LAKE PONTCHARTRAIN, LA. AND VICINITY

HIGH LEVEL PLAN

DESIGN MEMORANDUM NO. 19A
GENERAL DESIGN
SUPPLEMENT NO. 2

LONDON AVENUE OUTFALL CANAL
FRONTING PROTECTION
DRAINAGE PUMPING STATION NO. 3

ORLEANS PARISH

PREPARED FOR THE

BOARD OF LEVEE COMMISSIONERS
OF THE ORLEANS LEVEE DISTRICT
ORLEANS LEVEE BOARD PROJECT NO. 24914

BY

PEPPER AND ASSOCIATES, INC.
CONSULTING ENGINEERS

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LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
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PROJECT HISTORY

1. Background

A. Hurricane protection for the London Avenue Outfall Canal was presented in the report entitled "Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 19A - General Design, London Avenue Outfall Canal (DM 19A)." Two plans were presented for providing hurricane protection for London Avenue Canal. One plan concept, and the one recommended, was to provide fronting protection at/or near the lakefront end of the canal by the construction of a gated structure. The other plan, and the one supported by the Orleans Levee District (OLD) would require upgrading the height of the existing 2.4 miles of parallel levees along both sides of the canal. Also required would be floodproofing the bridges at Leon C. Simon Blvd., Robert E. Lee Blvd., Filmore Ave., Mirabeau Ave., Gentilly Blvd., modification to the Norfolk Southern Railroad bridge and fronting protection at Drainage Pumping Stations 3 and 4. With reference to the parallel levee plan, only the levees and floodwalls were adequately designed and presented in DM 19A. As stated in the DM, additional Design Supplements would be required for floodproofing the bridges and for fronting protection at the two pumping stations.

B. The parallel protection plan for London Avenue Outfall Canal was mandated by Congress in the FY 1992 Energy and Water Development Appropriations Act as the flood protection plan that best suits the intent of Congress. This fronting protection project at Drainage Pumping Station No. 3 (DPS#3) also falls under the same authority.

2. Purpose

This supplement to Design Memorandum No. 19A presents the essential data, assumptions, computations and criteria used in the design of the fronting protection at Drainage Pumping Station No. 3 and is prepared in sufficient detail to provide an adequate basis for preparing the plans and specifications.

3. Drainage Pumping Station No. 3

A. Location - DPS#3 is located at the southernmost end of London Avenue Outfall Canal where it commences. It is situated at the intersection of Abundance St. and North Broad Avenue (see plate 1). Existing flood protection is provided by a floodwall system on either side of the canal which ties into the foundation and building structure of the station.

B. Description - The station houses three 1000 CFS horizontal pumps, two 500 CFS horizontal pumps and four 20 CFS vertical constant duty pumps. The Sewerage and Water Board of New Orleans (S&WB) requires that the station be kept in operation at all times for the duration of the construction project with only one (1) pump being taken out of service at a time.

C. Structural - The foundation of the station consists of a reinforced concrete slab supported by timber piles. The discharge basin slab is also pile supported and is always under approximately 10 feet of water which fluctuates with the Lake Pontchartrain tide.

PROJECT PLAN

4. Flood Protection Plan

The S&WB has mandated that this flood protection project be accomplished by utilizing a sluice gate structure with concrete discharge tubes similar to the one constructed at DPS#19. This recommended plan was agreed upon after several coordination meetings with the S&WB. The structure will be constructed approximately 125 feet north of the station and each of the five (5) horizontal pumps will have an individual concrete discharge tube adjoining the sluice gate structure. There will be two (2) sluice gates at the termination of each concrete discharge tube for a total of ten (10) gates. This protection will incorporate the use of T-wall monoliths to tie in the new structure to the existing floodwalls of the canal.

DESCRIPTION OF PROPOSED STRUCTURES

5. General Description of Proposed Structures

The project plan is to construct a sluice gate structure across the entire width of the canal approximately 125 feet north of the station to provide fronting protection (see plates 3, 8, 14). There will be a six gate structure and a separate four gate structure that will be joined together at the center of the discharge basin by use of a T-wall monolith. Two (2) more T-walls will be used to connect the ends of the structure to the existing canal floodwalls while two (2) I-walls will adjoin the newly installed railroad gate monoliths. Each horizontal pump will have an isolated concrete discharge tube similar to Station 19. Ten (10) sluice gates will provide emergency closure capabilities in the event of pump failure. Each concrete discharge tube will be fronted by two (2) gates (see plates 5, 10, 19). The existing concrete discharge basin slab will have to be removed in the areas where the new structure will be constructed (see plates 8, 14). Pile-founded concrete T-walls and concrete capped steel sheet pile I-walls will tie the new protection to the existing protection. Power for all gate operators shall be supplied by "T2 Power Panel" within DPS#3.

A. Sluice Gate Monolith for 1000 CFS Pumps (G-2)

A gated monolith will be utilized in front of the discharge area of Pumps C, D & E (1000 CFS pumps) (see plates 3, 11, 13, 14, 18, 19). This monolith will house six (6) - 108" x 96" cast iron sluice gates. The structure will be reinforced concrete with a top El. 16.82 NGVD, founded on steel HP14x73 piles. Steel H-piles, in lieu of prestressed concrete piles, will be used because they tend to slice through the soil better imparting less vibration to the existing pump station structure. The operating floor will have a handrail surrounding it and will be constructed from concrete with steel bar grate sections for access to the gate hoisting assemblies (see plates 12, 15). Placement of reinforcing steel, embedded steel items, construction joints and water stops will conform to construction industry standards. Expansion joints, if required, between monoliths, will include 0.5 inch joint filler.

Dewatering slots for stoplogs will be provided for periodic monolith and gate inspection and maintenance. Monolith maintenance will include all required structural and cosmetic repairs, and debris removal. Gate maintenance will include functional checks and periodic replacement of the flush bottom seal.

B. Sluice Gate Monolith for 500 CFS Pumps (G-1)

A gated monolith will be utilized for Pumps A & B (500 CFS pumps) (see plates 3, 4, 7, 8, 18, 19). This monolith will house four (4) - 81" x 96" cast iron sluice gates. The remaining description is similar to the preceding, in paragraph A.

C. <u>Concrete Discharge Tubes</u>

Each horizontal pump will have an isolated concrete discharge tube approximately 90 feet long connecting the pump discharge pipe to the sluice gate structure (see plates 3, 8, 14). These tubes will also be reinforced concrete founded on steel HP14x73 piles. The highest floor elevation inside the tube (hump) will be El. 6.61 NGVD. The purpose of the hump is to keep normal lake water tides from siphoning back into the suction basin while allowing the end of the discharge tube to be totally submerged at all times. This allows any of the pumps to be primed at will. A steel pipe section approximately 20 feet long will connect the pump flange to the concrete discharge tube. The purpose of this pipe is to isolate new construction from the existing station and compensate for any differential settlement which may occur. The slightest movement could alter alignment of a pump, rendering it inoperable.

Each 1000 CFS pump will be temporarily shut-down, one at a time, for construction of the respective concrete discharge tube. Each tube will be constructed without interfering with flow through the existing adjacent discharge bells. After work is completed on the discharge tubes for the three 1000 CFS pumps and all are working properly, then the remaining two 500 CFS pumps may be shut-down simultaneously for construction of the final two concrete discharge tubes for pumps A & B.

D. <u>Gates</u>

Cast iron sluice gates with electrical motor-driven operators will be used based on S&WB requirements. There will be six (6) - 108" x 96" gates and four (4) - 81" x 96" gates. Each gate will have manual back-ups in case of power failure.

E. <u>T-Wall Monoliths</u>

There will be a total of three (3) T-wall monoliths (see plates 3, 9, 18, 20, 22, 24). One closure monolith will adjoin the two sluice gate structures at the center of the discharge basin. The other closure monoliths will adjoin the ends of the two sluice gate structures with the existing canal walls. These monoliths will be inverted T-type reinforced concrete structures, top El. 13.9 NGVD, founded on steel HP14x73 piles, with PZ-22 or equal steel sheet pile seepage cut-off. Steel H-piles, instead of prestressed concrete piles, are used for simpler handling, splicing requirements during placement, and for potential emergency cut-offs.

