PERCENT COARSER
.0	1/2 in	12.500	100.0	.0
2.3	3/8 in	9.500	99.5	.5
14.1	No 3	6.350	96.6	3.4
5.4	No 4	4.750	95.4	4.6
1.1	No 6	3.350	93.9	6.1
3.5	No 10	2.000	90.6	9.4
8.3	No 16	1.180	83.9	16.1
14.7	No 20	.850	75.0	25.0
32.8	No 30	.600	49.7	50.3
55.1	No 40	.425	18.7	81.3
59.9	No 50	.300	12.0	88.0
61.0	No 70	.212	10.4	89.6
63.2	No 100	.150	7.4	92.6
64.4	No 140	.106	5.7	94.3
64.9	No 200	.075	5.0	95.0

PERCENT GRAVEL = 4.6
PERCENT SAND = 90.4
PERCENT FINES = 5.0

D60 = .70
D30 = .49
D10 = .20
CU = 3.46
CC = 1.68

EDE





SPECIMEN NO.		⊙ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	21.9	22.4	21.8	
	DRY DENSITY, PCF	101.0	101.6	101.8	
	SATURATION, %	88.2	91.6	89.9	
	VOID RATIO	.669	.659	.655	
BEFORE SHEAR	WATER CONTENT, %	24.7	24.9	23.2	
	DRY DENSITY, PCF	108.6	105.5	109.7	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	.553	.597	.537	
	BACK PRESS., TSF	4.32	4.32	4.32	
MIN. PRIN. STRESS, TSF		1.00	2.00	4.00	
MAX. DEV. STRESS, TSF		1.21	2.80	3.83	
TIME TO FAILURE, MIN.		962	961	967	
RATE OF STRAIN, %/MIN		.016	.016	.016	
INITIAL DIAMETER, IN.		1.35	1.35	1.35	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED–STRAIN TEST

DESCRIPTION OF SPECIMENS: SILTY CLAY (CL), LT. BROWN; IRON OXIDE PARTICLES

LL	PL	PI	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	$\bar{R}$ TEST

REMARKS:

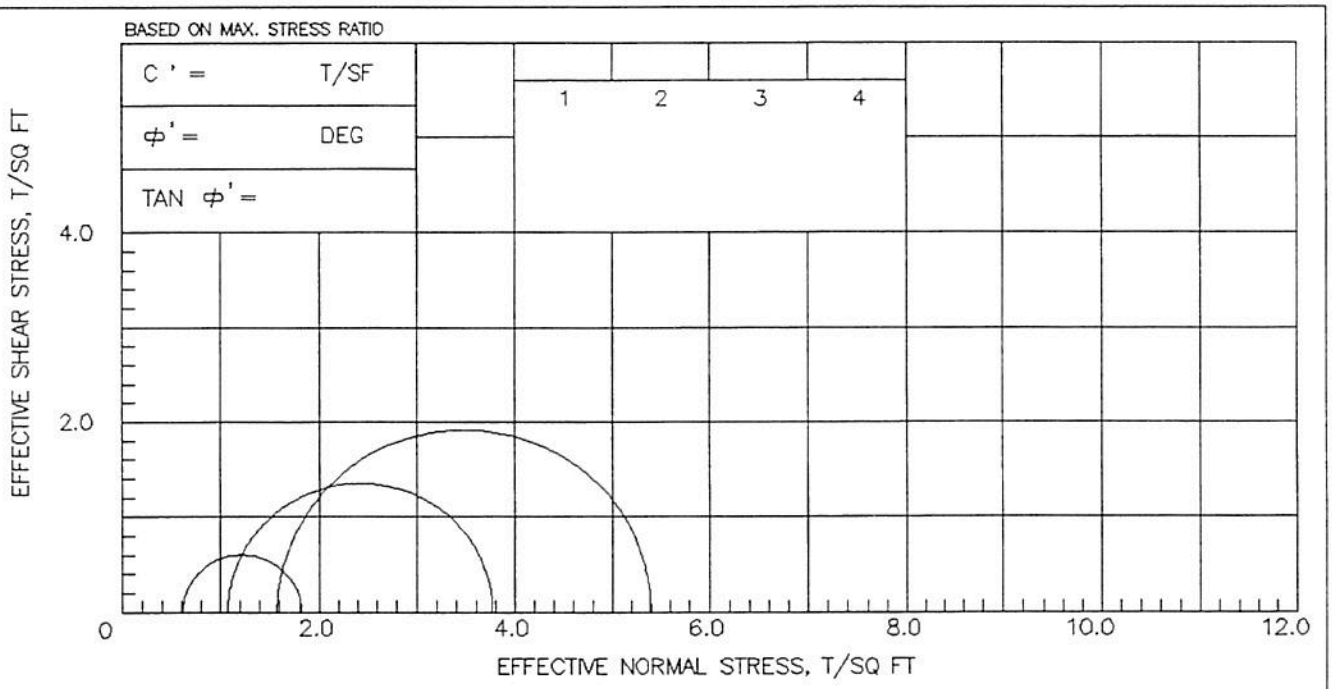
PROJECT DARLINGTON RESERVOIR

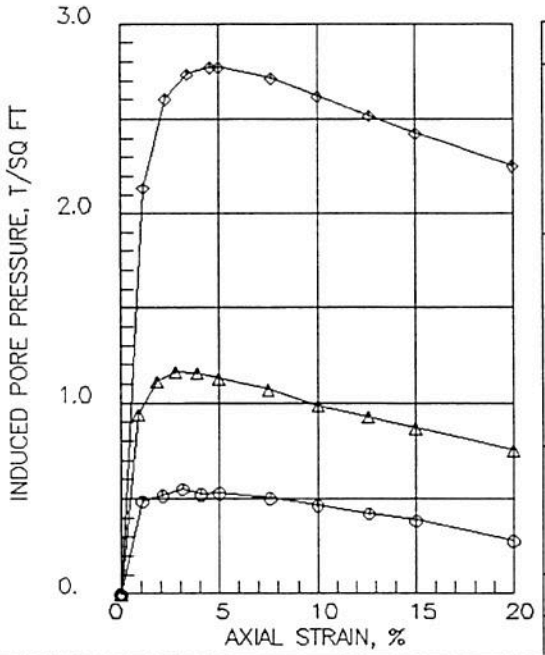
BORING NO. DD–11UA SAMPLE NO. 2C

DEPTH/ELEV 5.6 TECH. JL

LABORATORY USAE WES – STF/GL DATE 18 DEC 96

$\bar{R}$–TRIAXIAL COMPRESSION TEST REPORT





SPECIMEN NO.		⊙ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	21.9	22.4	21.8	
	DRY DENSITY, PCF	101.0	101.6	101.8	
	SATURATION, %	88.2	91.6	89.9	
	VOID RATIO	.669	.659	.655	
BEFORE SHEAR	WATER CONTENT, %	24.7	24.9	23.2	
	DRY DENSITY, PCF	108.6	105.5	109.7	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	.553	.597	.537	
	BACK PRESS., TSF	4.32	4.32	4.32	
@ σ_1'/σ_3' MAX	MIN PRIN EFF STRESS, TSF	.61	1.07	1.57	
	DEV. STRESS, TSF	1.21	2.70	3.83	
	TIME TO FAILURE, MIN.	962	809	967	
RATE OF STRAIN, %/MIN		.016	.016	.016	
INITIAL DIAMETER, IN.		1.35	1.35	1.35	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED–STRAIN TEST

DESCRIPTION OF SPECIMENS: SILTY CLAY (CL), LT. BROWN; IRON OXIDE PARTICLES

LL	PL	PI	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	$\bar{R}$ TEST

REMARKS:

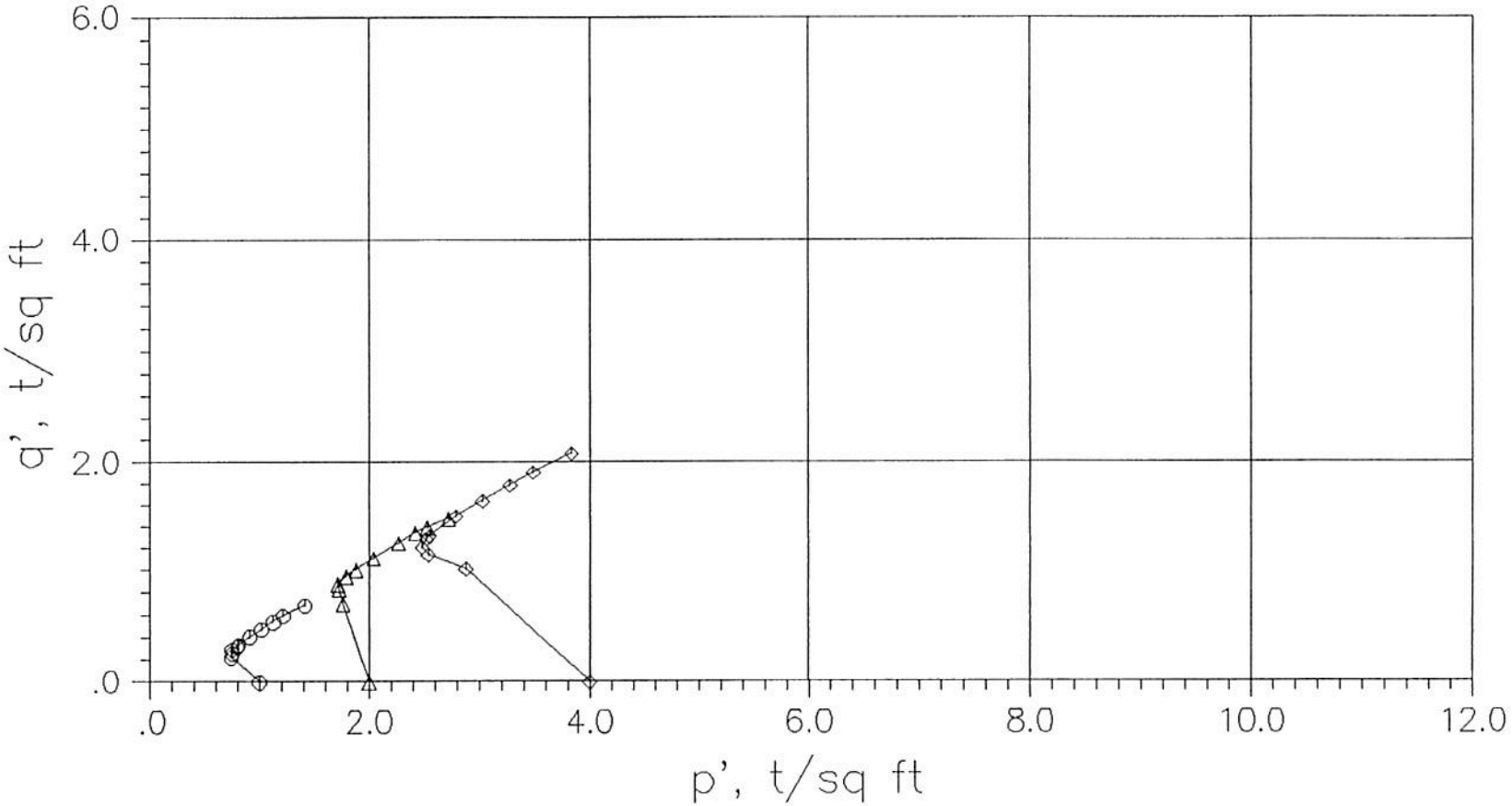
PROJECT DARLINGTON RESERVOIR

BORING NO. DD–11UA SAMPLE NO. 2C

DEPTH/ELEV 5.6 TECH. JL

LABORATORY USAE WES – STF/GL DATE 18 DEC 96

$\bar{R}$–TRIAXIAL COMPRESSION TEST REPORT



PROJECT DARLINGTON RESERVOIR	
BORING DD-11UA	SAMPLE NO. 2C
DEPTH/ELEV 5.6	DATE 18 DEC 96

Stress Paths

LABORATORY USAE WES – STF/GL

R-BAR TRIAXIAL COMPRESSION TEST REPORT

E: DARLINGTON RESERVOIR

BOR: DD-11UA SAM: 2C DEP: 5.6 DAT: 18 DEC 96 TEC: JL
DES: SILTY CLAY (CL), LIGHT BROWN; IRON OXIDE PARTICLES

SAMPLE TYPE = UNDISTURBED
GS: 2.70 (ESTIMATED)
LL: 0 PL: 0 PI: 0

WEIGHT OF SOLIDS = 113.51 I. HEIGHT = 3.000" I. DIAMETER = 1.348"

HEIGHT CHANGES: PRE-CONS. = .01900" DURING CONS. = .02700"

VOLUME CHANGE DURING CONS. = 3.5 CC TIME OF TEST = 1265 MINS.

BACK PRESSURE = 60.00 PSI (4.32 TSF) CHAMBER PRESSURE = 73.89

EFF CONFINING PRESSURE, TSF = 1.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	21.85	24.66
VOID RATIO	.669	.553
SATURATION, %	88.22	120.46
DRY DENSITY, PCF	101.01	108.56

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	EFFECTIVE E3 - TSF -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	60.0	.0	.00	.00	1.000	1.000	1.000	1.00	.00
.032	8.7	66.8	1.1	.49	.46	.970	.510	1.900	.74	.23
.062	10.3	67.2	2.1	.52	.54	1.020	.482	2.118	.75	.27
.092	11.7	67.7	3.1	.55	.61	1.051	.446	2.358	.75	.30
.120	12.8	67.3	4.1	.53	.66	1.130	.474	2.382	.80	.33
.148	13.6	67.4	5.0	.53	.69	1.157	.467	2.476	.81	.34
.224	16.9	67.0	7.6	.50	.83	1.330	.496	2.681	.91	.42
.295	20.2	66.5	10.0	.47	.97	1.503	.532	2.824	1.02	.49
.374	23.6	65.9	12.7	.42	1.10	1.676	.575	2.913	1.13	.55
.443	26.6	65.4	15.0	.39	1.21	1.818	.611	2.975	1.21	.60
.591	32.5	63.9	20.0	.28	1.39	2.107	.719	2.930	1.41	.69

RATE OF STRAIN = .016

R-BAR TRIAXIAL COMPRESSION TEST REPORT

E: DARLINGTON RESERVOIR

BOR: DD-11UA SAM: 2C DEP: 5.6 DAT: 18 DEC 96 TEC: JL
DES: SILTY CLAY (CL), LIGHT BROWN; IRON OXIDE PARTICLES

SAMPLE TYPE = UNDISTURBED
GS: 2.70 (ESTIMATED)
LL: 0 PL: 0 PI: 0

WEIGHT OF SOLIDS = 114.69 I. HEIGHT = 3.000" I. DIAMETER = 1.351"

HEIGHT CHANGES: PRE-CONS. = .01800" DURING CONS. = .02200"

VOLUME CHANGE DURING CONS. = 1.4 CC TIME OF TEST = 1265 MINS.

BACK PRESSURE = 60.00 PSI (4.32 TSF) CHAMBER PRESSURE = 87.78

EFF CONFINING PRESSURE, TSF = 2.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	22.36	24.86
VOID RATIO	.659	.597
SATURATION, %	91.62	112.35
RY DENSITY, PCF	101.61	105.52

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	EFFECTIVE E3 - TSF -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	60.0	.0	.00	.00	2.000	2.000	1.000	2.00	.00
.027	27.7	73.1	.9	.94	1.41	2.469	1.057	2.336	1.76	.71
.054	33.3	75.5	1.8	1.12	1.68	2.566	.884	2.903	1.73	.84
.081	35.2	76.2	2.7	1.17	1.76	2.596	.834	3.113	1.71	.88
.115	38.5	76.1	3.9	1.16	1.90	2.745	.841	3.265	1.79	.95
.148	41.4	75.7	5.0	1.13	2.02	2.894	.870	3.327	1.88	1.01
.222	46.9	74.9	7.5	1.07	2.23	3.160	.927	3.408	2.04	1.12
.296	54.2	73.7	10.0	.99	2.51	3.524	1.014	3.476	2.27	1.26
.374	60.1	72.9	12.6	.93	2.70	3.773	1.071	3.522	2.42	1.35
.444	64.0	72.1	15.0	.87	2.80	3.928	1.129	3.480	2.53	1.40
.592	71.9	70.5	20.0	.76	2.96	4.204	1.244	3.379	2.72	1.48

RATE OF STRAIN = .016

R-BAR TRIAXIAL COMPRESSION TEST REPORT

.E: DARLINGTON RESERVOIR

BOR: DD-11UA SAM: 2C DEP: 5.6 DAT: 18 DEC 96 TEC: JL
DES: SILTY CLAY (CL), LIGHT BROWN; IRON OXIDE PARTICLES

SAMPLE TYPE = UNDISTURBED
GS: 2.70 (ESTIMATED)
LL: 0 PL: 0 PI: 0

WEIGHT OF SOLIDS = 115.11 I. HEIGHT = 3.000" I. DIAMETER = 1.352"

HEIGHT CHANGES: PRE-CONS. = .01800" DURING CONS. = .04000"

VOLUME CHANGE DURING CONS. = 3.8 CC TIME OF TEST = 1265 MINS.

BACK PRESSURE = 60.00 PSI (4.32 TSF) CHAMBER PRESSURE = 115.56

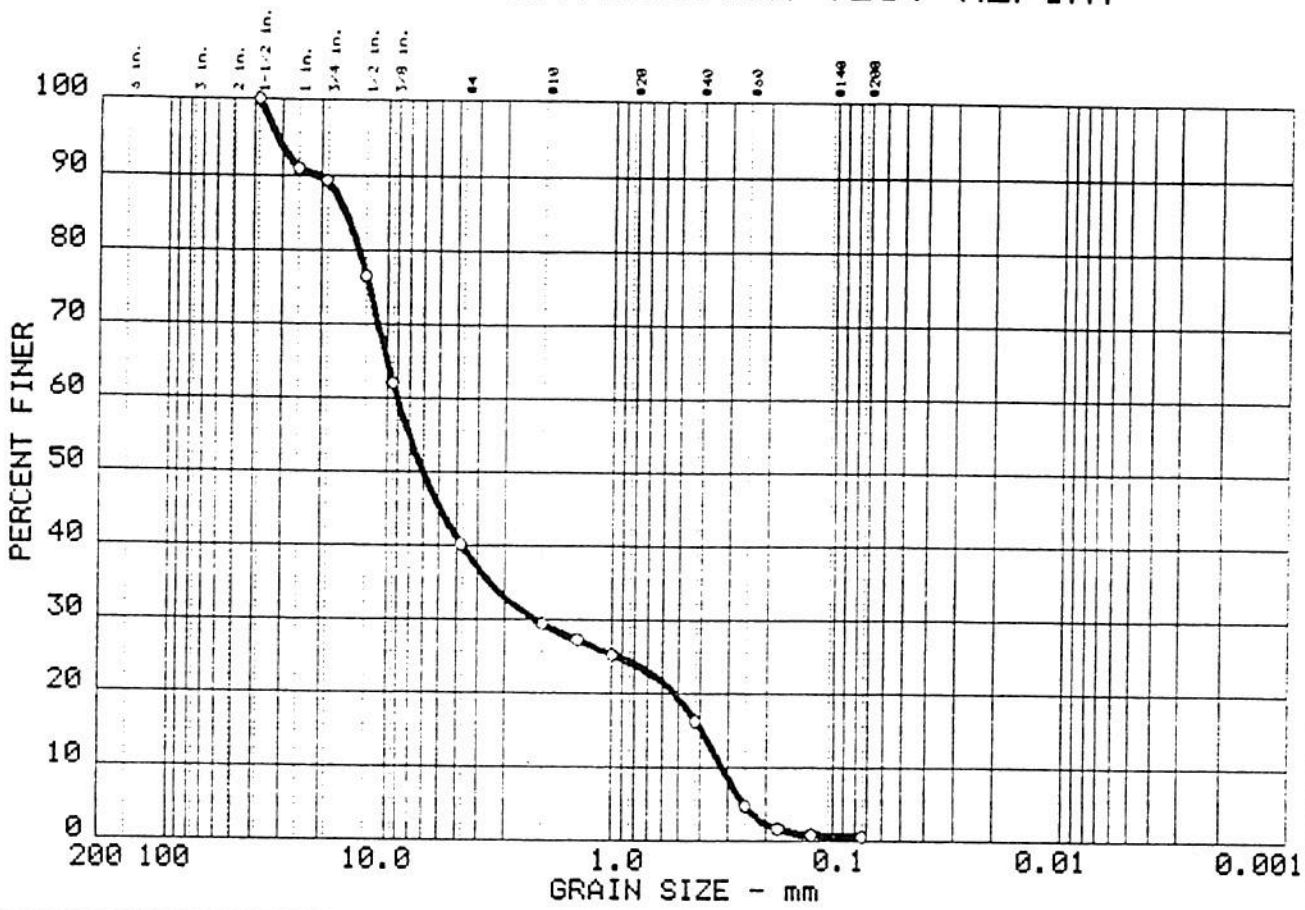
EFF CONFINING PRESSURE, TSF = 4.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	21.83	23.20
VOID RATIO	.655	.537
SATURATION, %	89.94	116.75
DRY DENSITY, PCF	101.83	109.70

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	E3	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	60.0	.0	.00	.00	4.000	4.000	1.000	4.00	.00
.033	38.9	89.7	1.1	2.14	2.04	3.900	1.862	2.095	2.88	1.02
.066	44.3	96.2	2.2	2.61	2.29	3.688	1.394	2.646	2.54	1.15
.099	47.6	98.0	3.4	2.74	2.44	3.701	1.264	2.928	2.48	1.22
.134	51.2	98.5	4.6	2.77	2.59	3.817	1.228	3.108	2.52	1.29
.147	52.5	98.5	5.0	2.77	2.64	3.871	1.228	3.151	2.55	1.32
.225	61.5	97.7	7.6	2.71	3.01	4.295	1.286	3.340	2.79	1.50
.294	69.2	96.4	10.0	2.62	3.30	4.679	1.379	3.392	3.03	1.65
.374	77.7	95.0	12.7	2.52	3.59	5.073	1.480	3.428	3.28	1.80
.441	85.0	93.7	15.0	2.43	3.83	5.402	1.574	3.433	3.49	1.91
.588	98.4	91.3	20.0	2.25	4.17	5.918	1.747	3.388	3.83	2.09

RATE OF STRAIN = .016

GRAIN SIZE DISTRIBUTION TEST REPORT



	Test	%+75mm	% GRAVEL	% SAND	% SILT	% CLAY
○	6	0.0	59.8	39.6	0.6	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			15.65	9.10	6.98	2.109	0.3926	0.3192	1.53	28.5

MATERIAL DESCRIPTION	USCS	AASHTO
○ GRAVEL POORLY GRADED (GP)		

Project No.: 96-24
Project: DARLINGTON RESERVOIR HWY 448
○ Location: BETWEEN DARLINGTON & GRAMERCY

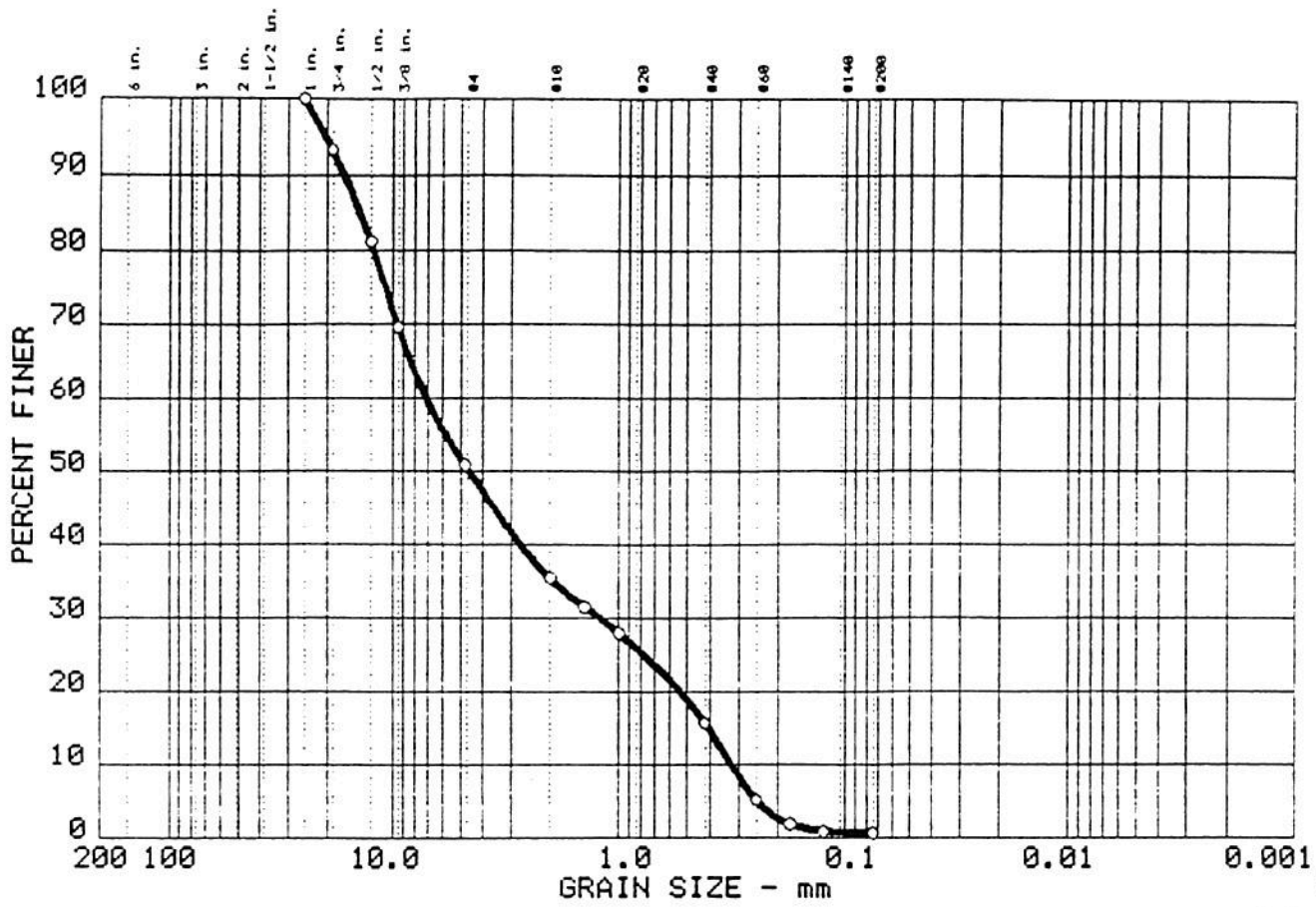
Date: 26NOV96

Remarks:
TECH JMC
BOR: DD-12T

GRAIN SIZE DISTRIBUTION TEST REPORT
USCOE, NEW ORLEANS

Figure No.01

GRAIN SIZE DISTRIBUTION TEST REPORT



	Test	%+75㎜	% GRAVEL	% SAND	% SILT	% CLAY
○	5	0.0	49.1	50.2	0.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			14.13	7.06	4.56	1.212	0.4013	0.3221	0.65	21.9

MATERIAL DESCRIPTION	USCS	AASHTO
○ FINE SAND (SP)		

Project No.: 96-24
Project: DARLINGTON RESERVOIR HWY 448
○ Location: BETWEEN DARLINGTON & GRAMERCY

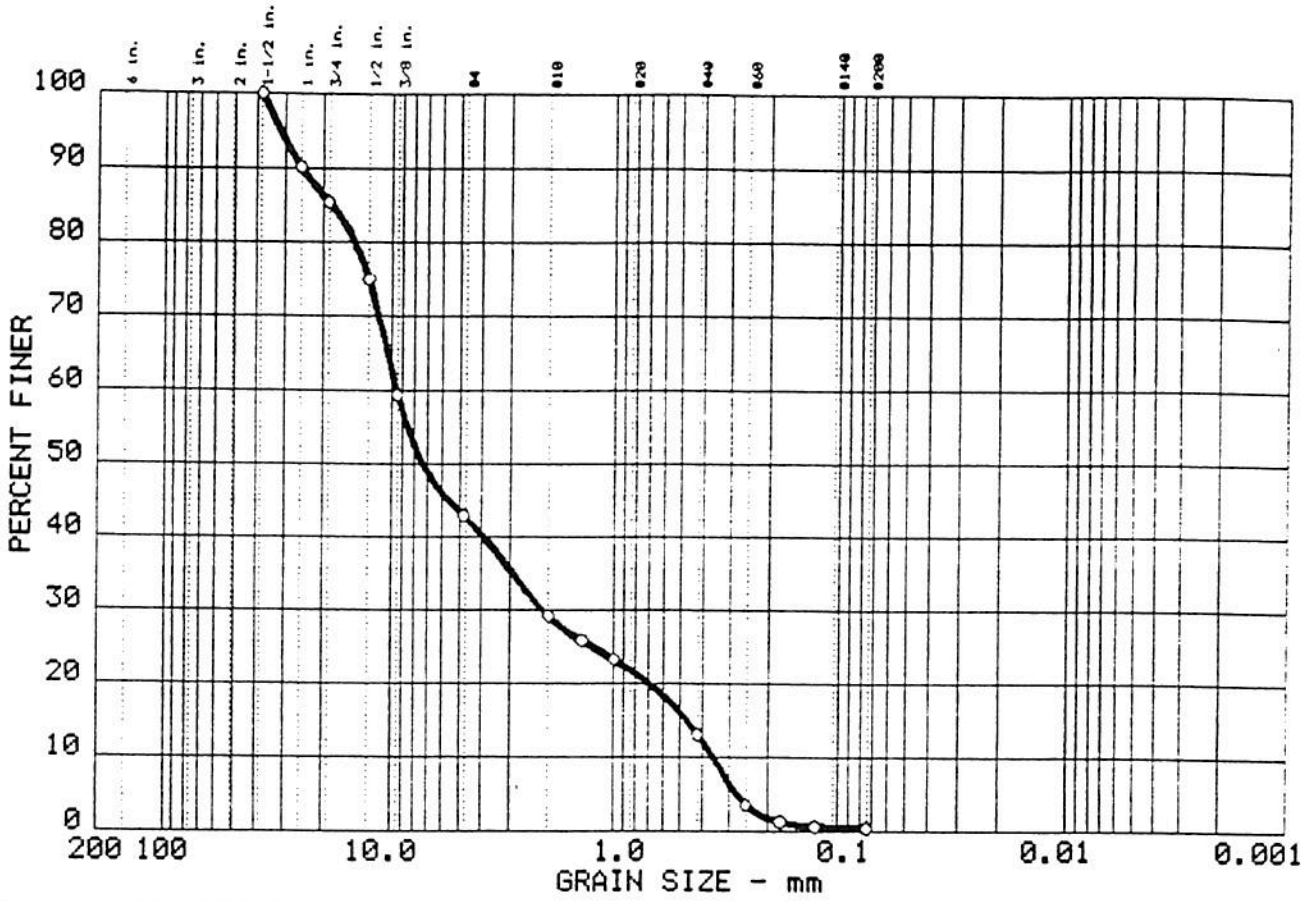
Date: 26NOV96

Remarks:
TECH JMC
BOR: DD-12T

GRAIN SIZE DISTRIBUTION TEST REPORT
USCOE, NEW ORLEANS

Figure No.02

GRAIN SIZE DISTRIBUTION TEST REPORT



	Test	%+75㎜	% GRAVEL	% SAND	% SILT	% CLAY
○	4	0.0	57.1	42.5	0.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			18.41	9.64	7.31	2.109	0.4613	0.3540	1.30	27.2

MATERIAL DESCRIPTION	USCS	AASHTO
○ GRAVEL POORLY GRADED (GP)		

Project No.: 96-24
Project: DARLINGTON RESERVOIR HWY 448
○ Location: BETWEEN DARLINGTON & GRAMERCY