F. I-Wall Monoliths

There will be a total of two (2) I-wall monoliths (see plates 3, 18, 19). Each I-wall will connect the ends of the T-walls to the ends of the existing I-walls with concrete caps on each side of the London Ave. Outfall Canal. The I-type floodwall will consist of steel sheet piles capped with a reinforced concrete wall. The top elevation will be 14.4 NGVD. This elevation will be 0.5 feet higher than the pile founded monoliths to account for settlement. Steel sheet piling sizes will include the existing and new PZ-22 or equal. Expansion joints in the floodwall will be spaced approximately 30 feet apart and/or at each change in direction and shall be adjusted to fall at the steel sheet pile interlocks.

G. Walkways and Operating Floor

The operating floor for the sluice gate structure will be constructed from concrete and shall have removable hot dipped galvanized steel bar grate sections for access to the gate hoisting assemblies (see plates 4, 6, 11, 12, 15). Aluminum handrails and posts will be installed at perimeter of each operating floor and along both sides of walkway adjoining the two (2) sluice gate monoliths. Access to sluice gate structure will be by walkway adjacent to concrete discharge tube for pump B and by stairway in between the two sluice gate monoliths.

H. Dewatering Bulkheads

Dewatering bulkheads, i.e., stoplogs, will be single, solid panels designed to fit the gated monolith dewatering slots. The stoplogs will be reinforced concrete, structural steel or aluminum and will provide water retention to canal water El. 4.0 NGVD.

I. Temporary Sheet Pile Dam

A temporary dam will be constructed across entire width of the London Ave. Outfall Canal to allow for a dewatered work area (see plate 30). Top of dam shall be El. 1.57 NGVD as mandated by the S&WB. The dam will consist of PZ-27 or equal cantilevered steel sheet piles with four (4) - 66" x 66" electrically operated butterfly gates. There will also be an access walkway at El. 4.0 NGVD attached to the dam for manual operation of the butterfly gates.

The butterfly gates will allow canal water to flood the dewatered work area quickly, so that the existing discharge bells can be sealed in order to start priming the horizontal pumps. Once the pumps are loaded and if there is more flow than the butterfly gates can accommodate, the excess flow will spill over the top of the temporary dam.

J. Temporary Concrete Weir

A temporary weir will be constructed in the discharge well for pumps A & B to keep these two (2) discharge bells sealed at all times. In this way, these two (2) horizontal pumps can be primed and loaded at any time required to help fill up the dewatered work area in less time than with just the four (4) butterfly gates located on the temporary dam. The constant duty flow will spill over the top of the weir, maintaining the water seal on both discharge bells.

K. Existing Canal Lining

Portions of the reinforced concrete lining of the London Ave. Outfall Canal which must be removed during construction will be replaced upon completion of the fronting protection.

STRUCTURAL DESIGN

6. Scope

The scope of the structural analyses and design is limited to preliminary determination of thicknesses for various structural concrete components, reinforcing in these components, sizes of structural steel elements and preliminary pile layouts for the Fronting Protection structures and the five (5) Horizontal Pump Discharge Tubes for Drainage Pumping Station No. 3.

The analyses and design methods used were simplified for the purpose of the preliminary design. Where possible, ACI Coefficients for determining bending moments and shears in continuous structures were used in lieu of resorting to manual or microcomputer based analytical solutions for continuity. A volume entitled "Preliminary Design Calculations" containing computations for the aforesaid is submitted separately.

STAAD III/ISDS, Release 20, a microcomputer based finite element solution for structural analyses and design, developed by Research Engineers, Inc. of Yorba Linda, CA will be used for the final design of this project.

7. References

Applicable provisions of the following codes, specifications, manuals and technical letters shall govern the design of various structures and components thereof.

A. COE publications

(1) EM 1110-1-2101 Working Stresses for Structural Design

(2) EM 1110-2-2104 Strength Design for Reinforced Concrete hydraulic structures.

(3) EM 1110-2-2502 Retaining and Floodwalls

(4) EM 1110-2-2906 Design of Pile Foundations

(5) EM 1110-2-2504 Design of Sheet Pile Walls

B. Technical publications

(1) American Concrete Institute, Building Code Requirements for Reinforced Concrete (ACI 318-89)

(2) American Institute of Steel Construction (AISC), Manual of Steel Construction, Allowable Stress Design, Ninth Edition, 1989.

(3) American Welding Society, Structural Welding Code, Steel, (AWS-D 1.1-88).

C. Computer Programs

(1) Pile Group Analysis (CPGA) WES Program X0080

(2) "CFRAME" WES Program X0030

(3) STAAD-III/ISDS, Finite Element Analyses and Design Program.

8. Design Criteria

A. General

The structural design calculations contained in the volume entitled "Preliminary Design Calculations" comply with all applicable provisions of the Codes, Specifications, Manuals and Technical letters listed in previous paragraphs.

B. <u>Material Weights</u>

The following weights for different materials listed below were used in the design computations:

ITEMS	Weight in PCF
Water	62.5
Normal Weight Concrete	150
Steel	490
Saturated Sand	122
Saturated Granular Backfill	122

C. <u>Design Stresses</u>

(1) <u>Structural Steel</u>: Allowable stresses shall be in accordance with AISC, Manual of Steel Construction, Allowable Stress Design, as modified by EM 1110-1-2101.

(2) <u>Welds</u>: Allowable stresses for the design of welds shall be in accordance with American Welding Society, Structural Welding Code,Steel, as modified by EM 1110-1-2101.

(3) <u>Steel Sheet Piling</u>: Allowable stresses for steel sheet pile walls shall be in accordance with EM 1110-2-2504.

(4) <u>Reinforced Concrete</u>

(a) Reinforced concrete design for the structural elements of the Fronting Protection shall be based on ultimate strength design methods and criteria set forth in EM 1110-2-2104. Allowable stresses and Load factors for the discharge tube design for Pumps A & B and Pumps C, D & E shall be based on American Concrete Institute, Building Code Requirements for Reinforced Concrete, (ACI 318-89).

(b) All concrete shall have a 28 day compressive strength of f'_c = 4000 psi.

(c) Maximum flexural reinforcement shall not exceed 0.375xpb.

(d) Reinforcing steel shall conform to ASTM A 615 Grade 60.

9. Loading Conditions

A. General

Fronting Protection structures were analyzed and designed subject to the following hydraulic loading conditions:

(1) Usual Loading Condition: Standard Project Hurricane (SPH) with still water level @ EL. 11.9 NGVD (32.33 C.D.)

(2) Unusual Loading Condition: Still water level 2'-0" above SPH water level, i.e., @ EL. 13.9 NGVD (34.33 C.D.)

For all loading conditions which included hydrostatic loading, two uplift conditions namely pervious and impervious uplift were considered to account for the effectiveness of the steel sheet pile cut-off wall.

(3) Discharge tubes for Pumps A & B and Pumps C, D & E were designed for both negative (priming loads) and positive hydrostatic pressures and the effect of hydrostatic uplift.

B. Sluice Gate Structures for Pumps A & B and Pumps C,D & E

Sewerage and Water Board's operations require that these gates at the north end of the discharge tubes remain open at all times except when water level in London Ave. Canal is at or above high point (EL. 6.61 NGVD or 27.04 C.D.) in the discharge tubes and one or more pumps are mechanically or otherwise inoperable. It is only at these times of emergency that the gates in the affected discharge tubes will be closed to prevent backflow into the suction basin.

The following load cases were investigated for both the foundation and structural design of these monoliths:

(1) Usual Conditions

(a) Gates Closed, Canal SWL @ EL. 11.9 NGVD (32.33 C.D.) Water Level in Discharge Tube @ EL. 6.61 NGVD (27.04 C.D.), Storm Wind, Backfill in place. Impervious Sheet pile cut-off.

(b) Same as above but Pervious sheet pile cut-off.