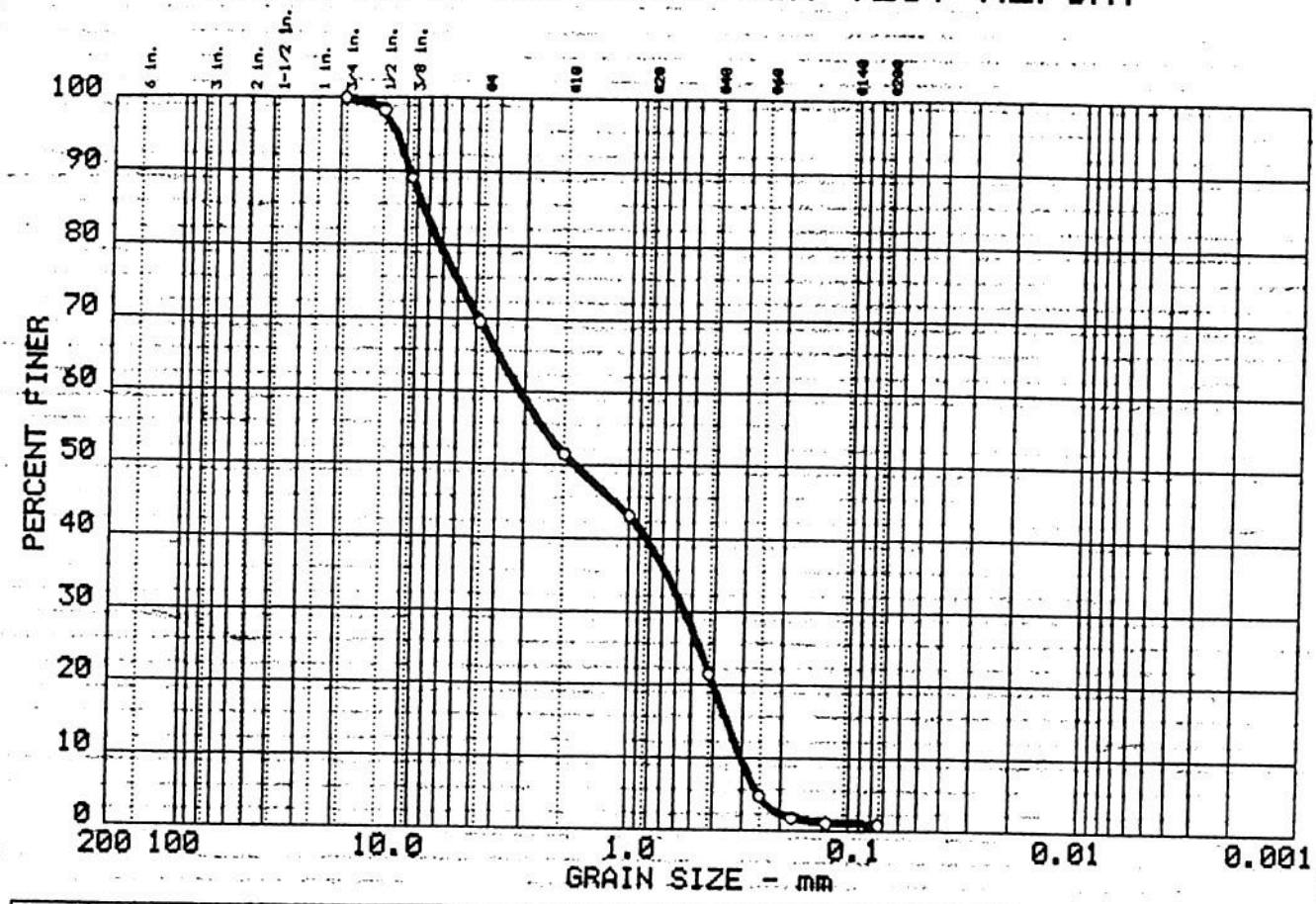
Date: 26NOV96

Remarks:
TECH JMC
BOR: DD-12T

GRAIN SIZE DISTRIBUTION TEST REPORT
USCOE, NEW ORLEANS

Figure No.03

GRAIN SIZE DISTRIBUTION TEST REPORT



	Test	%+75 mm	% GRAVEL	% SAND	% SILT	% CLAY
o	3	0.0	30.4	68.7	0.9	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o			8.46	3.14	1.77	0.546	0.3516	0.3027	0.31	10.4

MATERIAL DESCRIPTION	USCS	AASHTO
o SP,M,G,F,C.		

Project No.: 96-27
Project:
o Location:

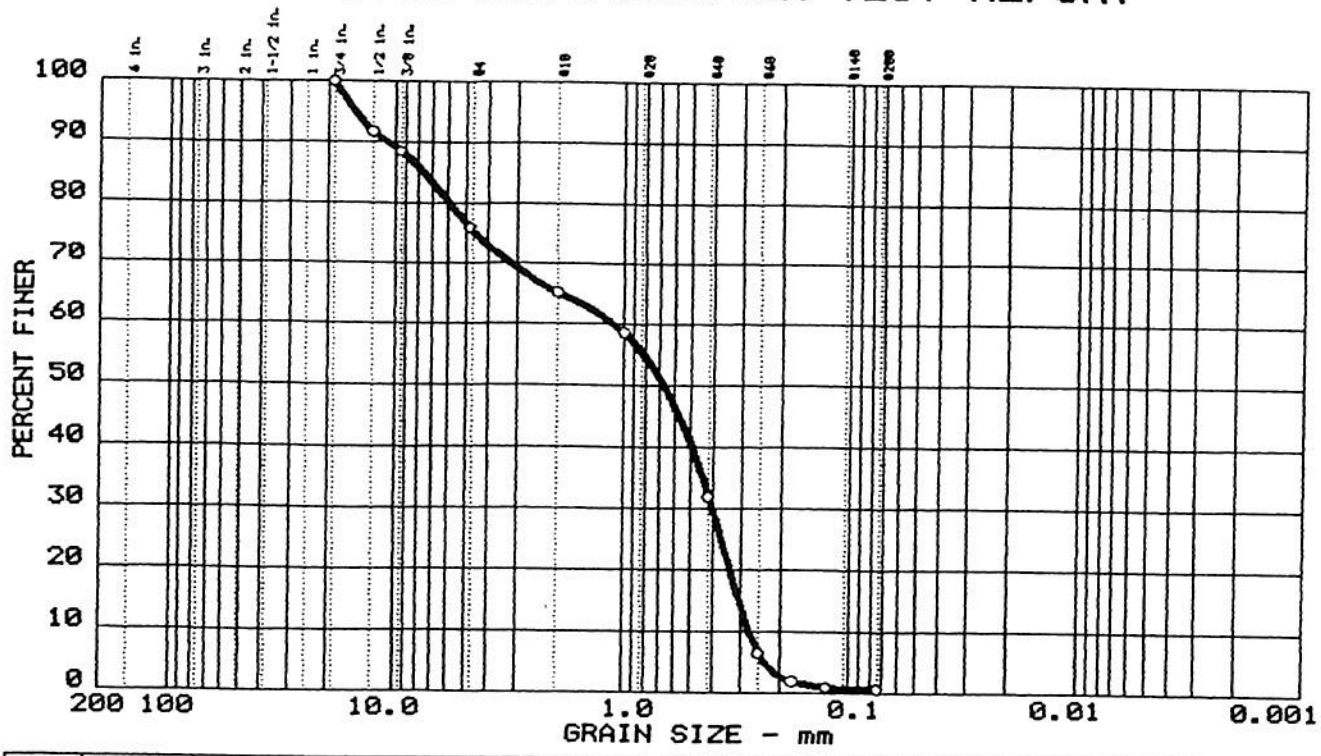
Date: 22NOV96

GRAIN SIZE DISTRIBUTION TEST REPORT
USCOE, NEW ORLEANS

Remarks:
PAN 37
CUP 88
TECH L.B.
SAMPLE # 4

Figure No.4

GRAIN SIZE DISTRIBUTION TEST REPORT



	Test	%+75mm	% GRAVEL	% SAND	% SILT	% CLAY
o	1	0.0	24.0	75.3	0.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o			7.59	1.11	0.66	0.405	0.3069	0.2757	0.54	4.0

MATERIAL DESCRIPTION	USCS	AASHTO
o SP,M,F,G,C		

Project No.: 96-24
Project: DD-12T
o Location:

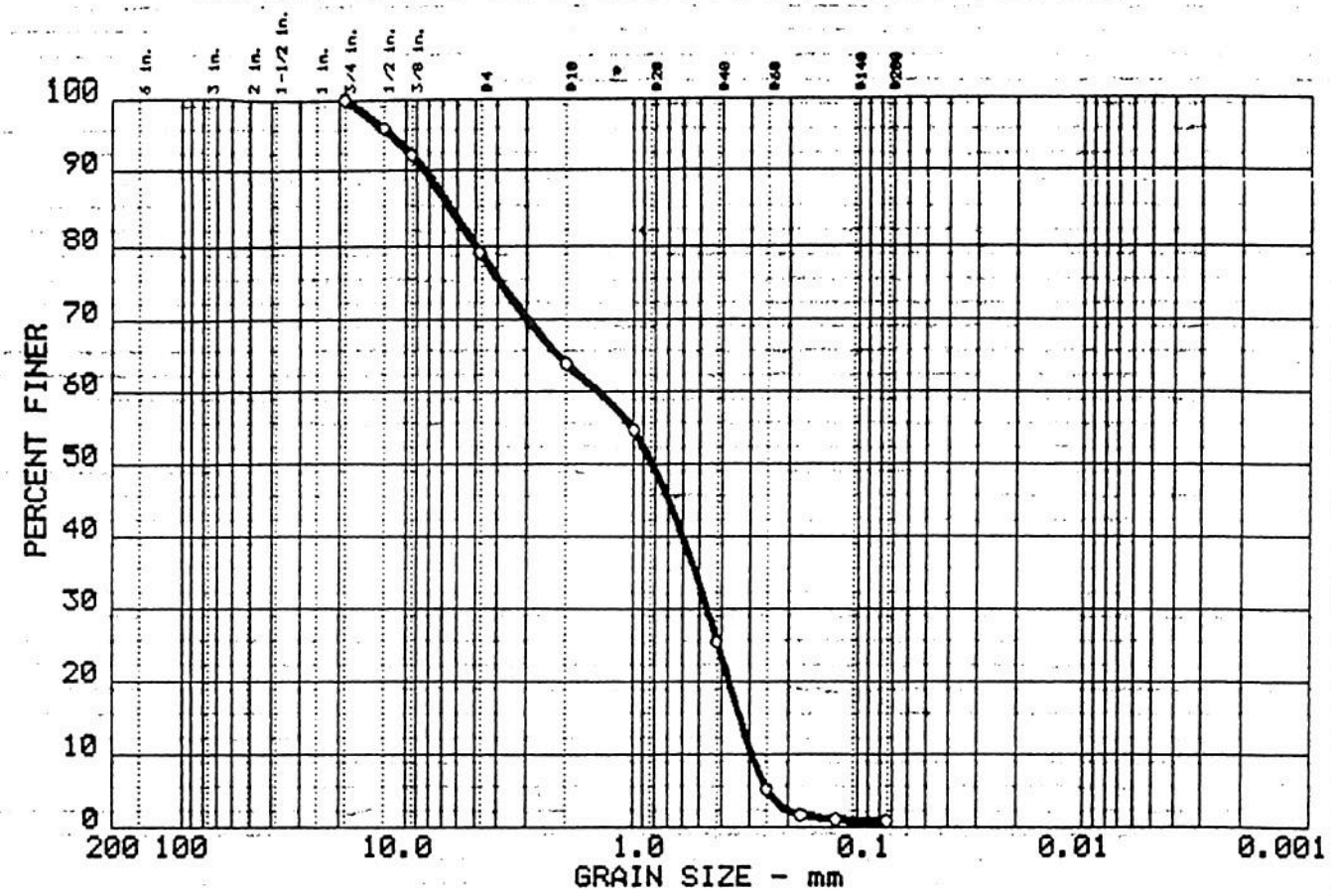
Date: 21NOV96

GRAIN SIZE DISTRIBUTION TEST REPORT
USCOE, NEW ORLEANS

Remarks:
PAN 0
CUP 00
TECH C.D.J.
96-24

Figure No.5

GRAIN SIZE DISTRIBUTION TEST REPORT



	Test	%+75	% GRAVEL	% SAND	% SILT	% CLAY
o	3	0.0	20.8	78.3	0.9	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o			6.31	1.41	0.82	0.465	0.3319	0.2924	0.52	4.8

MATERIAL DESCRIPTION	USCS	AASHTO
o SP-M,F,G,C,		

Project No.: 96-27
Project: STA 82+10
o Location:

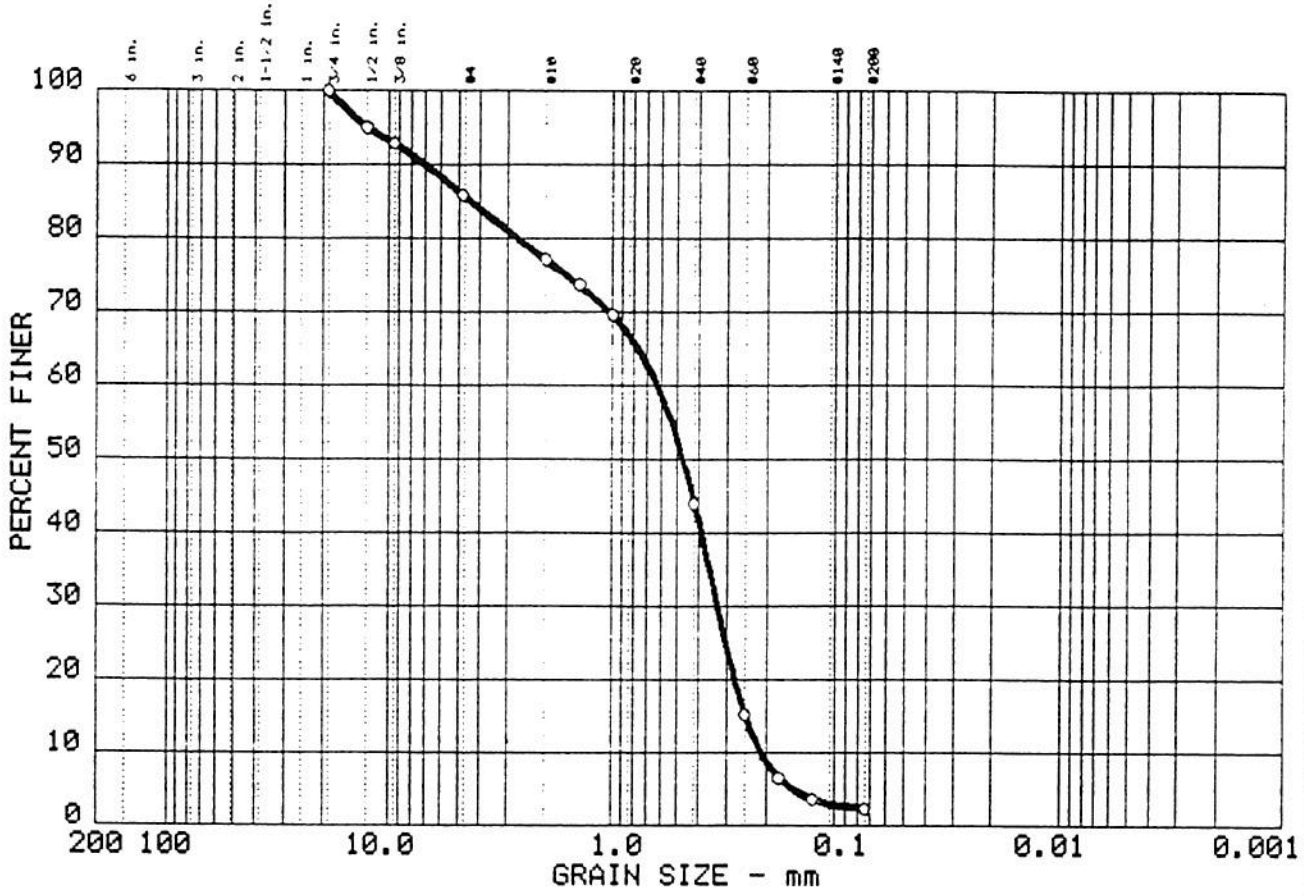
Date: 02DEC96

Remarks:
PAN 76
CUP 37
TECH L.B.
BOR.DD-12T

GRAIN SIZE DISTRIBUTION TEST REPORT
USCOE, NEW ORLEANS

Figure No.6

GRAIN SIZE DISTRIBUTION TEST REPORT



	Test	%+75㎜	% GRAVEL	% SAND	% SILT	% CLAY
o	7	0.0	14.1	83.5	2.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o			4.37	0.62	0.47	0.330	0.2486	0.2116	0.83	2.9

MATERIAL DESCRIPTION	USCS	AASHTO
o FINE SAND (SP)		

Project No.: 96-24
Project: DARLINGTON RESERVOIR HWY 448
o Location: BETWEEN DARLINGTON & GRAMERCY

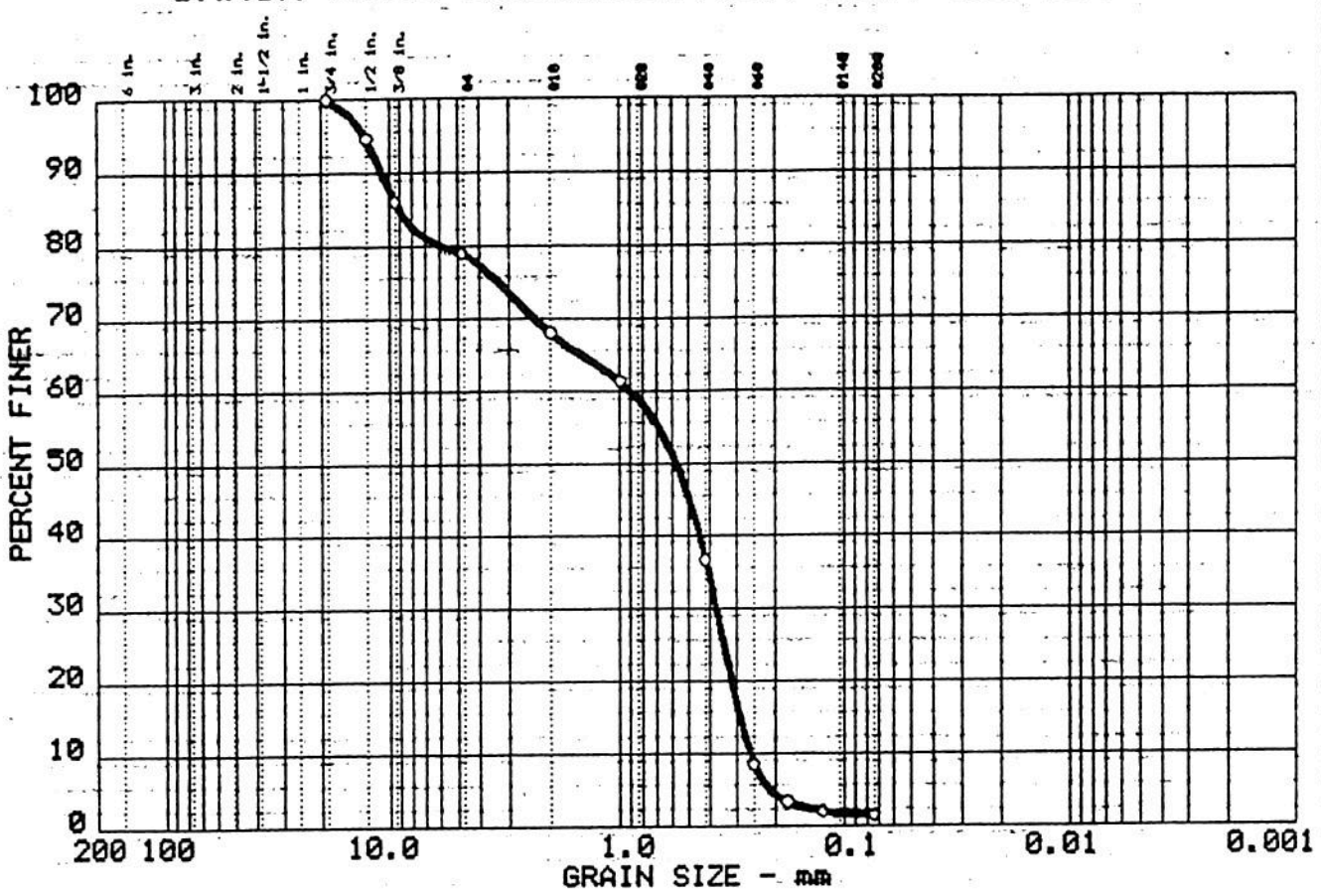
Date: 02DEC96

Remarks:
TECH JMC
BOR: DD-12T

GRAIN SIZE DISTRIBUTION TEST REPORT
USCOE, NEW ORLEANS

Figure No.07

GRAIN SIZE DISTRIBUTION TEST REPORT



	Test	%+75 mm	% GRAVEL	% SAND	% SILT	% CLAY
o	3	0.0	21.1	77.4	1.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o			9.12	0.89	0.57	0.373	0.2891	0.2591	0.60	3.4

	MATERIAL DESCRIPTION	USCS	AASHTO
o	SP		

Project No.: 96-24
Project: DDT-12T
o Location:

Date: 14NOV96

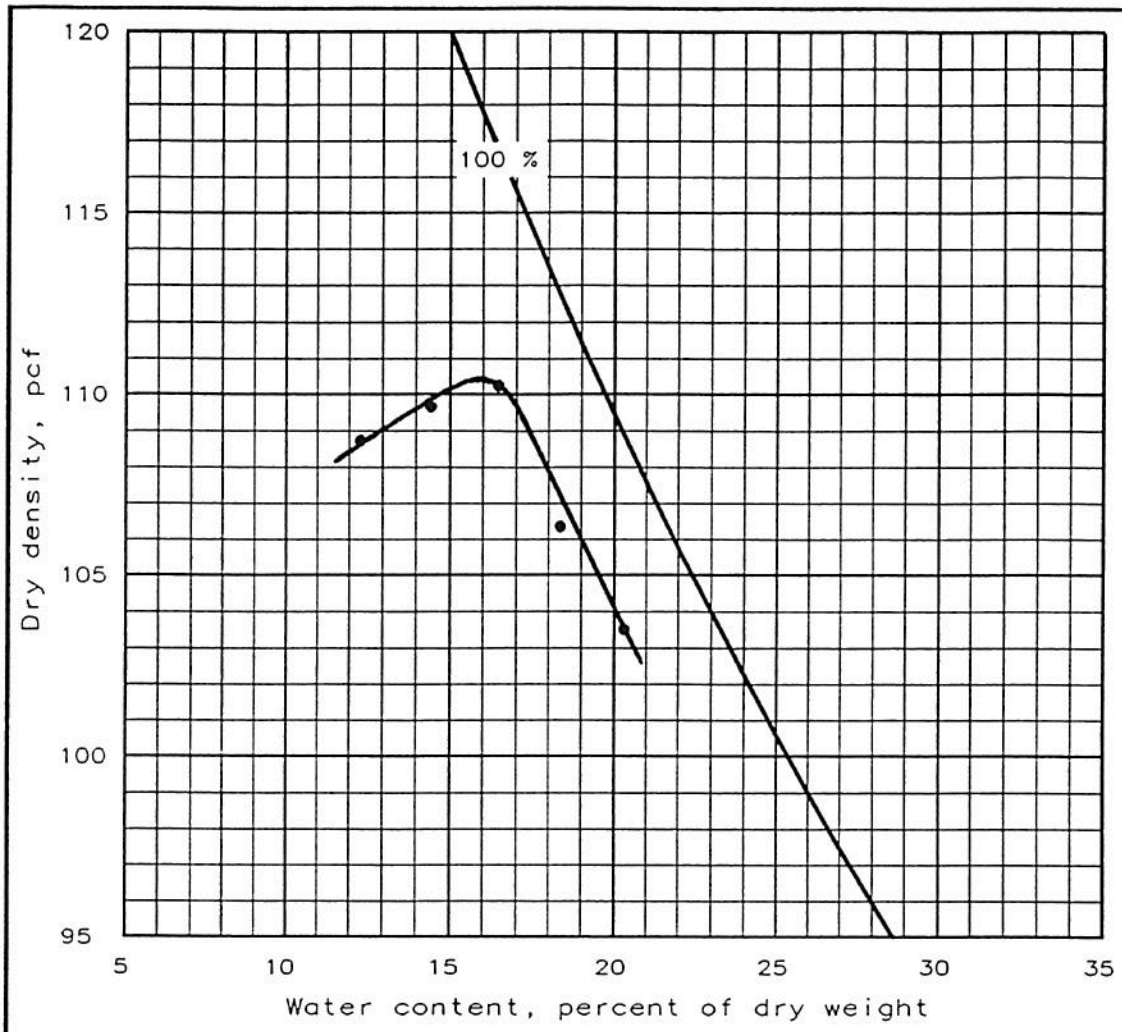
Remarks:
PAN 12
CUP 32
TECH L.B.
SAM 8

GRAIN SIZE DISTRIBUTION TEST REPORT
USCOE, NEW ORLEANS

Figure No. 8

DEPARTMENT OF THE ARMY, WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS, 3909 HALLS FERRY ROAD
VICKSBURG, MS 39180

WORK ORDER NO. 0897
Req. No.
Contract No.



Standard compaction test EM-1110-2-1906
25 blows per each of 3 layers, with 5.50 lb. sleeve rammer
and 12.0 inch drop. 4 inch diameter mold

Sample No.	Elev/ Depth	Classification	G	LL	PL	% > No.4	% > 3/4 in.
6		CLAY (CL), BROWN	2.70	35	20		
		BROWN					

Sample No.	6
Water content, percent	
Optimum water content, percent	15.8
Max. dry density, pcf	110.4

Remarks:

Project: DARLINGTON RESERVOIR

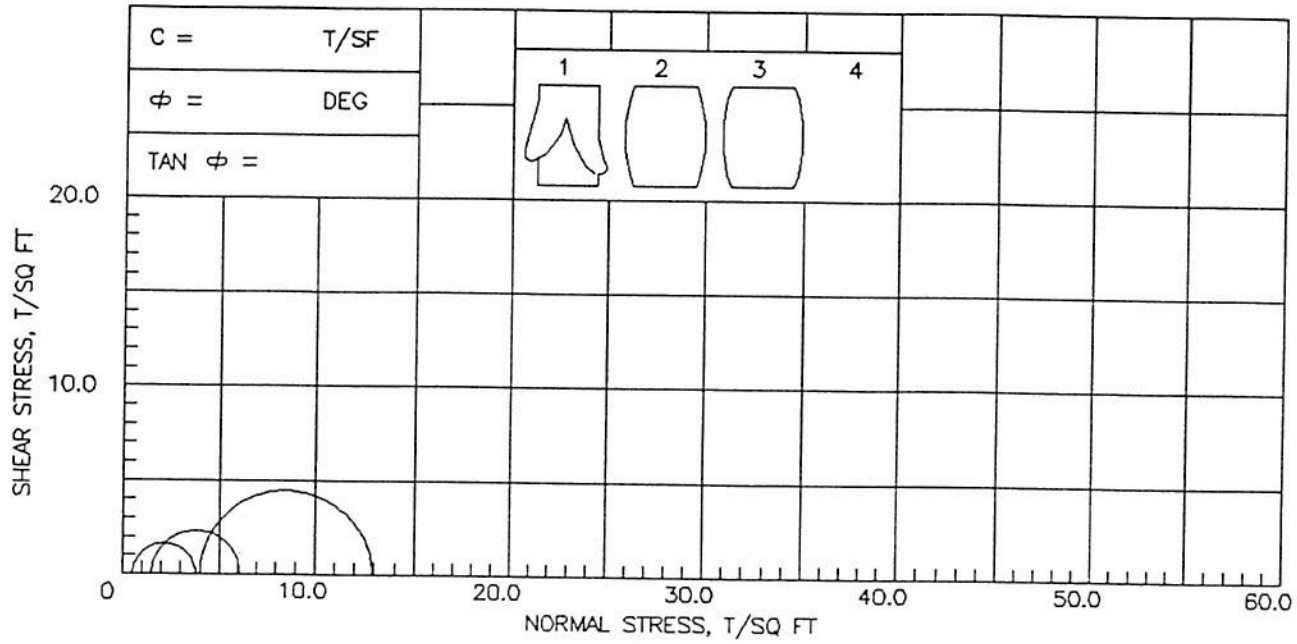
Lab No.: STF/GL

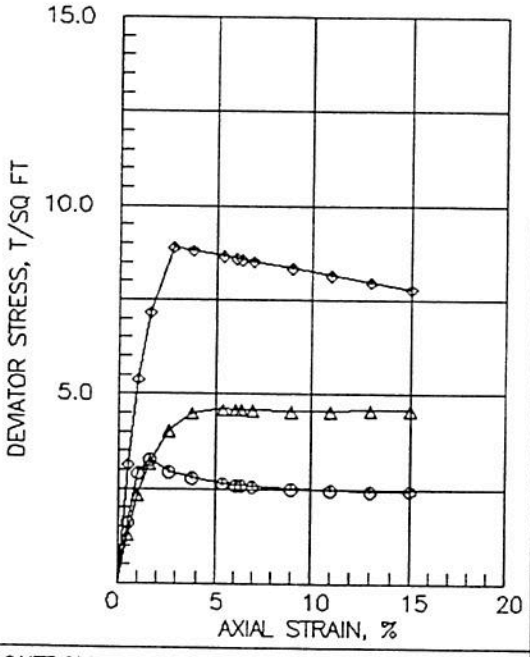
Area:

Boring No.: DD-13T

Date: 3-17-97

COMPACTION TEST REPORT





SPECIMEN NO.		○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	7.3	7.3	7.3	
	DRY DENSITY, PCF	101.9	99.2	102.0	
	SATURATION, %	30.1	28.1	30.2	
	VOID RATIO	.654	.699	.652	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		.50	1.50	4.00	
MAX. DEV. STRESS, TSF		3.28	4.61	8.91	
TIME TO FAILURE, MIN.		4	24	17	
STRAIN RATE INCR., %			7	6	
INITIAL DIAMETER, IN.		1.37	1.37	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CL), BROWN

95 % MAX. DRY DENS. & DRY OF OPT. WC

LL 35	PL 20	PI 15	GS 2.70 (ESTIMATED)	COMPACTED SPECIMEN	Q TEST

REMARKS:

MOLDING CONDITIONS:

DRY DENSITY = 104.8 PCF

WATER CONTENT = 7.0 %

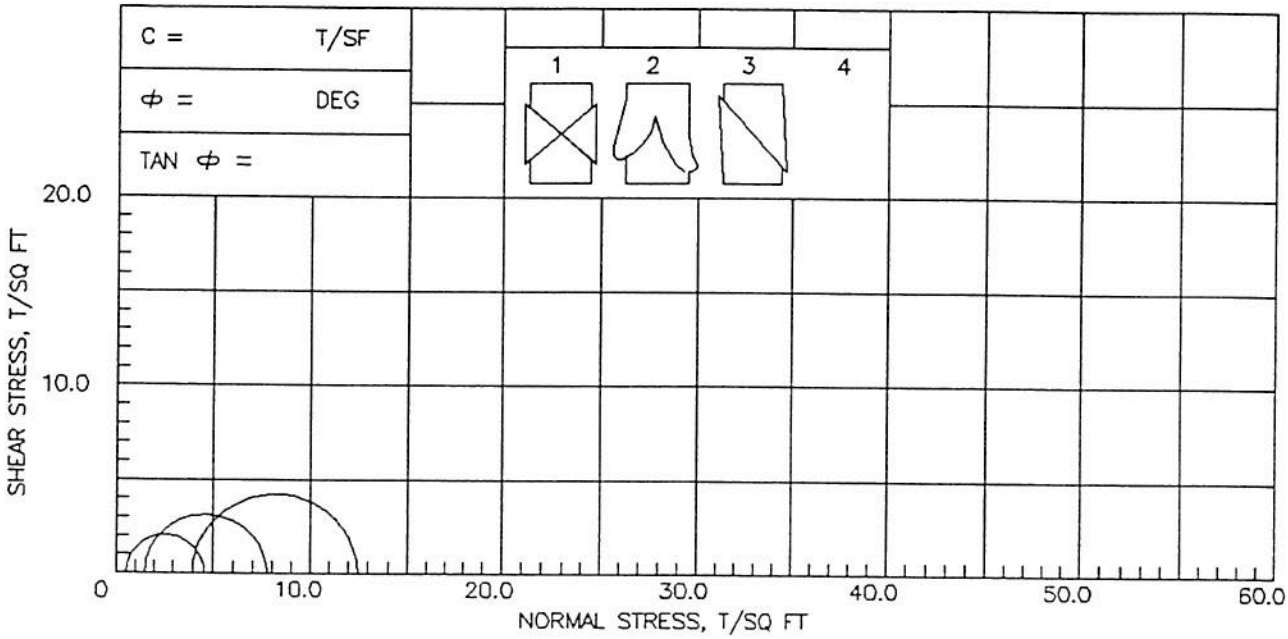
PROJECT DARLINGTON RESERVOIR

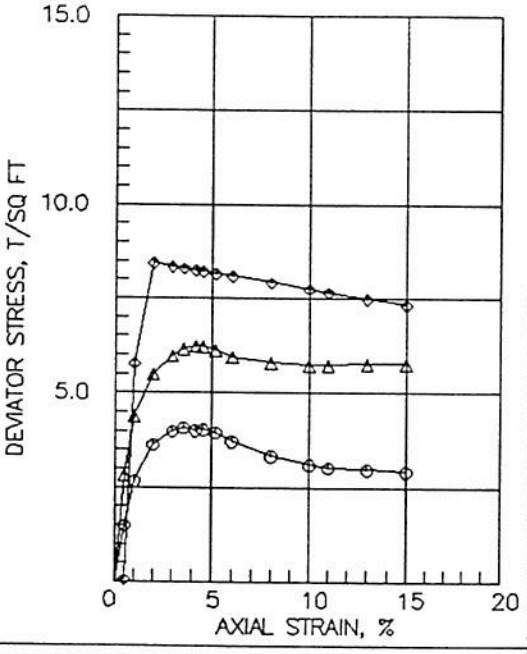
BORING NO. DD-13T SAMPLE NO. 6

DEPTH/ELEV 8-FT TECH. US

LABORATORY USAE WES - STF/GL DATE 26 MAR 97

TRIAXIAL COMPRESSION TEST REPORT





SPECIMEN NO.		○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	15.1	15.1	15.1	
	DRY DENSITY, PCF	109.0	109.3	109.3	
	SATURATION, %	74.4	75.0	75.0	
	VOID RATIO	.546	.542	.542	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		.50	1.50	4.00	
MAX. DEV. STRESS, TSF		4.09	6.23	8.45	
TIME TO FAILURE, MIN.		9	20	6	
STRAIN RATE INCR., %			10	6	
INITIAL DIAMETER, IN.		1.36	1.36	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CL), BROWN