(2) Unusual Conditions

(a) Gates Closed, Canal SWL @ EL. 13.9 NGVD (34.33 C.D.) Water Level in Discharge Tube @ EL. 6.61 NGVD (27.04 C.D.), Storm Wind, Backfill in place. Impervious Sheet pile cut-off.

(b) Same as above but Pervious sheet pile cut-off.

(3) Maintenance Conditions

(a) Stop logs in place @ all pumps or at any pump, Canal SWL @ EL. 3.82 NGVD (24.25 C.D.), Water Level in discharge @ EL. -9.18 NGVD (11.25 C.D.), Operating Wind, Backfill in place, Impervious sheet pile cut-off.

(b) Same as above but Pervious sheetpile cut-off.

(4) Construction Condition

(a) Completed Structure in place prior to watering. No wind load or earth loads. No hydrostatic loads.

C. T-Wall Monoliths

The following loading conditions were investigated for both the foundation and structural design of these monoliths:

(1) Usual Conditions

(a) Canal SWL @ EL. 11.9 NGVD (32.33 C.D.), Storm Wind, Backfill in place, Impervious sheet pile cut-off.

(b) Canal SWL @ EL. 11.9 NGVD (32.33 C.D.), Storm Wind, Backfill in place, Pervious sheet pile cut-off.

(2) Unusual Conditions

(a) Canal SWL @ EL. 13.9 NGVD (34.33 C.D.), Storm Wind, Backfill in place, Impervious sheet pile cut-off.

(b) Canal SWL @ EL. 13.9 NGVD (34.33 C.D.), Storm Wind, Backfill in place, Pervious sheet pile cut-off.

(c) Canal SWL @ EL. -9.18 NGVD (11.25 C.D.), Operating Wind, Backfill in place, Impervious sheet pile cut-off.

(d) Canal SWL @ EL. -9.18 NGVD (11.25 C.D.), Operating Wind, Backfill in place, Pervious sheet pile cut-off.

(3) <u>Construction Condition</u>

(a) Completed T-Wall in place, No backfill, No water in canal, No wind.

D. <u>Discharge Tubes for Pumps A & B and Pumps C, D & E</u>

The discharge tubes were designed for both positive and negative hydrostatic pressures. The negative hydrostatic pressure resulting from priming of horizontal pumps at Drainage Pumping Station No. 3 was assumed to be equal to 18 ft. of H_2O. The Sewerage & Water Board is presently in the process of installing a pressure gauge on the discharge side of impeller of horizontal Pump D. This will facilitate determination of actual negative pressure that may be exerted on the discharge tube structures. Final design will be based on the field measured vacuum loads during pump priming and operation.

10. <u>Structural Design</u>

As indicated previously, analyses and design methods were simplified for the purpose of arriving at preliminary thicknesses and reinforcing in structural elements of each of the proposed fronting protection and discharge tube structures. PC based finite element programs will be used in the final design of these structures.

Each of the fronting protection and discharge tube structures were designed as follows:

A. <u>Sluice Gate Structures for Pumps A & B and Pumps C,D & E</u>

Design computations were performed for Sluice Gate Structure, G-2, at Pumps C, D & E discharge only (see plate 23). Since the Sluice Gate Structure G-1 at Pumps A & B discharge is structurally identical, dimensionally smaller in plan and subjected to same loading intensities as the gate structure at Pumps C, D & E, reinforcing and thicknesses identical to gate structure at Pumps C, D & E were used for structural components of gate structure at Pumps A & B without further computations (see plate 21). Pile layout, however, was determined separately for each foundation.

Different components of gate structures were designed as follows:

(1) Longitudinal Walls (North & South Walls): These walls were assumed continuous over transverse walls and simply supported at east and west walls. It was further assumed that the longitudinal wall will transfer loads horizontally to the transverse walls. Reinforcing in these walls was determined based on the flexural stresses caused by out of plane loading on these walls.

(2) Transverse Walls: Interior walls were assumed to be fixed at base slab, continuous over slab @ EL. 25.25 C.D. and simply supported at operating platform level for out of plane loading conditions. For in plane loading, fixity was assumed at the base slab and bracing perpendicular to weaker axis was assumed to be furnished by the slab @ EL. 25.25 and the operating platform at EL. 37.25. Both in plane and out of plane loadings were used to determine flexural and shear stresses. East and West walls were also designed in similar fashion.

(3) Columns: The column between gates in each discharge tube was designed for in plane loading. This column was assumed pinned at each end. The column directly in front of this column and on flood side of the gates was also designed in similar fashion.

(4) Base Slab: In longitudinal direction, Base slab was assumed simply supported at east and west walls of the gate structures and continuous at interior walls and columns. Vertical components of pile loads were assumed to act as point loads on the base slab at their respective locations. Flexural and shear stresses were determined and reinforcing was provided based on these assumptions. Adequacy of base slab to span longitudinally over piles was also checked. In the transverse direction, minimum flexural reinforcing was provided for the purpose of the preliminary design.

B. T-Wall Monoliths

T-Wall was designed as a pile supported cantilever retaining wall.

(1) Stem: Stem was assumed fixed at the base slab and was designed to transfer loads to the base slab vertically. Out of plane loading was used to determine flexural and shear reinforcing.

(2) Base slab: Base slab was designed to transfer load horizontally to the piles. It was assumed fixed at the face of stem with vertical components of pile loads acting as point loads at the pile locations.

C. Discharge Tubes

Design calculations were performed for the discharge tubes at Pumps C, D & E only. The top, bottom and foundation slabs span shorter distances in discharge tubes for Pumps A & B. Also both tubes are subjected to identical loading intensities. Therefore, reinforcing and thicknesses identical to discharge tubes for Pumps C, D & E were selected for discharge tubes at Pumps A & B without further computations. Pile layout computations for each were performed separately (see plates 25, 26).

The discharge tubes were designed for both positive and negative hydrostatic pressures.

(1) Top Slab: Top slab was designed as a continuous slab simply supported at east and west walls of the tubes and continuous over interior walls. Vacuum load was assumed to be equal to 18 ft. of H_2O for the purpose of the preliminary design. Final design will be based on field measured vacuum loads resulting from priming of the horizontal pumps.

(2) Walls: Walls were designed as compression members subjected to combined axial and bending stresses. Both in plane and out of plane loadings were used to determine reinforcing.

(3) Bottom slab: Bottom slab was designed in the manner similar to top slab design.

(4) Foundation slab: Foundation slab was designed continuous over interior walls and simply supported at east and west walls of the discharge tubes. Pile loads were assumed as point loads acting on the foundation slab at the pile locations. Out of plane loading was used to determine flexural and shear stresses.

11. Cathodic Protection and Corrosion Control

Cathodic protection and corrosion control for steel sheet piling, steel gates, corner plates and all other ferrous metal components of the fronting protection plan shall be provided.

METHOD OF CONSTRUCTION

12. General

All construction will be performed in dry conditions behind the Temporary Sheet Pile Dam. The Contractor will have to vacate the work area during all rain events in which the pumps are operating (loaded). Only one 1000 CFS pump may be taken out of service at a time during the entire construction process. The construction easement shall include the vacant property west of the existing discharge basin. All electrical and piping relocations will be coordinated with the Sewerage and Water Board.

13. SUGGESTED GENERAL CONSTRUCTION SEQUENCE

A. (1) Construct a cantilevered steel sheeting dam at C.O.E. Sta. 2+58 E/BL (Treasure St.) across the canal with sill EL. 22.0 C.D. with four (4) 66" sq. butterfly gates; then dewater area between temporary dam and DPS#3. Water may be emptied into the Florida Ave. Canal.

(2) Simultaneously construct an 8" wide concrete wall (weir) in discharge well of Pumps A and B, at an elevation approximately 6 inches above bottom lip of higher hood to keep both discharge hoods sealed at all times. This is required to allow dewatered work area to be flooded within a 15 minute period to seal discharge hoods so that remaining pumps can be primed by vacuum. Pumps A & B are to be kept in service until all three (3) 1000 CFS pumps have been returned to service with their respective concrete discharge tubes.