MAX. DRY DENS. & OPT. WC

LL 35	PL 20	PI 15	GS 2.70 (ESTIMATED)	COMPACTED SPECIMEN	Q TEST

REMARKS:

MOLDING CONDITIONS:

DRY DENSITY = 110.3 PCF

WATER CONTENT = 15.6 %

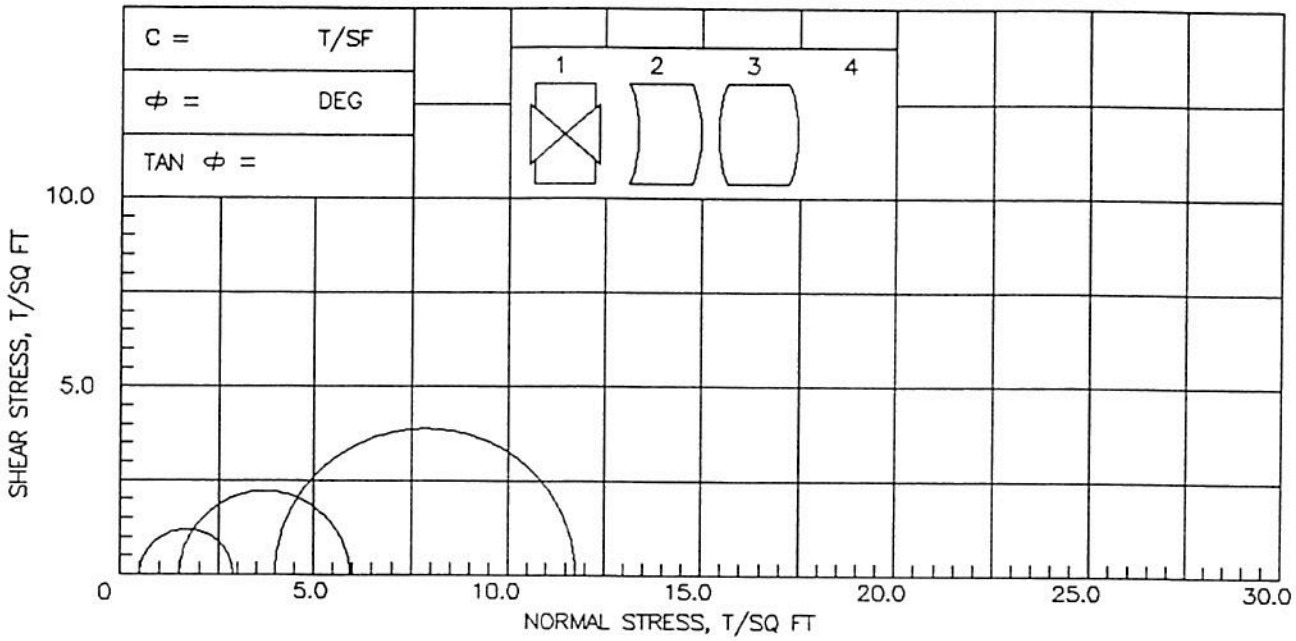
PROJECT DARLINGTON RESERVOIR

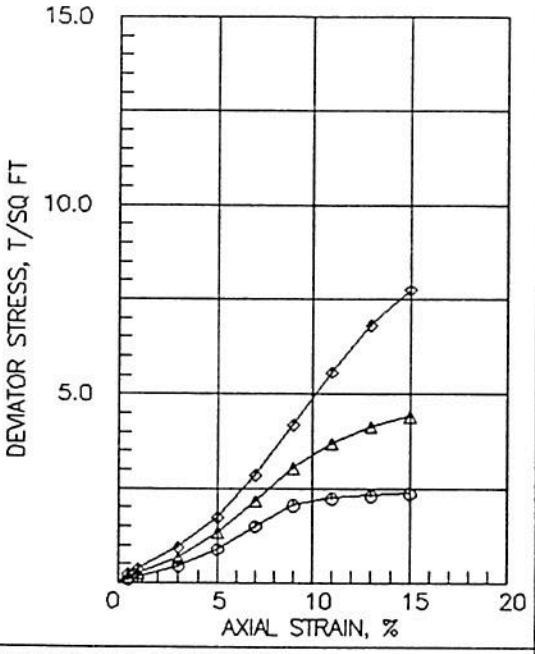
BORING NO. DD-13T | SAMPLE NO. 6

DEPTH/ELEV 8-FT | TECH. US

LABORATORY USAE WES - STF/GL | DATE 26 MAR 97

TRIAXIAL COMPRESSION TEST REPORT





SPECIMEN NO.		◇ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	19.0	19.1	19.1	
	DRY DENSITY, PCF	104.1	103.7	104.9	
	SATURATION, %	83.0	82.6	84.8	
	VOID RATIO	.619	.625	.607	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		.50	1.50	4.00	
MAX. DEV. STRESS, TSF		2.38	4.43	7.77	
TIME TO FAILURE, MIN.		37	36	37	
STRAIN RATE INCR., %					
INITIAL DIAMETER, IN.		1.37	1.36	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CL), BROWN

95 % MAX. DRY DENS. & WET OF OPT. WC

LL 35	PL 20	PI 15	GS 2.70 (ESTIMATED)	COMPACTED SPECIMEN	Q TEST

REMARKS:

MOLDING CONDITIONS:

DRY DENSITY = 104.8 PCF

WATER CONTENT = 19.5 %

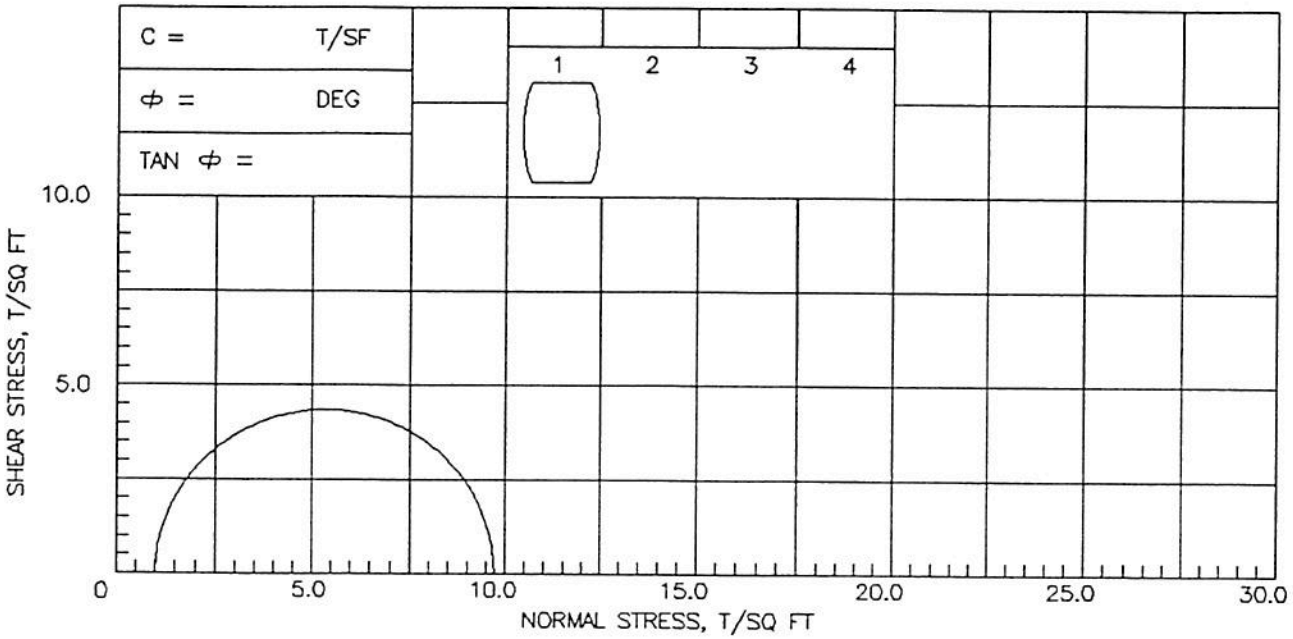
PROJECT DARLINGTON RESERVOIR

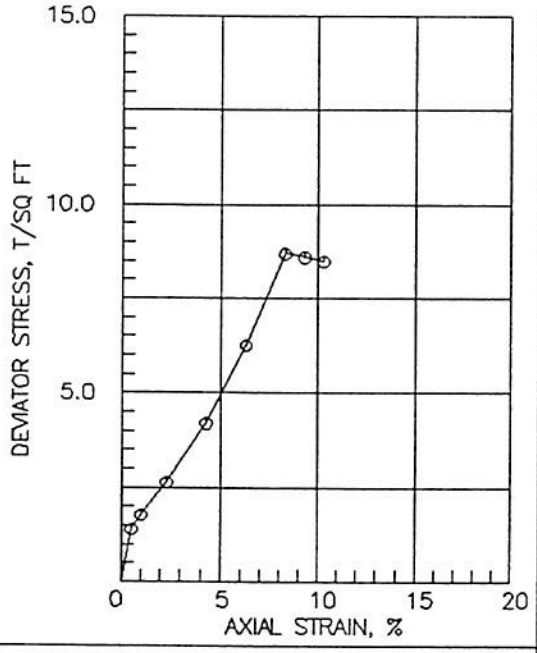
BORING NO. DD-13T | SAMPLE NO. 6

DEPTH/ELEV 8-FT | TECH. US

LABORATORY USAE WES - STF/GL | DATE 27 MAR 97

TRIAXIAL COMPRESSION TEST REPORT





SPECIMEN NO.		○ 1	2	3	4
INITIAL	WATER CONTENT, %	19.4			
	DRY DENSITY, PCF	106.0			
	SATURATION, %	88.8			
	VOID RATIO	.590			
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		1.00			
MAX. DEV. STRESS, TSF		8.70			
TIME TO FAILURE, MIN.		21			
STRAIN RATE INCR., %					
INITIAL DIAMETER, IN.		1.37			
INITIAL HEIGHT, IN.		3.00			

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CL), BROWN

LL 35	PL 20	PI 15	GS 2.70 (ESTIMATED)	COMPACTED SPECIMEN	Q TEST

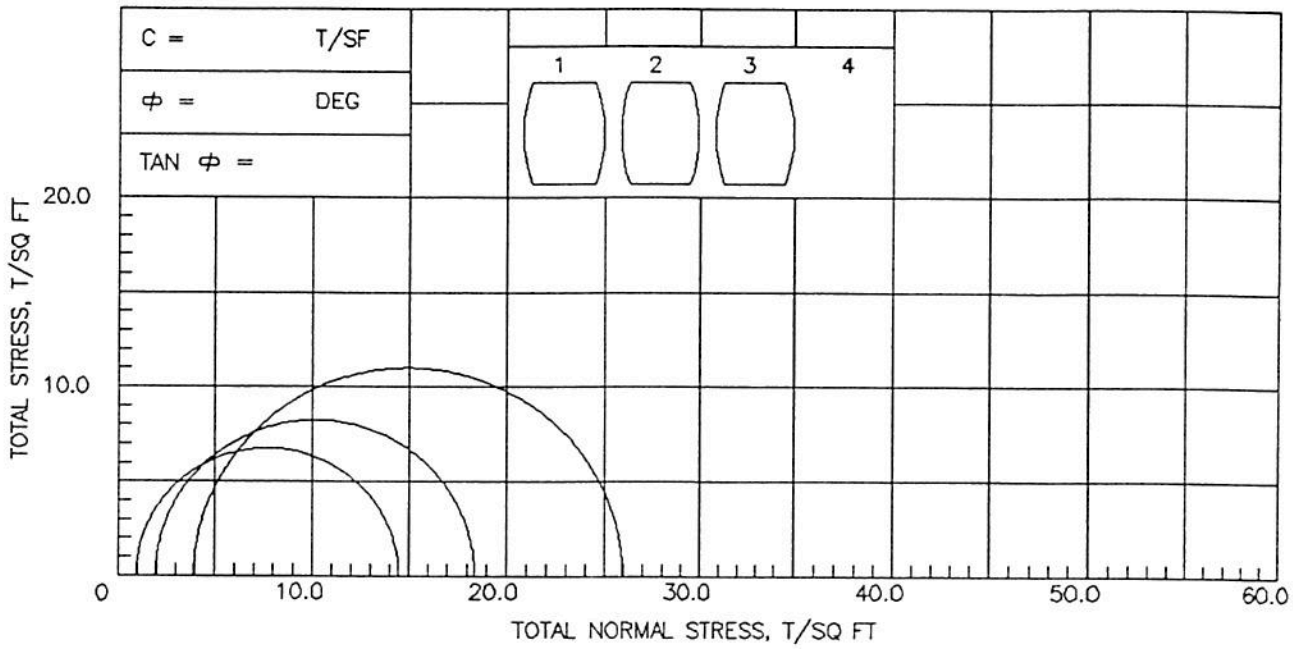
REMARKS: CONSOLIDATED AT 3 TSF,
UNLOADED TO 1 TSF, AND SATURATED;

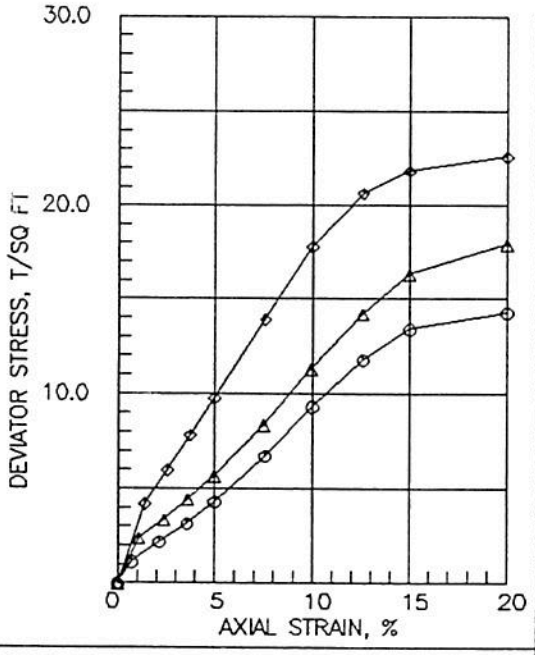
MOLDING CONDITIONS:
DRY DENSITY = 104.8 PCF
WATER CONTENT = 19.5 %

PROJECT DARLINGTON RESERVOIR

BORING NO. DD-13T | SAMPLE NO. 6
DEPTH/ELEV 8-FT | TECH. US
LABORATORY USAE WES - STF/GL | DATE 18 APR 97

TRIAXIAL COMPRESSION TEST REPORT





SPECIMEN NO.		○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	15.5	15.6	15.6	
	DRY DENSITY, PCF	109.4	108.5	109.7	
	SATURATION, %	77.3	76.0	78.4	
	VOID RATIO	.541	.554	.536	
BEFORE SHEAR	WATER CONTENT, %	19.8	19.5	19.0	
	DRY DENSITY, PCF	111.5	111.2	113.9	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	.512	.516	.480	
	BACK PRESS., TSF	5.04	5.04	4.32	
MIN. PRIN. STRESS, TSF		1.00	2.00	4.00	
MAX. DEV. STRESS, TSF		13.41	16.36	21.88	
TIME TO FAILURE, MIN.		1020	1022	1022	
RATE OF STRAIN, %/MIN		.015	.015	.015	
INITIAL DIAMETER, IN.		1.37	1.37	1.37	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CL), BROWN