(3) Butterfly valves are to be opened to flood work area within a 15 minute period to seal remaining pump discharge hoods; and left open until all station pumps are "shut down".

B. (1) Temporarily relocate 48" dia. SFM North of RR Bridge along side RR R/W. This requires jacking under RR tracks on East and West sides of London Ave. Canal; and passes over I-wall on East and West sides of canal. Pipe to be supported on steel H-piles and steel cap bents.

(2) Relocate any electrical feeder cables that are in the way of new construction.

C. (1) Break out bottom slab of existing discharge basin to allow construction of sluice gate structure across full width of canal.

(2) Drive all foundation piling, place reinforcing steel and cast reinforced concrete base slab of sluice gate structure.

D. Construct walls of sluice gate structure across full width of discharge basin with gates operational.

E. (1) Take Pump C out of service.

(2) Remove discharge piping from flange inside building wall including discharge hood.

(3) Close sluice gates for Pump C in sluice gate structure.

F. Drive steel sheeting on east side of proposed concrete discharge tube and on west side in space between discharge hood for Pump D. Steel sheeting on both sides to connect to station building and sluice gate structure (see plate 29).

G. (1) Construct concrete discharge tube for Pump C and install steel transition section between pump flange and concrete discharge tube.

(2) Restore Pump C to service and open sluice gates for Pump C.

(3) Remove steel sheeting on east and west sides of new concrete discharge tube of Pump C.

H. (1) Take Pump D out of service.

(2) Remove steel discharge piping from flange inside building wall including discharge hood.

(3) Close sluice gates for Pump D in sluice gate structure.

I. Drive steel sheeting between Pumps D and E in a location to permit construction of concrete discharge tube for Pump D and connect to station building and sluice gate structure (see plate 29).

J. (1) Construct concrete discharge tube for Pump D and install steel transition section between pump flange and concrete discharge tube.

(2) Restore Pump D to service and open sluice gates for Pump D.

K. Remove steel sheeting between Pumps D and E.

L. (1) Take Pump E out of service.

(2) Remove steel discharge piping from flange inside building including discharge hood.

(3) Close sluice gates for Pump E in sluice gate structure.

M. (1) Construct concrete discharge tube for Pump E and install steel transition section between pump flange and concrete discharge tube.

(2) Restore Pump E to service and open sluice gates for Pump E.

N. (1) Close sluice gates for Pumps A & B in sluice gate structure.

(2) Take Pumps A & B out of service.

(3) Remove steel discharge piping including discharge hoods from flanges inside building wall.

(4) Relocate constant duty pump discharge piping to Marigny Gate closure location.

(5) Install low sill dam on east side of Marigny Gate to keep water from backing up from the Florida Avenue Canal. Remove existing butterfly gate (Marigny Gate). Seal opening with a concrete retaining wall and provide sleeve for CD piping.

(6) Remove low sill dam from east side of Marigny Gate.

(7) Remove existing London Ave. Gate and related structures.

(8) Remove existing west retaining wall from London Ave. Gate to station building (see plate 2).

O. (1) Construct concrete discharge tube for Pumps A & B and install steel transition sections between pump flanges and concrete discharge tubes.

(2) Restore Pumps A & B to service.

P. (1) Relocate 48" dia. SFM to permanent location on protected (south) side of sluice gate structure.

(2) Remove temporary 48" dia. SFM including pile bents. Fill holes in concrete lining of canal.

Q. Construct Monolith T-3 on West side of canal.

(1) Break out existing canal bottom slab to permit removal of existing timber piles in conflict with new steel H-piles.

(2) Drive new steel H-piles.

(3) Construct foundation and stem.

(4) Restore concrete canal bottom.

R. Construct I-wall on East and West sides of Discharge Basin and tie to existing flood protection I-wall.

S. (1) Remove temporary sheet pile dam at Treasure St. and repair concrete lining.

ACCESS ROADS

14. Vehicular Access

Vehicular access to the project site is available via many public streets. Streets adjacent to the site are Abundance St., A. P. Tureaud (formerly London Ave.) and Florida Ave. from the west side, and N. Broad Ave. from the east side. Access to construct the Temporary Sheet Pile Dam may be gained from Treasure St. on the east side and from Florida Ave. on the west side. A temporary earthen ramp will have to be constructed in the Florida Ave. R/W to cross the Norfolk Southern Railroad tracks for access to the canal from the west side. The nearest grade level crossing of the London Ave. Outfall Canal is at the Gentilly Blvd. Bridge, approximately 1300 ft. north of the station.

RELOCATIONS

15. General

Under the authorizing law, local interests are responsible for the accomplishment of "... all necessary alterations and relocations to roads, railroads, pipelines, cables, wharves, drainage structures and other facilities made necessary by the construction work...".

16. Utility Relocation

Where relocated utility lines cross steel sheet piling, steel sleeves will be installed to allow the utility lines to pass through the floodwall. Water tight seals will be placed around the lines. Temporary bypass lines may be required.

A. 48" Diameter Sewer Force Main

The 48" diameter S.F.M. must be relocated twice. It has to be temporarily relocated to the north side of the railroad tracks in order to clear the area where the sluice gate structure will be constructed. This relocation will require approximately 515 feet of pipe (see plate 27).

After the sluice gate structure and concrete discharge tubes are completed, the temporarily relocated S.F.M. must be relocated to the permanent location on the protected side of the sluice gate structure. This final relocation will require approximately 355 feet of pipe (see plate 3).

As mandated by the Sewerage and Water Board, the outage time must be kept to a minimum (under 8 hours).

B. Electrical Feeder Lines

The following Sewerage and Water Board electrical feeder lines will be affected by project construction:

(1) FL-340, FL-400, FL-432, FL-506 & FL-508.

(2) The above feeders will be relocated in either duct banks with blank spares and/or in concrete encased PVC conduit dyed red in accordance with the requirements of the S&WB with respect to relocation and routing.

(3) All new cable will be provided for feeders 400, 340 and 432. Cable shall be 500 MCM lead covered, three conductor, 15 KV, EPR cable to S&WB specification. Feeder 506 will be similar but 750 MCM in size.

(4) Feeders 340 and 508 currently mounted on the North wall of DPS #3 to approximately the midpoint of the station will be routed along the eastern half of the wall to east end of the building, then mounted on the underside of the existing walkway to a point where it will be spliced into existing underground cable. Permanent relocation of Feeders 340 & 508 will be in the sluice gate structure as required by S&WB.

C. Telephone Cable

The existing S.C.B. aerial telephone cable feeding the station will have to be relocated. It currently enters the building on the north wall of the station near Pump C. This line is in direct conflict with the proposed work since it spans over the existing discharge basin.

D. Power Poles

The three (3) existing S&WB power poles which are located in the levee which is to be degraded along the west side of the discharge basin have to be temporarily relocated and eventually removed. The electrical and communication lines which are attached to the poles and span over the railroad tracks shall be rerouted underground to the north side of the railroad tracks.

E. Electrical Transmission Lines

The New Orleans Public Service, Inc. (NOPSI), electrical transmission lines which cross the London Ave. Outfall Canal at Treasure St. shall be de-energized during the construction and dismantling of the Temporary Sheet Pile Dam. The proximity of these electrical lines to the pile driving leads during installation of the sheet piles causes an unsafe condition if the lines remain energized. This work is to be coordinated with NOPSI.

MECHANICAL

17. General

The design of the mechanical system for the fronting protection will include provisions for ten (10) gate assemblies and one (1) electrically operated valve with manual override to flush out the Florida Ave. Canal. The temporary sheet pile dam will also have four (4) electrically operated butterfly gates with manual override to flood the work area prior to priming the pumps.

The design is based on the use of equipment and material that are available as standard industry products. In the selection of equipment, consideration will be given to ease of operation, reliability and ease of maintenance.