MAX. DRY DENS. & OPT. WC

LL	PL	PI	GS 2.70 (ESTIMATED)	COMPACTED SPECIMEN	$\bar{R}$ TEST

REMARKS:

MOLDING CONDITIONS:

DRY DENSITY = 110.3 PCF

WATER CONTENT = 15.6 %

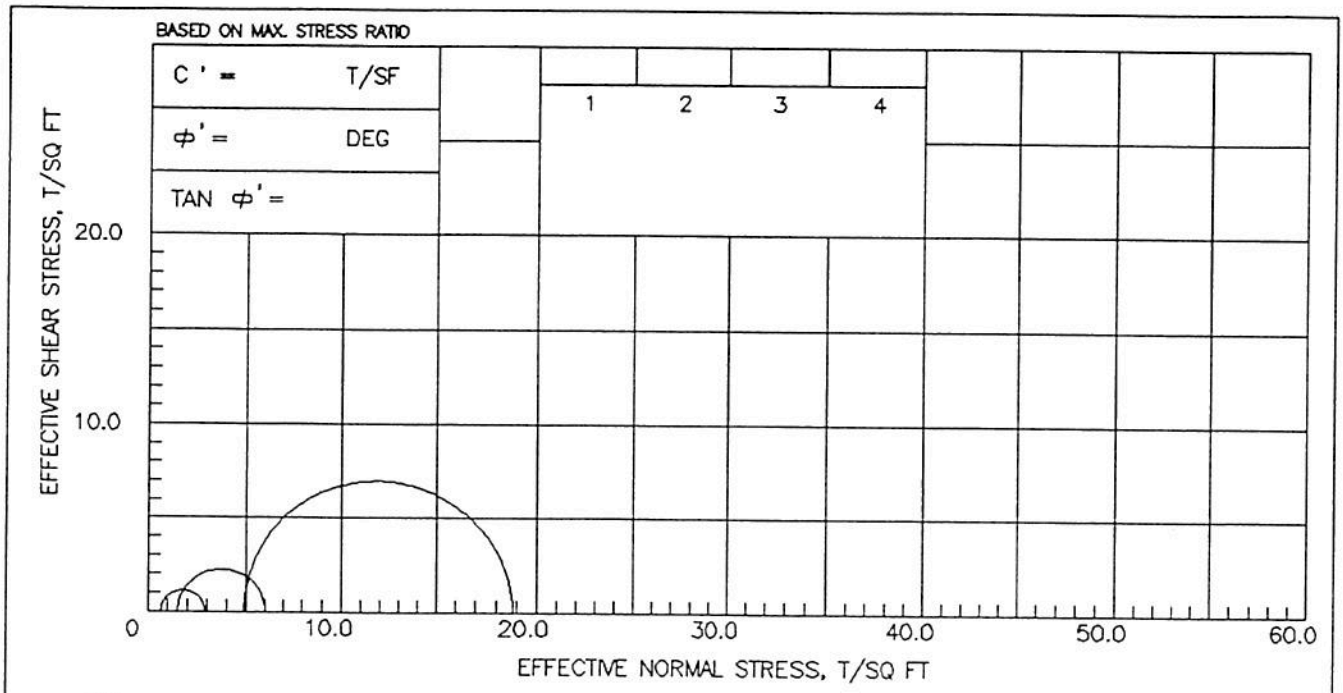
PROJECT DARLINGTON RESERVOIR

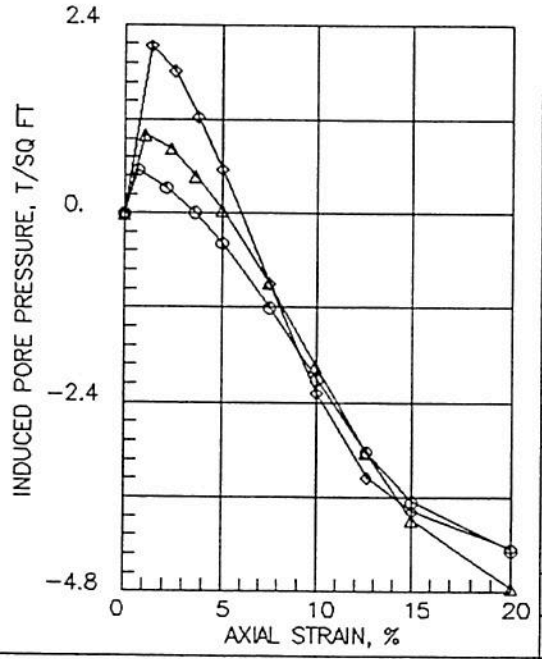
BORING NO. DD-13T | SAMPLE NO. 6

DEPTH/ELEV 8-FT | TECH. US

LABORATORY USAE WES - STF/GL | DATE 04 APR 97

$\bar{R}$-TRIAXIAL COMPRESSION TEST REPORT





SPECIMEN NO.		○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	15.5	15.6	15.6	
	DRY DENSITY, PCF	109.4	108.5	109.7	
	SATURATION, %	77.3	76.0	78.4	
	VOID RATIO	.541	.554	.536	
BEFORE SHEAR	WATER CONTENT, %	19.8	19.5	19.0	
	DRY DENSITY, PCF	111.5	111.2	113.9	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	.512	.516	.480	
	BACK PRESS., TSF	5.04	5.04	4.32	
@ σ'1/σ'3 MAX	MIN PRIN EFF STRESS, TSF	.67	1.52	4.89	
	DEV. STRESS, TSF	2.22	4.46	13.94	
	TIME TO FAILURE, MIN.	146	248	515	
RATE OF STRAIN, %/MIN		.015	.015	.015	
INITIAL DIAMETER, IN.		1.37	1.37	1.37	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED–STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CL), BROWN

MAX. DRY DENS. & OPT. WC

LL	PL	PI	GS 2.70 (ESTIMATED)	COMPACTED SPECIMEN	$\bar{R}$ TEST

REMARKS:

MOLDING CONDITIONS:

DRY DENSITY = 110.3 PCF

WATER CONTENT = 15.6 %

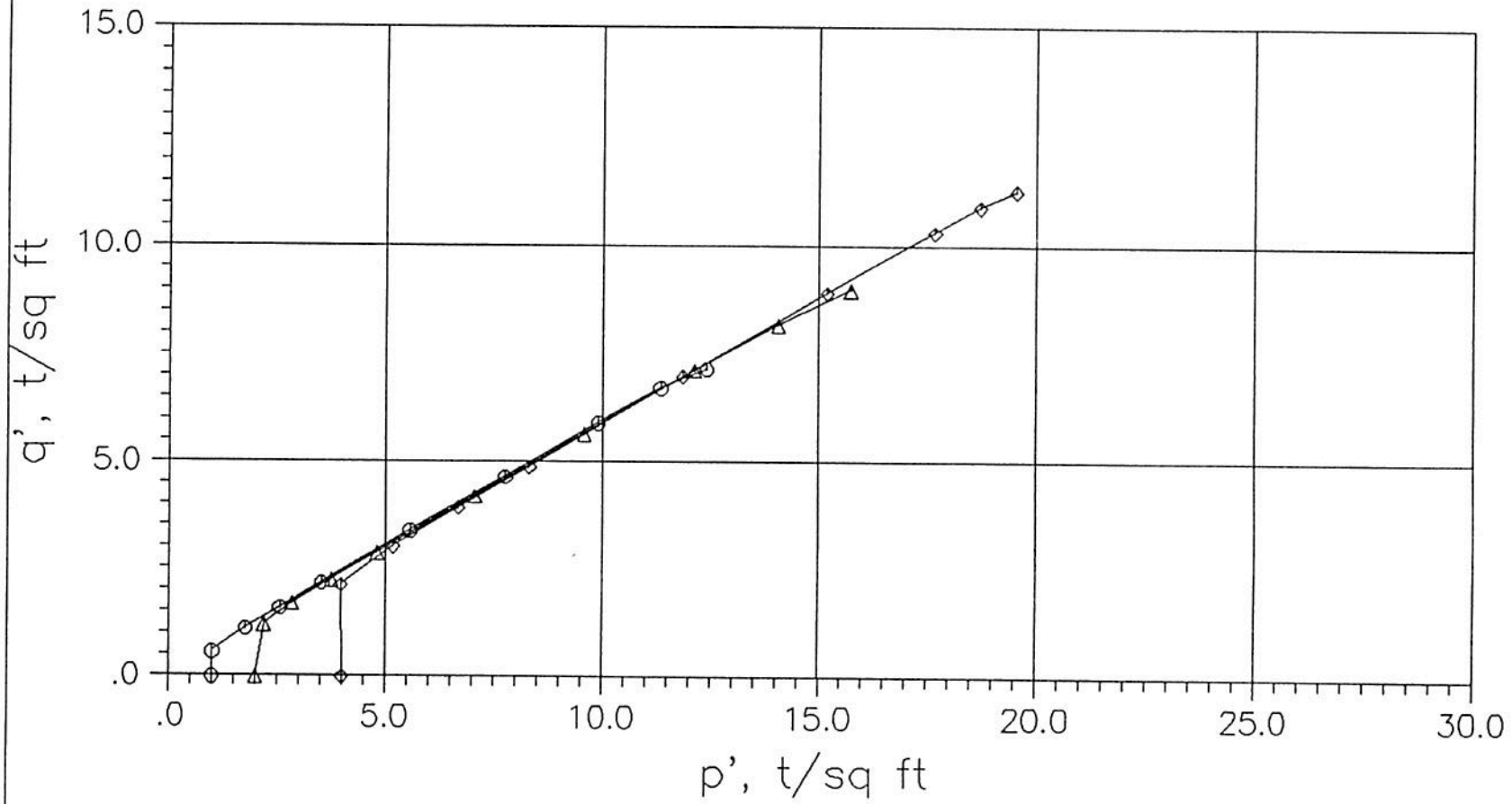
PROJECT DARLINGTON RESERVOIR

BORING NO. DD–13T SAMPLE NO. 6

DEPTH/ELEV 8–FT TECH. US

LABORATORY USAE WES – STF/GL DATE 04 APR 97

$\bar{R}$–TRIAXIAL COMPRESSION TEST REPORT



PROJECT DARLINGTON RESERVOIR	
BORING DD-13T	SAMPLE NO. 6
DEPTH/ELEV 8-FT	DATE 04 APR 97

Stress Paths

LABORATORY USAE WES - STF/GL

R-BAR TRIAXIAL COMPRESSION TEST REPORT

TLE: DARLINGTON RESERVOIR

BOR: DD-13T SAM: 6 DEP: 8-FT DAT: 04 APR 97 TEC: US
DES: CLAY (CL), BROWN
MAX. DRY DENS. & OPT. WC

REM:
MOLDING CONDITIONS:
DRY DENSITY = 110.3 PCF
WATER CONTENT = 15.6 %
SAMPLE TYPE = COMPACTED
GS: 2.70 (ESTIMATED)
LL: 0 PL: 0 PI: 0

WEIGHT OF SOLIDS = 126.98 I. HEIGHT = 3.000" I. DIAMETER = 1.370"

HEIGHT CHANGES: PRE-CONS. = .01200" DURING CONS. = .01000"

VOLUME CHANGE DURING CONS. = .5 CC TIME OF TEST = 1351 MINS.

BACK PRESSURE = 70.00 PSI (5.04 TSF) CHAMBER PRESSURE = 83.89

EFF CONFINING PRESSURE, TSF = 1.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	15.49	19.85
VOID RATIO	.541	.512
SATURATION, %	77.33	104.73
DRY DENSITY, PCF	109.40	111.50

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	EFFECTIVE E3 - TSF -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	70.0	.0	.00	.00	1.000	1.000	1.000	1.00	.00
.021	23.3	77.8	.7	.56	1.14	1.582	.438	3.607	1.01	.57
.064	46.0	74.6	2.1	.33	2.22	2.893	.669	4.325	1.78	1.11
.107	66.6	70.2	3.6	.01	3.17	4.158	.986	4.219	2.57	1.59
.149	92.1	64.8	5.0	-.37	4.32	5.698	1.374	4.146	3.54	2.16
.225	147.8	53.4	7.6	-1.20	6.75	8.947	2.195	4.076	5.57	3.38
.298	209.7	40.8	10.0	-2.10	9.33	12.427	3.102	4.006	7.76	4.66
.375	273.3	28.2	12.6	-3.01	11.80	15.813	4.009	3.944	9.91	5.90
.447	319.3	19.3	15.0	-3.65	13.41	18.059	4.650	3.884	11.35	6.70
.596	361.6	10.8	20.0	-4.26	14.29	19.554	5.262	3.716	12.41	7.15

RATE OF STRAIN = .015

R-BAR TRIAXIAL COMPRESSION TEST REPORT

TLE: DARLINGTON RESERVOIR

BOR: DD-13T SAM: 6 DEP: 8-FT DAT: 04 APR 97 TEC: US
DES: CLAY (CL), BROWN
MAX. DRY DENS. & OPT. WC

REM:
MOLDING CONDITIONS:
DRY DENSITY = 110.3 PCF
WATER CONTENT = 15.6 %
SAMPLE TYPE = COMPACTED
GS: 2.70 (ESTIMATED)
LL: 0 PL: 0 PI: 0

WEIGHT OF SOLIDS = 126.25 I. HEIGHT = 3.000" I. DIAMETER = 1.372"

HEIGHT CHANGES: PRE-CONS. = .01200" DURING CONS. = .02300"

VOLUME CHANGE DURING CONS. = .9 CC TIME OF TEST = 1347 MINS.

BACK PRESSURE = 70.00 PSI (5.04 TSF) CHAMBER PRESSURE = 97.78

EFF CONFINING PRESSURE, TSF = 2.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	15.60	19.49
VOID RATIO	.554	.516
SATURATION, %	75.99	101.90
DRY DENSITY, PCF	108.45	111.16

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	EFFECTIVE E3 - TSF -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	70.0	.0	.00	.00	2.000	2.000	1.000	2.00	.00
.032	49.4	84.0	1.1	1.01	2.41	3.403	.992	3.430	2.20	1.21
.070	70.1	81.6	2.4	.84	3.38	4.541	1.165	3.898	2.85	1.69
.108	93.8	76.6	3.6	.48	4.46	5.983	1.525	3.924	3.75	2.23
.148	121.8	70.6	5.0	.04	5.71	7.665	1.957	3.917	4.81	2.85
.222	183.6	57.8	7.5	-.88	8.38	11.257	2.878	3.911	7.07	4.19
.296	254.5	43.2	10.0	-1.93	11.30	15.231	3.929	3.876	9.58	5.65
.372	329.8	28.1	12.5	-3.02	14.23	19.244	5.017	3.836	12.13	7.11
.445	390.3	16.1	15.0	-3.88	16.36	22.244	5.880	3.783	14.06	8.18
.593	455.3	4.1	20.0	-4.74	17.97	24.712	6.744	3.664	15.73	8.98

RATE OF STRAIN = .015

R-BAR TRIAXIAL COMPRESSION TEST REPORT

`LE: DARLINGTON RESERVOIR

BOR: DD-13T SAM: 6 DEP: 8-FT DAT: 04 APR 97 TEC: US
DES: CLAY (CL), BROWN
MAX. DRY DENS. & OPT. WC

REM:
MOLDING CONDITIONS:
DRY DENSITY = 110.3 PCF
WATER CONTENT = 15.6 %
SAMPLE TYPE = COMPACTED
GS: 2.70 (ESTIMATED)
LL: 0 PL: 0 PI: 0

WEIGHT OF SOLIDS = 126.64 I. HEIGHT = 3.000" I. DIAMETER = 1.366"

HEIGHT CHANGES: PRE-CONS. = .01400" DURING CONS. = .02600"

VOLUME CHANGE DURING CONS. = 1.6 CC TIME OF TEST = 1345 MINS.