18. Sluice Gate Operators

The sluice gates will be individually closed only to prevent backflow when a pump is disabled or a power outage occurs during hurricane or flood conditions. Operation will be by local and remote push button control and indicating lights. Operation of the ten (10) sluice gates will be by individual electric actuators that will require approximately ten (10) minutes per gate to fully close or open. Starting the actuators one at a time and allowing two gates to operate simultaneously will provide a total operating time, from fully open to fully closed, of approximately 50 minutes for all ten gates. Each actuator will be furnished with either a bracket for mounting a portable air motor or an electrical hook up for a portable generator to operate the gates in the event of a power outage. Two portable air motors or one portable generator will be provided.

Limit switches in the actuator's control panel will control the gate's open and closed positions, while torque limiting switches, also in the control panel, will automatically stop the motor if the gate were to encounter an obstruction during its upward or downward motion. Additionally, circuit breakers in the station's electrical control panel will automatically interrupt power to the motor in order to prevent it from developing its locked rotor torque.

19. Vacuum Pumps

Due to the increased volume of the proposed concrete discharge tubes, both existing size 7 vacuum pumps inside the station shall be replaced with new Nash 2002 vacuum pumps. Each pump shall be powered by a 25 Hz motor through a gear box.

Tie-in of new vacuum pumps to existing vacuum lines shall occur in such a way as to not render the remaining two (2) vacuum pumps inoperable. Either existing valves shall be closed or the ends of the lines where they will be cut must be plugged to maintain the vacuum system in an operating condition. Only one vacuum pump can be taken out of service at a time.

ELECTRICAL DESIGN

20. General

The design of the electrical system for the ten gate motors and controllers will include provisions for power and control. The design is based on criteria provided by the Sewerage and Water Board, concerning space conduit routing and power source availability, and on the use of equipment and material that are available as standard products of the electrical industry. Gate operation procedures will require that one gate be operated at a time. In the selection of materials and equipment, consideration will be given to ease of operation, reliability, and ease of maintenance. The Standards of the National Electrical Manufacturers Association (NEMA), the Institute of Electrical and Electronic Engineers (IEEE), and the American National Standards Institute (ANSI) will be used as guides in the selection of electrical equipment. The design of circuits and conduit system will conform to the 1993 National Electrical Code (NEC) and the National Electrical Safety Code.

21. Power Sources & Distribution

A. General

The station power supply for the main pumps is a 6600 Volt, 25 Hz, 3 phase service generated by the Sewerage and Water Board. Lighting and convenience outlets are supplied with the usual 120V, 60 Hz electrical service. The Sewerage and Water Board requests that power to all other motors be maintained as 240 volt, 25 Hz, 3 Ø electrical service.

B. Loads

(1) Vacuum Pumps. Replace two existing Size 7 Nash Vacuum Pumps with two 125 HP Nash Size CL-2002, to operate on 240V, 25 Hz, 3 phase electrical service.

(2) Sluice Gate Operators

(a) Power for the ten (10) sluice gate operators shall come from "T2 Power Panel" inside DPS #3. A spare 100 ampere fusible switch is available for the feeder which will be common to all gate operators.

(b) All gate operators shall be powered by 230V, 3 phase, 25 Hz motors.

(c) Remote control circuits for each operator will be run from the console to each operator in nine conductor trays rated 600V, stranded copper, 90°C, THHN/THWN insulated color coded control cables (3 spare conductors).

(3) Temporary Butterfly Gates

(a) Power for the four (4) butterfly gate operators for the temporary sheet pile dam to be the same as that for the sluice gate operators in Item (2) above.

(b) The butterfly gates shall be remotely controlled from the "T2 Power Panel" inside DPS #3.

(c) Power for the lighting of the service catwalks across the temporary sheet pile dam, shall be 120V, 60 Hz, controlled by a photo-cell contactor arrangement. Lamps will be High Pressure Sodium Vapor, 250 watts, pole mounted.

(4) One (1) Gate Operator will be required for the 4' Ø fresh water (Lake) flush valve. Power supply for the elevated remote operation shall be the same as that for the sluice gate in Item (2) above.

(5) Voltage Drop Requirements

Conductors will be sized to prevent voltage drops from exceeding three (3%) percent at the furtherest utilization point of each circuit.

22. Conduit and Boxes

A. Conduit All above ground and interior wiring to be installed in rigid metal conduit except that motors and other electrical equipment subject to vibration, will be connected with liquid-tight flexible metal conduit.

All conduit buried below grade will be in a steel reinforced red concrete envelope of 3" minimum thickness. In some areas, as requested by the Sewerage and Water Board, feeder cables will be run in concrete duct banks.

B. Pull and Junction Boxes All pull and junction boxes will be of cast metal of sufficient thickness, with bosses to accommodate the required threads for the conduit connectors and meet NEC requirements.

23. Gate Motor Operator Control Pushbuttons

A. Control for all gate motors shall be open/close pushbuttons and end of travel pilot lights.

B. Local control on the operators will include stop-open-close pushbuttons with pilot lights.

C. Remote control will be located on the DPS#3 Auxiliary System Control Console consisting of only open-close pushbuttons and pilot lights for each operator.

HYDROLOGY AND HYDRAULICS

24. General

Design Memorandum No. 19A General Design, London Avenue Outfall Canal, Orleans Parish, presents the hydraulic analysis performed for the London Avenue Outfall Canal to determine the required levee/floodwall height for hurricane protection.

25. Hydraulic Design

Discussions were held with Sewerage and Water Board personnel regarding the recommended plan for fronting protection at Drainage Pumping Station No. 3. The Sewerage and Water Board mandated that the flood protection be accomplished by utilizing a sluice gate structure with concrete discharge tubes similar to the one constructed at Drainage Pumping Station No. 19. The elevations of the top of the Temporary Sheet Pile Dam and the hump inside the concrete discharge tubes were both set by the S&WB, from there many decades of experience in operating DPS#3 and the other pump stations which discharge into canals leading into Lake Pontchartrain.

The S&WB initiated a model study of the pump installation at DPS#19. The one-seventh scale model included a suction basin, pump, concrete discharge tube and sluice gate structure. The results of the model test indicated that the head losses were not significant and agreed with the theoretically calculated head losses. The S&WB agreed to this arrangement and decided to construct DPS#19 utilizing the concrete discharge tubes and sluice gate structure. DPS#19 has been successfully operating since 1991.

GEOLOGY

26. General

A. Scope

The geology presented herein is based on the geology from Design Memorandum No. 19A General Design, London Avenue Outfall Canal (January 1989), which was based on regional, local surface and subsurface information. Additional subsurface information supplemented the data from GDM No. 19A. It is intended to present a general project overview of the pertinent geologic data and interpretation.

B. Physiography and Topography

The project is located within the Central Gulf Coastal Plain region on the flanks of the Mississippi River Deltaic Plain and normal to the Lake Pontchartrain shoreline in Orleans Parish. Pronounced physiographic features of the area are lakes, shorelines, canals, an abandoned Mississippi River delta, the Mississippi River, beach ridges, marshes and swamps. Elevations in the vicinity vary from -15.0 feet NGVD in Lake Pontchartrain to +20.0 feet NGVD along the crown of the mainline Mississippi River levees.

C. Surface Investigation

Aerial photographs, topographic maps, and geologic maps were used in conjunction with published literature to define the geologic setting of the project area.

D. Subsurface Investigation

One 3-inch diameter undisturbed soil boring, 125 feet in depth, was made on 4 August 1994 under A-E contract. An additional two (2) A-E contract borings were made in 1985. The USACE also drilled two (2) undisturbed borings in 1971. Information from all five (5) subsurface investigations was utilized in the analyses. All borings are included on the Soil Boring Profiles (plate 31) in order to present the most geologically complete interpretation. All borings encountered artificial fill and Holocene soils. Those borings exceeding 70 feet generally encountered the Pleistocene horizon. The boring data, used in conjunction with other available data, was the primary source for site specific geologic foundation interpretations.

E. Geophysical Investigation

No geophysical methods were used at the project site. Present refractive methods would not have delineated the various Holocene environments.

27. Regional Geology

Reference Design Memorandum No. 19A General Design, London Avenue Outfall Canal for information on regional geology.