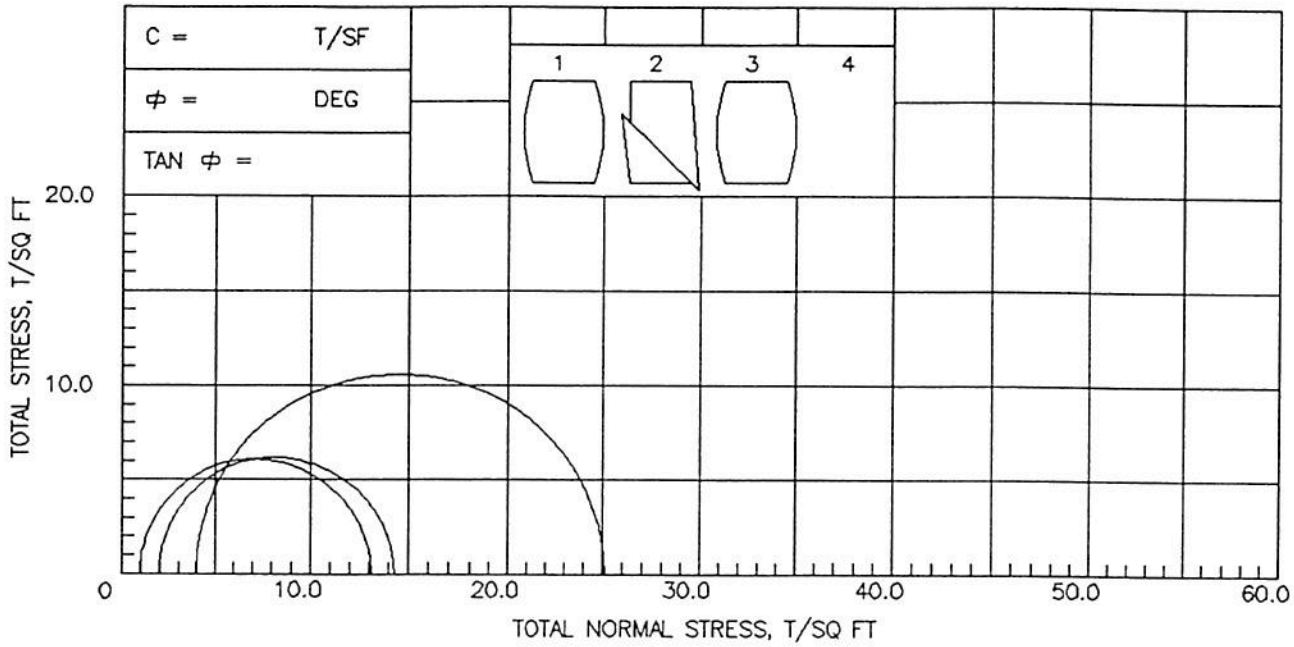
BACK PRESSURE = 60.00 PSI (4.32 TSF) CHAMBER PRESSURE = 115.56

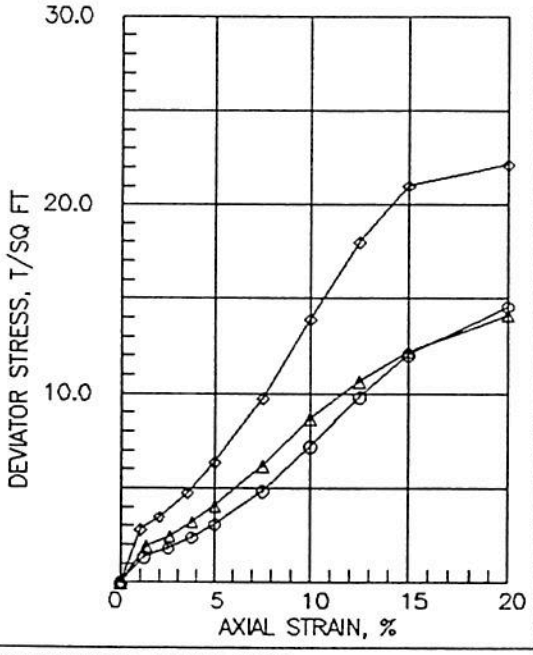
EFF CONFINING PRESSURE, TSF = 4.00

	INITIAL	BEFORE SHEAR
,ATER CONTENT, %	15.56	19.05
VOID RATIO	.536	.480
SATURATION, %	78.39	107.07
DRY DENSITY, PCF	109.74	113.86

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	EFFECTIVE E3 - TSF -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	60.0	.0	.00	.00	4.000	4.000	1.000	4.00	.00
.041	85.2	89.7	1.4	2.14	4.23	6.087	1.862	3.269	3.97	2.11
.076	122.5	85.3	2.6	1.82	6.00	8.181	2.179	3.755	5.18	3.00
.111	161.9	77.2	3.8	1.24	7.84	10.598	2.762	3.838	6.68	3.92
.148	204.5	67.9	5.0	.57	9.77	13.201	3.431	3.847	8.32	4.88
.224	300.0	47.7	7.6	-.89	13.94	18.830	4.886	3.854	11.86	6.97
.296	394.0	28.4	10.0	-2.28	17.83	24.107	6.275	3.842	15.19	8.92
.373	470.3	13.6	12.6	-3.34	20.67	28.011	7.341	3.816	17.68	10.34
.444	511.8	7.6	15.0	-3.77	21.88	29.650	7.772	3.815	18.71	10.94
.592	561.9	1.1	20.0	-4.24	22.61	30.846	8.240	3.743	19.54	11.30

RATE OF STRAIN = .015





SPECIMEN NO.		⊙ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	19.2	19.1	19.3	
	DRY DENSITY, PCF	105.0	105.0	104.5	
	SATURATION, %	85.7	85.3	85.1	
	VOID RATIO	.605	.605	.613	
BEFORE SHEAR	WATER CONTENT, %	19.6	19.3	19.3	
	DRY DENSITY, PCF	106.5	107.1	107.7	
	SATURATION, %	91.1	90.9	92.4	
	VOID RATIO	.582	.573	.565	
	BACK PRESS., TSF	5.04	5.04	5.04	
MIN. PRIN. STRESS, TSF		1.00	2.00	4.00	
MAX. DEV. STRESS, TSF		12.06	12.23	21.04	
TIME TO FAILURE, MIN.		1012	1010	1013	
RATE OF STRAIN, %/MIN		.015	.015	.015	
INITIAL DIAMETER, IN.		1.36	1.37	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CL), BROWN

95 % MAX. DRY DENS. & WET OF OPT. WC

LL	PL	PI	GS 2.70 (ESTIMATED)	COMPACTED SPECIMEN	$\bar{R}$ TEST

REMARKS:

MOLDING CONDITIONS:

DRY DENSITY = 104.8 PCF

WATER CONTENT = 19.5 %

PROJECT DARLINGTON RESERVOIR

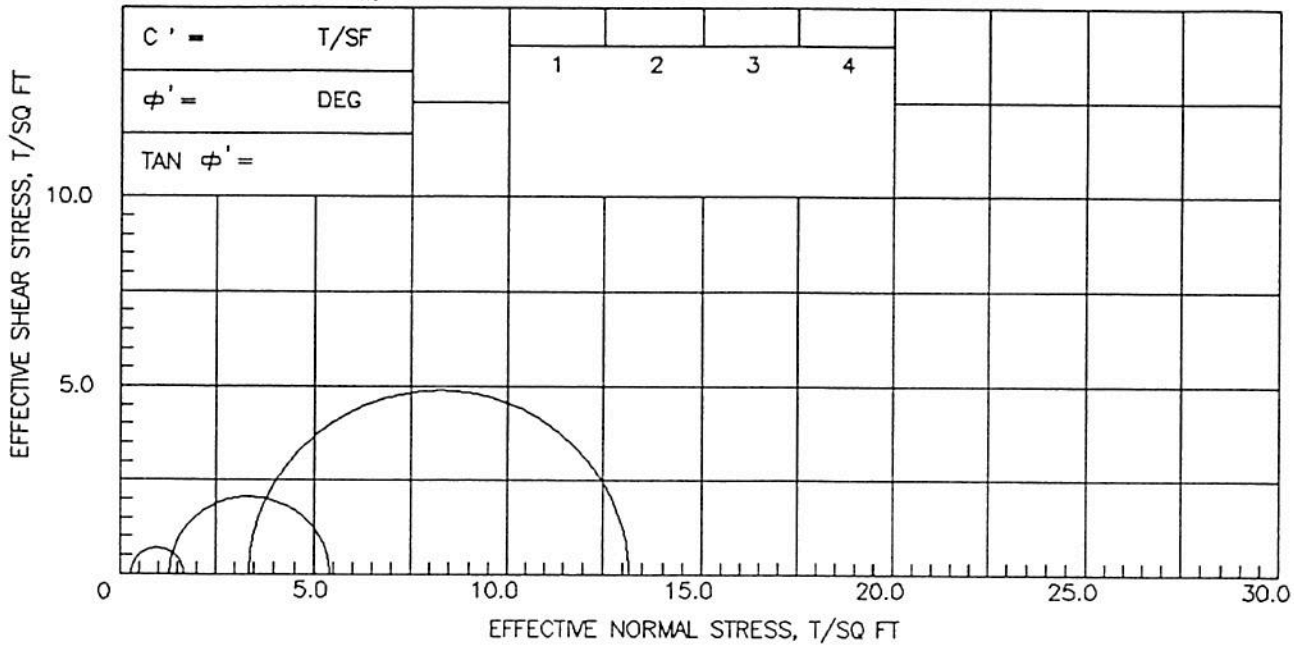
BORING NO. DD-13T SAMPLE NO. 6

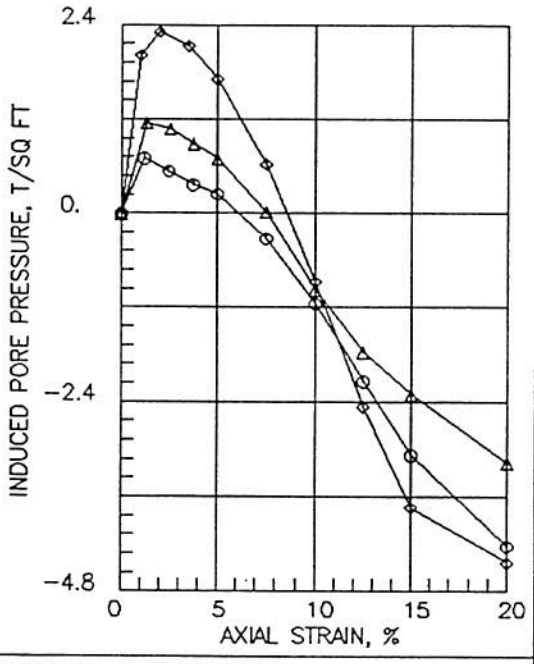
DEPTH/ELEV 8-FT TECH. US

LABORATORY USAE WES - STF/GL DATE 31 MAR 97

$\bar{R}$-TRIAXIAL COMPRESSION TEST REPORT

BASED ON MAX. STRESS RATIO





SPECIMEN NO.		○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	19.2	19.1	19.3	
	DRY DENSITY, PCF	105.0	105.0	104.5	
	SATURATION, %	85.7	85.3	85.1	
	VOID RATIO	.605	.605	.613	
BEFORE SHEAR	WATER CONTENT, %	19.6	19.3	19.3	
	DRY DENSITY, PCF	106.5	107.1	107.7	
	SATURATION, %	91.1	90.9	92.4	
	VOID RATIO	.582	.573	.565	
	BACK PRESS., TSF	5.04	5.04	5.04	
@ σ_1'/σ_3' MAX	MIN PRIN EFF STRESS, TSF	.29	1.29	3.37	
	DEV. STRESS, TSF	1.37	4.11	9.78	
	TIME TO FAILURE, MIN.	81	338	507	
RATE OF STRAIN, %/MIN		.015	.015	.015	
INITIAL DIAMETER, IN.		1.36	1.37	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CL), BROWN

95 % MAX. DRY DENS. & WET OF OPT. WC

LL	PL	PI	GS 2.70 (ESTIMATED)	COMPACTED SPECIMEN	$\bar{R}$ TEST

REMARKS:

MOLDING CONDITIONS:

DRY DENSITY = 104.8 PCF

WATER CONTENT = 19.5 %

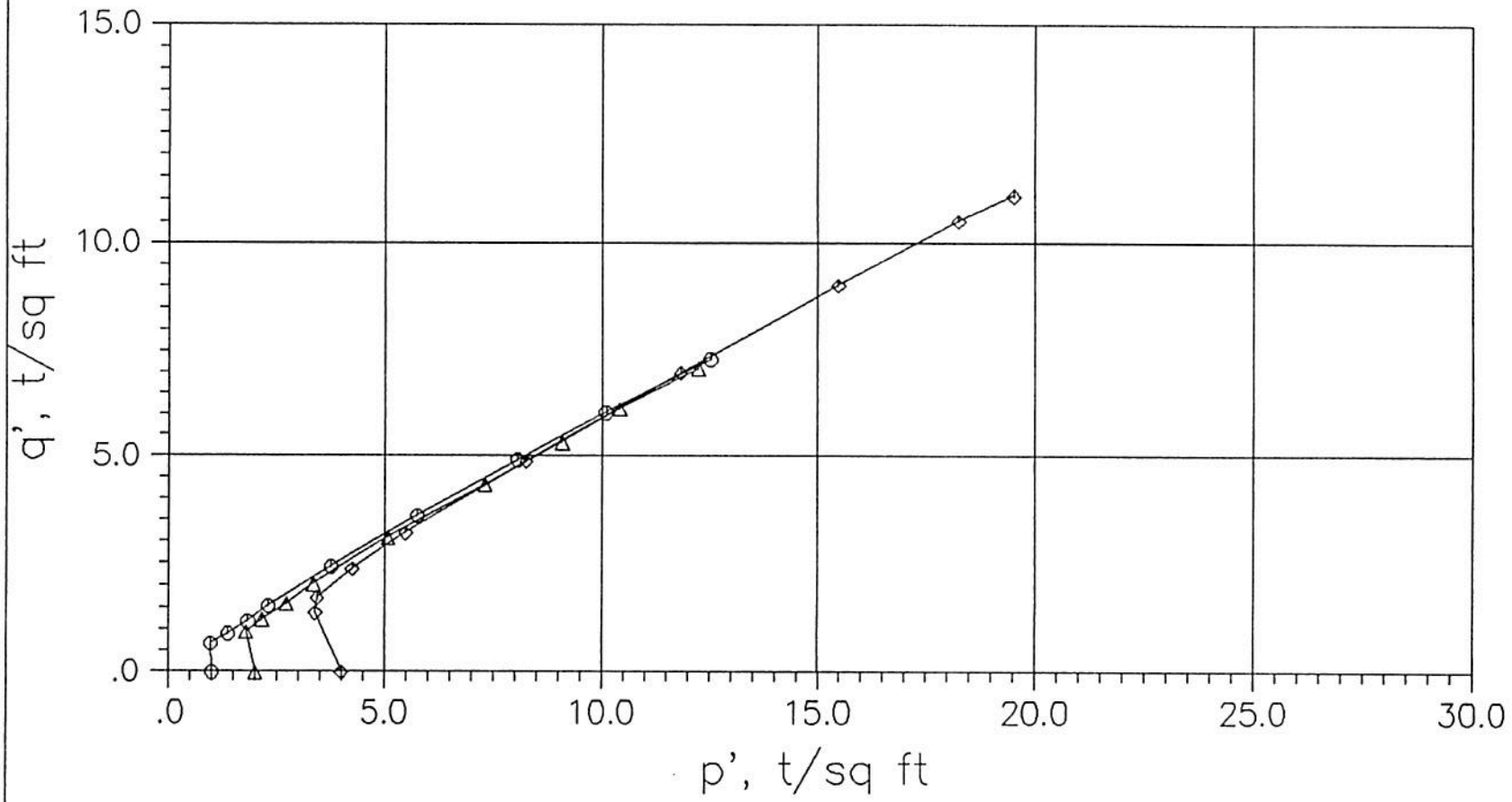
PROJECT DARLINGTON RESERVOIR

BORING NO. DD-13T | SAMPLE NO. 6

DEPTH/ELEV 8-FT | TECH. US

LABORATORY USAE WES - STF/GL | DATE 31 MAR 97

$\bar{R}$-TRIAXIAL COMPRESSION TEST REPORT



PROJECT DARLINGTON RESERVOIR	
BORING DD-13T	SAMPLE NO. 6
DEPTH/ELEV 8-FT	DATE 31 MAR 97

Stress Paths

LABORATORY USAE WES - STF/GL

R-BAR TRIAXIAL COMPRESSION TEST REPORT

TLE: DARLINGTON RESERVOIR

BOR: DD-13T SAM: 6 DEP: 8-FT DAT: 31 MAR 97 TEC: US
DES: CLAY (CL), BROWN
95 % MAX. DRY DENS. & WET OF OPT. WC

REM:
MOLDING CONDITIONS:
DRY DENSITY = 104.8 PCF
WATER CONTENT = 19.5 %
SAMPLE TYPE = COMPACTED
GS: 2.70 (ESTIMATED)
LL: 0 PL: 0 PI: 0

WEIGHT OF SOLIDS = 120.86 I. HEIGHT = 3.000" I. DIAMETER = 1.364"

HEIGHT CHANGES: PRE-CONS. = .00000" DURING CONS. = .00500"

VOLUME CHANGE DURING CONS. = 1.0 CC TIME OF TEST = 1348 MINS.