28. Site Geology

A. Site Location and Description

The project site is in Orleans Parish at the southern end of the London Avenue Outfall Canal. A review of the Soil Boring Profiles (plate 31) details geologic structure crossing below the existing discharge basin. Subsurface elevations at the top of Pleistocene average -65 feet NGVD. Depth to top of Pleistocene increases southward from the lakeshore to Drainage Pumping Station No. 3.

Historically, the site stratigraphic sequence indicates a period of aerially exposed Pleistocene prior to an early Holocene marine transgression. Evidence of a gulf water transgression and the subsequent development of the Pontchartrain Embayment is present as a locally extensive basal bay-sound deposit. The clayey bay-sound deposit averages 20 feet in thickness and provides parenting material for the overlying Pine Island Beach trend. Estimated ages of the beach and bay-sound deposits are respectively 5,000 and 7,000 years.

Isolation of the embayment by the eastward prograding Cocodrie Delta (4,600 to 3,500 years before present) marked the end of the marine conditions. Cocodrie aged deposits appear to be absent or obscured in the immediate area. This is possibly a result of two factors: (1) the deltaic material was eroded after abandonment and (2) the remaining material closely resembles the overlying lacustrine and further testing would be necessary to differentiate.

The later prograding St. Bernard Delta, 2,800-1,700 years ago, represented the last major period of active deltaic sedimentation within the area. The surficial marsh deposit was deposited during recent time. West of the project, marsh type deposits are found within the confines of Lake Pontchartrain. This may be evidence of an expanding lake resulting from the shoreline retreat.

The surficial marsh veneer, 5 to 15 feet thick throughout most of the London Avenue Canal, represents the last stage of sedimentation in the area. Marsh type sediments are a result of annual Mississippi River overbank flooding and subsequent deposition of clay and silt size particles landward of the natural levees.

A review of borings in the vicinity of the artificial levee indicates that the additional overburden acts as a surcharge, in some instances consolidating the underlying marsh deposit to less than half the original thickness. Along the centerline of the artificial levee, the additional loading of soil has, to a lesser extent, similarly affected the underlying lacustrine deposit.

B. <u>Detailed Holocene Environmental Descriptions</u>

(1) Bay-sound deposits are fine to coarse grain sediments bottoming bays and sounds. Average thickness is 15 feet in the project area. Reworking of the bottom portion by burrowing marine organisms produces a mottled appearance and inclusions of materials that are distinct from the surrounding sediment. Colors are typically light gray to gray.

(2) Beach deposits are typically fine sands with large quantities of shells and shell fragments. The sands, generally well sorted with few clay lenses, are well suited for founding projects. Subsidence due to soil compaction is relatively minimal. The base elevation of the deposit remains a relatively constant -50 feet NGVD. This deposit is the remnant Pine Island Beach trend.

(3) The marsh deposits are highly compressible organic soils that typically cover 95 percent of the area. They grade vertically downward from peat to organic clays and silts. Generally, soil moistures exceed 100 percent, color varies from light gray to black, and consistencies vary from very soft to medium.

C. <u>Detailed Pleistocene Soil Descriptions</u>

The Pleistocene soils are a result of both deltaic and marine deposition. They represent both the regressive and transgressive phases and associated environments of an earlier Mississippi River deltaic system. The soils are, therefore, similar to the overlying Holocene. However, due to desiccation, Pleistocene deposits are distinguished by a decrease in moisture contents, a stiffening of consistencies, a decrease in sampling penetration rates, an increase in oxidized sediments and the presence of calcareous concretions.

D. Foundation Conditions

Representative geologic site conditions are displayed on cross-sections shown on plate 31. The massive beach deposit has greatly influenced the stratigraphic geometry of the area.

E. Future Investigations

Subsurface field investigations have been completed, and no future investigations are anticipated.

29. Conclusion

Current geologic information indicates generally favorable foundation conditions with regard to future construction. Further addition of fill may result in increased settlement rates, due to marsh soil compaction. Differential settlement may result in areas where organic contents are extremely high and relatively thick. Should future construction in the immediate project vicinity require dewatering, local settlement may occur due to oxidation of organics and consolidation of sediment.

GEOTECHNICAL INVESTIGATION AND DESIGN

30. General

This section includes the geotechnical investigation, description of subsoil conditions and foundation analysis performed for the proposed fronting protection plan at Drainage Pumping Station No. 3 located at the southern end of London Avenue Outfall Canal in New Orleans, Louisiana. The plan consists of I-walls, pile supported T-walls and sluice gate structure and a temporary sheet pile dam.

Analyses and recommendations are based, in part, on data obtained from the soil boring. The nature and extent of variations in subsoil conditions may not become evident until construction. If variations then appear, it will be necessary to reevaluate the recommendations. Conclusions and recommendations are to some degree subjective and should only be used for design purposes. Results of the soil boring and laboratory tests are contained in Appendix A.

31. Previous Geotechnical Investigations

In order to utilize all of the available information at the site, the soil borings and laboratory tests from previous geotechnical investigations by the Department of the Army, New Orleans District, Corps of Engineers (USACE) and Eustis Engineering were used in the analyses. The USACE borings were made in 1971 and are identified as Borings 1-LUW and 2-LUE. Eustis Engineering's borings were made in 1985 and are identified as Borings B-1 and B-36. The boring locations are shown on plate 2 and Figure 1.

The study included a review of the previous geotechnical investigations and the drilling of an additional undisturbed boring, B-1 (1994), to supplement the previous data. Soil mechanics laboratory tests performed on samples obtained from the boring were used to evaluate the physical properties of the subsoils. Engineering analyses, based on all of the available data, were made to determine soil design parameters, lateral earth pressures, pile load capacities in compression and tension for various embedments of steel H-piles, estimates of settlement, and modulus of horizontal subgrade reaction. In addition, analyses were made to determine the maximum bending moment and recommended tip embedment for a temporary cofferdam in the canal and for permanent I-wall structures. Also, analyses were made to determine seepage control measures to control underseepage during high water events.

32. <u>Field Exploration</u>

One A-E undisturbed sample type soil test boring, 125 feet in depth, was made on 4 August 1994 at the location shown on plate 2 and Figure 1. The boring was located at the site using a plot plan furnished by Pepper and Associates, Inc. A detailed descriptive log of the boring is shown in both tabular and graphical form in Appendix A.

The boring was made with a truck mounted rotary type drill rig, and samples of cohesive or semi-cohesive subsoils were obtained at close intervals or changes in stratum using a 3-in. diameter thinwall Shelby tube sampling barrel. Samples were immediately extruded from the sampling barrel, inspected and visually classified by Eustis Engineering's soil technician. Pocket penetrometer tests were performed on the soil samples to give a general indication of their shear strength or consistency and the results of these tests are shown on the boring log under the column headed "PP." Representative samples were placed in moisture proof containers and sealed for preservation.

Samples of cohesionless soil were recovered during the performance of in situ Standard Penetration Tests. This test consists of driving a 2-in. diameter splitspoon sampler 1 foot into the soil after it is first seated 6 inches. A 140-lb weight dropped 30 inches is used to advance the sampler. The number of blows required to drive the sampler 1 foot is recorded and is indicative of the relative density of the subsoils tested. Results of the Standard Penetration Tests are recorded on the boring log under the column headed "SPT." Representative samples obtained from the Standard Penetration Test were sealed in glass jars for preservation of their natural moisture content.

Upon completion of drilling operations, the boring was backfilled with a cement-bentonite grout in accordance with current regulatory requirements.

33. Laboratory Tests

Soil mechanics laboratory tests consisting of natural water content, unit weight, and either unconfined compression shear (UC) or unconsolidated undrained triaxial compression shear (OB) were performed on undisturbed samples obtained from the boring. In addition, Atterberg liquid and plastic limits were performed on selected representative samples to aid in classification of the subsoils and to give an indication of their relative compressibility. The results of the laboratory tests are tabulated on the boring log.