BACK PRESSURE = 70.00 PSI (5.04 TSF) CHAMBER PRESSURE = 83.89

EFF CONFINING PRESSURE, TSF = 1.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	19.19	19.64
VOID RATIO	.605	.582
SATURATION, %	85.69	91.06
DRY DENSITY, PCF	105.04	106.53

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	EFFECTIVE E3 - TSF -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	70.0	.0	.00	.00	1.000	1.000	1.000	1.00	.00
.036	27.7	79.8	1.2	.71	1.37	1.660	.294	5.636	.98	.68
.074	37.6	77.5	2.5	.54	1.83	2.289	.460	4.976	1.37	.91
.112	50.0	75.1	3.7	.37	2.40	3.034	.633	4.794	1.83	1.20
.150	65.8	73.5	5.0	.25	3.12	3.866	.748	5.168	2.31	1.56
.225	105.7	65.6	7.5	-.32	4.88	6.193	1.317	4.703	3.75	2.44
.300	161.0	54.1	10.0	-1.14	7.23	9.371	2.145	4.369	5.76	3.61
.375	225.7	40.4	12.5	-2.13	9.85	12.980	3.131	4.145	8.06	4.92
.449	284.5	27.4	15.0	-3.07	12.06	16.131	4.067	3.966	10.10	6.03
.599	365.8	11.3	20.0	-4.23	14.60	19.823	5.226	3.793	12.52	7.30

RATE OF STRAIN = .015

R-BAR TRIAXIAL COMPRESSION TEST REPORT

TLE: DARLINGTON RESERVOIR

BOR: DD-13T SAM: 6 DEP: 8-FT DAT: 31 MAR 97 TEC: US
DES: CLAY (CL), BROWN
95 % MAX. DRY DENS. & WET OF OPT. WC

REM:
MOLDING CONDITIONS:
DRY DENSITY = 104.8 PCF
WATER CONTENT = 19.5 %
SAMPLE TYPE = COMPACTED
GS: 2.70 (ESTIMATED)
LL: 0 PL: 0 PI: 0

WEIGHT OF SOLIDS = 121.69 I. HEIGHT = 3.000" I. DIAMETER = 1.369"

HEIGHT CHANGES: PRE-CONS. = .00000" DURING CONS. = .01000"

VOLUME CHANGE DURING CONS. = 1.5 CC TIME OF TEST = 1345 MINS.

BACK PRESSURE = 70.00 PSI (5.04 TSF) CHAMBER PRESSURE = 97.78

EFF CONFINING PRESSURE, TSF = 2.00

	INITIAL	BEFORE SHEAR
WATER CONTENT, %	19.13	19.30
VOID RATIO	.605	.573
SATURATION, %	85.31	90.90
DRY DENSITY, PCF	104.99	107.14

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	EFFECTIVE E3 - TSF -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	70.0	.0	.00	.00	2.000	2.000	1.000	2.00	.00
.039	39.0	86.1	1.3	1.16	1.91	2.756	.841	3.277	1.80	.96
.076	51.2	85.0	2.5	1.08	2.48	3.402	.920	3.698	2.16	1.24
.113	67.3	82.3	3.8	.89	3.22	4.336	1.114	3.890	2.73	1.61
.150	87.0	79.8	5.0	.71	4.11	5.405	1.294	4.176	3.35	2.06
.225	134.8	70.3	7.5	.02	6.20	8.179	1.978	4.134	5.08	3.10
.299	194.1	56.6	10.0	-.96	8.69	11.654	2.965	3.931	7.31	4.34
.374	245.0	45.5	12.5	-1.76	10.66	14.427	3.764	3.833	9.10	5.33
.448	289.1	38.0	15.0	-2.30	12.23	16.530	4.304	3.841	10.42	6.11
.598	355.5	26.0	20.0	-3.17	14.15	19.315	5.168	3.738	12.24	7.07

RATE OF STRAIN = .015

R-BAR TRIAXIAL COMPRESSION TEST REPORT

TLE: DARLINGTON RESERVOIR

BOR: DD-13T SAM: 6 DEP: 8-FT DAT: 31 MAR 97 TEC: US
DES: CLAY (CL), BROWN
95 % MAX. DRY DENS. & WET OF OPT. WC

REM:
MOLDING CONDITIONS:
DRY DENSITY = 104.8 PCF
WATER CONTENT = 19.5 %
SAMPLE TYPE = COMPACTED
GS: 2.70 (ESTIMATED)
LL: 0 PL: 0 PI: 0

WEIGHT OF SOLIDS = 119.54 I. HEIGHT = 3.000" I. DIAMETER = 1.360"

HEIGHT CHANGES: PRE-CONS. = .00300" DURING CONS. = .01900"

VOLUME CHANGE DURING CONS. = 1.9 CC TIME OF TEST = 1341 MINS.

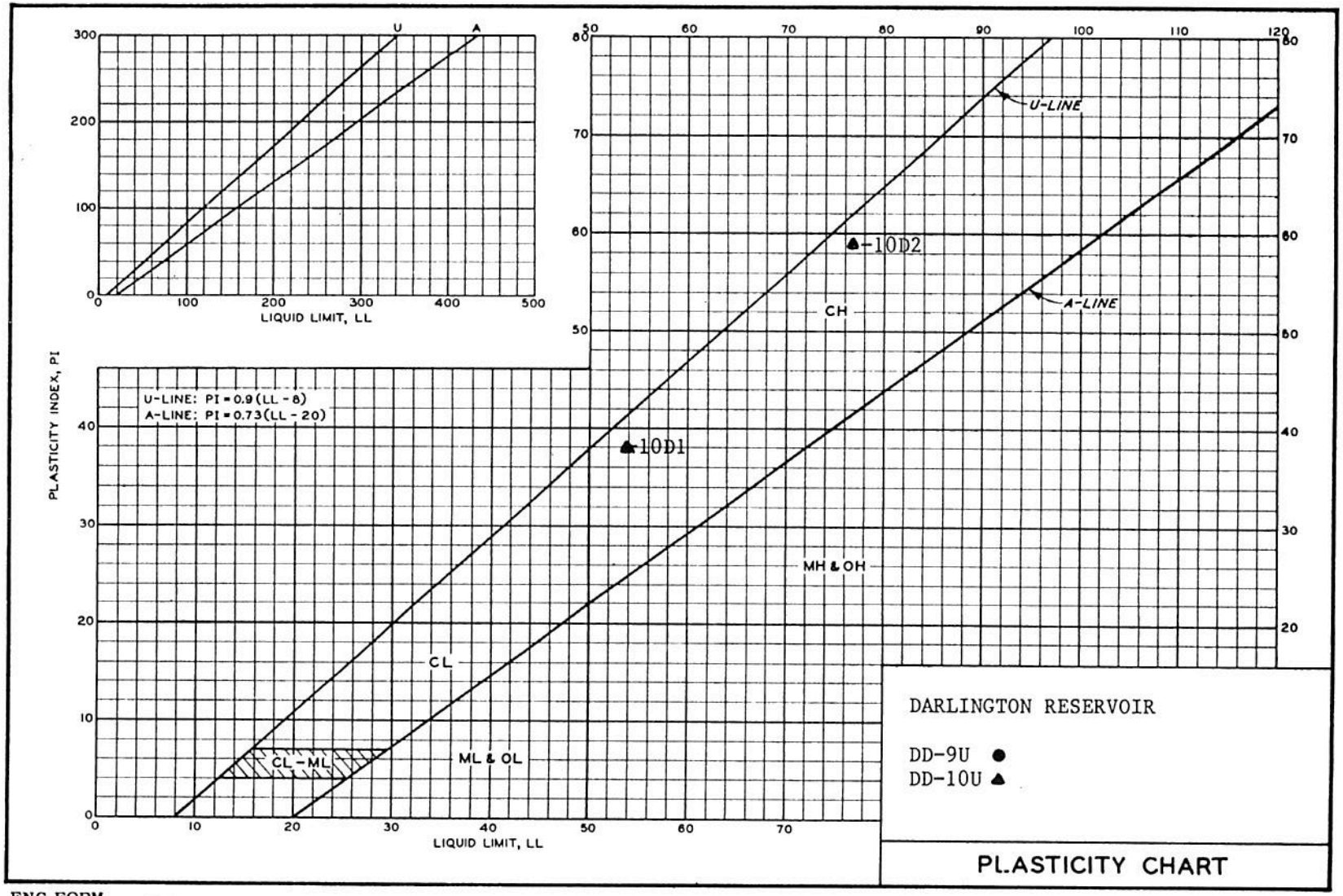
BACK PRESSURE = 70.00 PSI (5.04 TSF) CHAMBER PRESSURE = 125.56

EFF CONFINING PRESSURE, TSF = 4.00

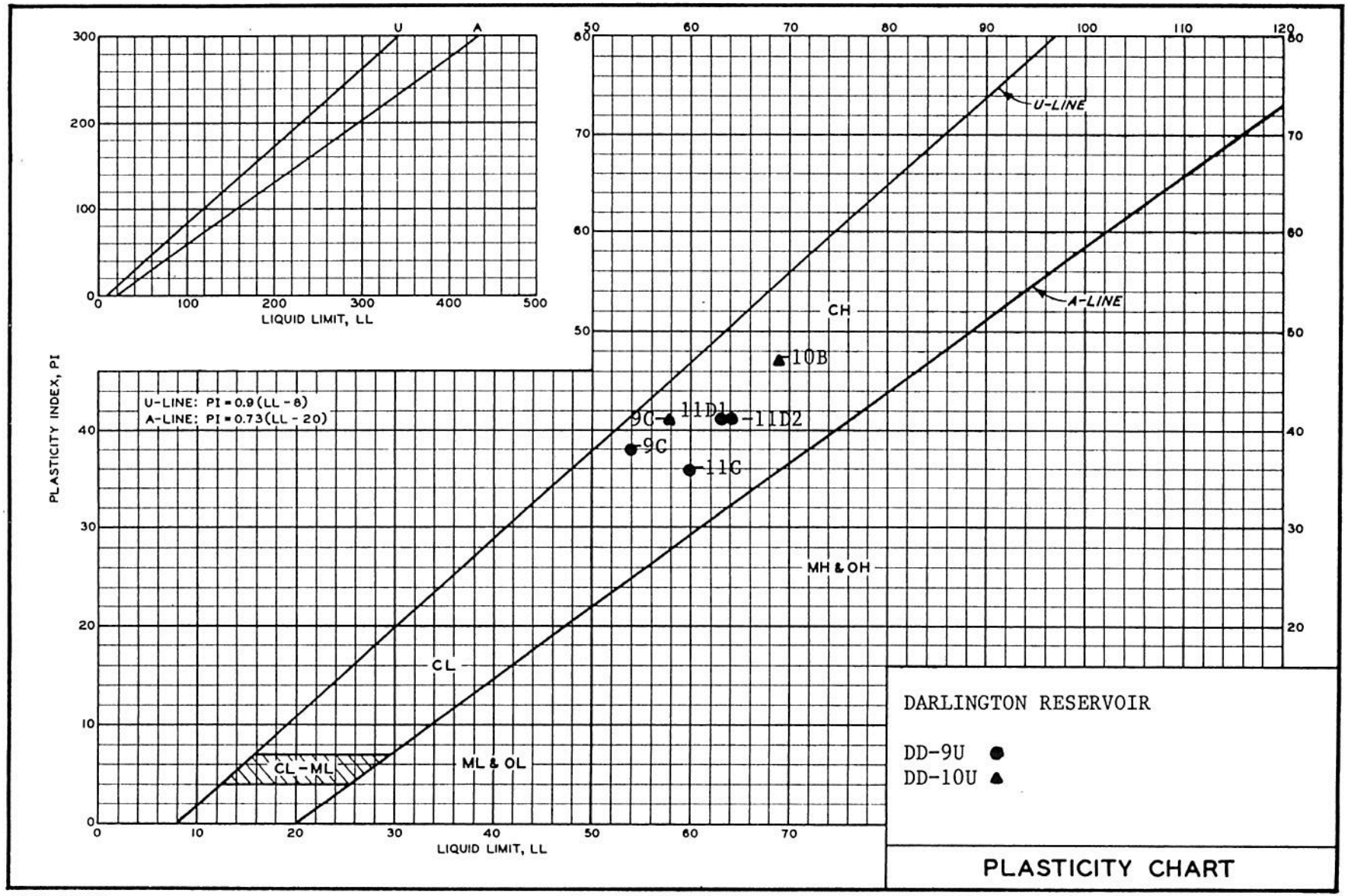
	INITIAL	BEFORE SHEAR
WATER CONTENT, %	19.31	19.34
VOID RATIO	.613	.565
SATURATION, %	85.07	92.39
DRY DENSITY, PCF	104.51	107.70

V DEF IN.	AXIAL LOAD LBS	PORE PRESS PSI	STRAIN %	IND POR PRESS TSF	DEVIAT STRESS TSF	EFFECTIVE E1 - TSF -	EFFECTIVE E3 - TSF -	RATIO E1/ E3	NORML STRES TSF	SHEAR STRES TSF
.000	.0	70.0	.0	.00	.00	4.000	4.000	1.000	4.00	.00
.030	56.0	98.0	1.0	2.02	2.81	4.795	1.984	2.416	3.39	1.41
.059	70.3	102.2	2.0	2.32	3.49	5.175	1.682	3.077	3.43	1.75
.104	97.5	99.7	3.5	2.14	4.77	6.632	1.862	3.562	4.25	2.39
.149	133.0	93.8	5.0	1.71	6.41	8.692	2.287	3.801	5.49	3.20
.224	208.6	78.8	7.5	.63	9.78	13.146	3.366	3.905	8.26	4.89
.298	305.7	58.0	10.0	-.86	13.95	18.811	4.864	3.867	11.84	6.97
.373	406.6	35.9	12.5	-2.46	18.03	24.486	6.455	3.793	15.47	9.02
.447	488.3	18.2	15.0	-3.73	21.04	28.769	7.729	3.722	18.25	10.52
.596	546.5	8.5	20.0	-4.43	22.16	30.589	8.428	3.630	19.51	11.08

RATE OF STRAIN = .015



ENG FORM 4334 JUN 70 (E M 1110-2-1906) TRANSLUCENT


U
A
300
200
100
0
0
100
200
300
400
500
LIQUID LIMIT, LL
50
60
70
80
90
100
110
120
U-LINE
A-LINE
CH
10B
9C
11D1
11D2
9C
11C
MH & OH
CL
CL - ML
ML & OL
U-LINE: PI = 0.9 (LL - 8)
A-LINE: PI = 0.73 (LL - 20)
PLASTICITY INDEX, PI
LIQUID LIMIT, LL
DARLINGTON RESERVOIR
DD-9U
DD-10U
PLASTICITY CHART