Grain size analyses were performed on three samples of cohesionless soil to determine their particle distribution (PD) curve. The results of these tests are plotted on separate sheets in Appendix A following the boring log.

34. Description of Subsoil Conditions

A. Topography

Ground elevations at the boring locations are referenced to the National Geodetic Vertical Datum (NGVD). On the west side of the canal, Boring 1-LUW is at EL. 3.5 and Boring 1 (1985) is at EL. 4.0. On the east side of canal, Boring 36 is at EL. 10.0 and Boring 2-LUE is at EL. 7.0. At the southern end of the canal, Boring 1 (1994) is at EL. 0.0.

B. Geology

Recent Holocene deposits overlie older Pleistocene deposits. Upper Holocene soils are deltaic plain deposits that overlie nearshore Gulf deposits. Nearshore Gulf deposits interface with the Pleistocene formation.

C. Stratigraphy

(1) Holocene Deposits. Based on the five available soil borings, Holocene deposits can be divided into five distinct strata. The first stratum consists of artificial fill and natural levee deposits to EL. -13 to -17. This stratum is composed predominantly of CH and CL soils. These soils are oxidized and precompressed. The second stratum

contains intradelta deposits of ML, SM and SP soil ranging from EL. -23.5 to -27.5. The third stratum consists of prodeltaic deposits of CH soil to EL. -40 to -43. Deposits to these depths form the deltaic plain. Deltaic plain deposits appear normally consolidated. The deltaic plain is underlain by nearshore Gulf deposits of SP, SM, SC and CL soils to EL. -57 to -62. Beneath this, nearshore Gulf deposits of predominantly CH soil continue to EL. -63.5 to -67.5. Nearshore Gulf deposits appear slightly precompressed.

(2) Pleistocene. The geologically identified Pleistocene formation begins at EL. -63.5 to -67.5. These soils are precompressed and consist predominantly of CH and CL soil with isolated strata of ML and SP soil. Surficial Pleistocene deposits are oxidized to EL.-88.5. Pleistocene deposits continue to the final boring depths of 75 to 125 feet below the existing ground surface (EL. -71.5 to -125).

D. Groundwater

Observations of the groundwater were made during the field investigation on 4 August 1994. An auger boring, located 12 feet east of Boring 1, was made without the addition of water to a depth of 12 feet. After an elapsed period of nine hours, the depth to groundwater was measured to be 6 feet below the existing ground surface (approximately EL. -6.0). The depth to groundwater will vary with climatic conditions, drainage improvements, fluctuations of the water level in the canal and other factors. The depth to groundwater should be determined by those persons responsible for construction immediately prior to beginning work.

35. Foundation Analysis

A. Furnished Information

A temporary sheet pile dam with four (4) butterfly gates will be constructed across the canal at Treasure St. to provide a dewatered work area to construct the fronting protection. The existing discharge pipes will be extended approximately 107 feet to the north and a sluice gate structure will be placed at the northern end of the concrete discharge tubes to form a permanent barrier across the canal. A 25 ft. long portion of the sluice gate structure will have a T-wall monolith between discharge Pump A and discharge Pump C. The east and west ends of the sluice gates will tie into T-wall structures running north and then into I-wall structures to the Norfolk Southern Railroad embankment. Low water level in the canal is EL. -1 and hurricane level is EL. 13.9. The bottom of the discharge basin is at EL. -9.18.

B. Soil Design Parameters

Soil shear strengths and unit weights from the five borings were plotted versus elevation to develop soil design parameters for the project. A total of 59 shear tests was utilized from the borings. These included 30 unconfined compression shear (UC) tests, 12 unconsolidated undrained triaxial compression shear 1-point (OB) tests, 12 unconsolidated undrained triaxial compression shear 3-point (Q) tests, 4 consolidated drained direct shear (S) tests and, 1 consolidated undrained triaxial compression shear (R) test. The soil design parameters are tabulated on Figure 2.

C. Lateral Earth Pressures

At Rest Pressures. Analyses were made to determine the lateral earth pressures acting on pile supported concrete walls below ground. Lateral pressures on buried structures should be determined using at rest lateral earth pressure coefficients. The lateral earth pressure coefficient (K_o) is 0.55 for granular sand backfill and 1.0 for in situ clay soils. For granular sand backfill, a design lateral earth pressure of 95 psf per linear foot of depth is recommended. For clay backfill, a design lateral earth pressure of 110 psf per linear foot of depth is recommended. These values include the effects of soil and water acting on the walls.

D. Pile Foundations

(1) Estimated Pile Load Capacities. Furnished information indicates that the proposed structures will be supported by 14-in. steel H-piles driven from EL. -10. Pile load capacity curves in compression and tension are plotted on Figure 3. The analyses include an estimated factor of safety of 2 against a soil shear failure.

(2) Batter Piles. The estimated pile load capacities shown on Figure 3 are for piles driven vertically and may be used to determine the pile load capacity for batter piles. The vertical capacity will be equal to the vertical component of a batter pile driven to the same tip elevation. From this relationship, geometry may be used to determine the

axial capacity and horizontal component of the batter piles. This method is shown in more detail on Figure 4.

(3) Structural Capacity. The estimated pile load capacities are based on a soil-pile relationship only. The structural capability of the individual piles to transmit these loads and any connections between the piles and the structure, especially in tension, should be determined by the structural engineer.

(4) Pile Group Capacity and Spacing. Furnished information indicates a 60-ton design load capacity will be used for construction. This will require piles being driven to a tip of EL. -77. Piles driven to this tip elevation will derive their supporting capacity primarily through skin friction, and it will be necessary to consider the effect of group action for piles driven in groups. In this regard, the supporting value of the friction piles driven in groups should be investigated on the basis of group perimeter shear by the formula shown on Figure 5. For pile groups used in tension, the second term of the formula is deleted. The minimum center to center pile spacing within a pile group or row of piles should be determined in accordance with the pile spacing formula also shown on Figure 5.

(5) Pile Driving. A daily driving record should be kept for all piles. The driving record should include the date, type and size of pile, length and embedment of pile, hammer make and model, driving energy and number of blows per foot of penetration. An accurate driving record is especially important to verify the piles are installed to the required tip embedment and to give an indication of any unusual driving characteristics which may indicate pile damage.

USACE specifications usually require a hammer having striking parts that weigh at least 67% of the weight of the driven pile. Steel H-piles can be driven with a single acting air hammer developing 19,500 ft-lbs of energy per blow. This hammer is recommended for a pile with a 60-ton allowable compressive capacity.

(6) Dynamic Pile Test (DPT). The steel H-piles should have a cross section which is structurally sufficient to facilitate driving of the piles

without damage. Driving stresses and drivability of the piles with the selected hammer and appurtenant driving equipment should be evaluated by dynamic analysis (WEAP). Structural requirements can then be verified by the structural engineer and installation criteria can be established.

DPT can be performed with a pile driving analyzer (PDA) on steel H-piles to evaluate their capacity during and after installation. A PDA can monitor driving stresses during installation, evaluate the static capacity and evaluate pile integrity during or after installation. A PDA can also monitor energy transferred to the pile by the hammer to evaluate installation efficiency. Data obtained with a PDA should be evaluated by a geotechnical engineer familiar with the subsurface conditions in order to properly interpret PDA information and make appropriate recommendations.

(7) Vibrations. Pile driving will cause vibrations which may affect nearby structures, pavements and underground utilities. It is recommended that peak particle velocities due to pile driving be monitored at critical structures or pavements with a seismograph during all pile driving operations. The record of peak particle velocities will provide information in assessing potential damage and the need for changes in the driving operations.

Peak particle velocities of 0.25 of an inch per second as measured by the seismograph are generally regarded as a vibration level uncomfortable to human perception. Peak particle velocities in excess of 0.5 of an inch per second (measured at a structure) may induce damage to the structure. Therefore, for sustained peak particle velocities in excess of 0.25 of an inch per second at a pavement or structure of concern, Eustis Engineering should be notified. If peak particle velocities reach 0.5 of an inch per second, pile driving operations should be terminated and consideration should be given to altering installation criteria.

(8) Test Piles and Pile Load Test. A test pile should be installed within the excavation cofferdam. The test pile program can be used to establish installation criteria for the job piles and will give an indication of the driving resistance and vibrations. The test pile should be allowed to

"set" for at least 28 days after driving, and then should be load tested to failure in accordance with the New Orleans Building Code. If DPT is considered for job pile evaluation, pile load tests should be coordinated with DPT to establish relationships between dynamic and static tests.

Alternately, a test pile program outside of the excavation may be considered because of construction time constraints.

(9) Estimated Settlement. For pile foundations embedded in the underlying Pleistocene formation at tip EL. -77, it is estimated that settlement of the sluice gate structure and T-walls will be ¼ to ¾ of an inch. This estimate of settlement does not include the elastic deformation of the piles or settlement due to the placement of fill near pile foundations. This estimate of settlement is based on the assumption that the foundation design will utilize single rows of piles on relatively wide spacings of 8 to 10 feet with 3 to 4 feet between piles in each row. Small isolated pile groups with two to three piles per group have also been assumed. The minimum center to center spacing between pile groups should be no closer than two times the largest group dimension. All piles used for construction should be driven to the same tip elevation in order to minimize differential settlement. If final plans differ from these assumptions, additional settlement analyses should be performed.

(10) Subgrade Moduli. Analyses were made to estimate the modulus of horizontal subgrade reaction for laterally loaded piles. The modulus of horizontal subgrade reaction has been estimated at between EL. -10 and -90. Results are plotted on Figure 6. The modulus of horizontal subgrade reaction will be influenced by the width of the pile and the spacing of piles perpendicular to the lateral load.

E. Temporary Dam Across Canal

(1) Design Conditions. Furnished information indicates a temporary cantilevered sheetpile dam will be constructed across London Avenue Canal at Treasure Street. The top of the dam will be at sill EL. 1.57 NGVD and the dam will have four (4) butterfly gates. The purposed of the dam is to provide a dewatered working area between the dam and Drainage Pumping Station No. 3 for construction of the

fronting protection. During operating conditions at the pumping station, the butterfly gates will be opened to flood the work area. This will allow water in the canal to flow over and through the dam toward Lake Pontchartrain.

(2) Stability. Analyses for the temporary dam were made using the Corps' program "CWALSHT" and Q-case soil conditions. The analyses assume a horizontal ground surface at EL. -10 on both sides of the dam. The water surface was assumed at EL. 1.57 on the flood side and EL. -10 on the protected side. The results show a maximum bending moment of 67,283 foot-pounds occurs at EL. -25.46 using a factor of safety of 1.0 applied to the soil shear strengths. Using a factor of safety of 1.5, the sheetpile wall for the temporary dam should be driven to tip EL. -54.52. Values of shear, moment and deflection are tabulated on the computer printouts in Appendix B.

(3) Dewatering and Pressure Relief. The analyses assume hydrostatic pressures on the cohesionless intradeltaic deposits occurring between EL. -13 and -27.5 do not exceed EL. -15. Hydrostatic pressures in the cohesionless nearshore Gulf deposits between EL. -40 and -62 are assumed not to exceed EL. 4.5. In order to achieve these hydrostatic pressures, it will be necessary to install a dewatering and hyrostatic pressure relief system.

The pressure relief system should be comprised of a series of wells or wellpoints capable of lowering the hydrostatic heads to the levels assumed in the analyses. The system should be designed and installed by a dewatering and pressure relief contractor experienced in pressure relief installation. The recommended system should be reviewed for adequacy by a representative of the owner.

It should be noted that prolonged operation of the dewatering and pressure relief system may cause settlement of the adjacent ground surface and structures. Therefore, operation of the system should be minimized by expeditious construction.

F. Temporary Cofferdams at Discharge Tubes

(1) Design Conditions. Furnished information indicates the discharge basin adjacent to Drainage Pumping Station No. 3 will be dewatered for construction of fronting protection across the full width of the

canal. The Sewerage & Water Board of New Orleans (S&WB) requires the pumping station to be operational during specified weather events and that the discharge basin be flooded within 15 minutes to restore pumping capacity at this station.

After the sluice gates and T-walls have been installed across the canal, the suggested sequence of construction indicates steel sheeting will be driven on the east and west sides of Pump C to allow the concrete discharge tube to be built in the dry. The S&WB will only allow one pump to be taken out of service at a time. This will require a separate cofferdam for Pumps C and D. Cofferdams are not required at the other pumps since the sluice gates can be closed for protection in these areas and the discharge tubes will already be in place at Pumps C and D.

(2) Stability. Analyses were made for a cantilevered sheetpile wall using Q-case soil conditions and the Corps' "CWALSHT" program. The bottom of the cofferdam excavation was assumed at EL. -11.28 and the water on the flood side was assumed at EL. 1.57. Using a factor of safety of 1.0 applied to the soil shear strengths, the maximum bending moment is 89,333 foot-pounds and occurs at EL. -28.06. A factor of safety of 1.5 was applied to the soil shear strengths to determine the top embedment. The analyses indicate sheetpiles for the cofferdam should be installed to EL. -61.37 to provide an adequate factor of safety against failure by rotation. Computer printouts of the analyses are included in Appendix B.

G. I-Wall Structure

(1) Stability. A limited length of I-wall will be constructed on both sides of the canal between the railroad embankment and T-wall structure. The horizontal ground line on both sides of the I-wall was furnished at EL. 8.57. The still water level (SWL) or flowline was furnished at EL. 11.9. The flowline plus 2 feet of freeboard will result in EL. 13.9. The top of wall will be constructed to EL. 14.4 to account for future settlement.

Based on criteria developed by the USACE, several analyses were performed to determine the required tip penetration and pressure diagram. A summary of the analyses is shown in Appendix C together with a flow chart developed by the USACE. In addition, the computer output for the program "CWALSHT" for the design condition is included. Results indicate the sheetpile wall should be installed to tip EL. -0.80. The maximum bending moment is 2,398 ft-lbs. Shear, moment and deflection information is also included in Appendix C.

(2) Seepage Control. Analyses were made to determine the recommended sheetpile penetration for seepage cut-off beneath the T-wall and sluice gate structure. Using Harr's method, it is recommended that a 25-ft sheetpile cutoff be utilized which will provide a factor of safety of at least 4 against piping. With the top of the monolith slab at EL. -9.18, this will result in a tip at EL. -34.18. Based on Lane's weighted creep ratio, this tip elevation will provide a creep ratio of 4 which is adequate for soft to medium stiff clays.

H. Documentation of Existing Conditions

Installation of piles and sheetpiles and operation of the dewatering and pressure relief system may cause vibrations and settlement that could adversely effect adjacent structures or utilities. It is highly recommended that a program be undertaken to document the conditions of existing structures and utilities prior to construction. Documentation should be a photographic and video tape record by a registered civil engineer.

I. Stability Analyses

The stability of the T-wall structures at Station 0+62 to 0+87 and Station 1+57 to 2+07 was determined using the method of planes and design soil parameters shown on Figure 2. The USACE program, "Stability with Uplift," was used for the analyses. Failure conditions toward the canal during low water, EL. -1.0, and toward the protected side during high water, EL. 11.9 were analyzed. The analyses indicate the most critical condition occurs during low water. A factor of safety of 1.31 occurs for the T-wall structure at Station 0+62 and 0+87 between discharge tubes A and C. For the T-wall structure at Station 1 + 57 to 2+07, the analyses indicate a factor of safety of 1.66 during low water. These factors of safety are considered adequate for the structures. Results of the stability analyses are shown on Figures 7 and 8.

SOURCES OF CONSTRUCTION MATERIALS

36. Concrete

A. Description

The project plan consists of constructing a sluice gate structure across the entire width of the London Avenue Outfall Canal which will connect to the recently upgraded floodwalls on both sides of the canal. This protection will incorporate the use of I-walls and T-walls in addition to the sluice gate structure.

(1) A sluice gate structure will be placed in front of the discharge area for the five (5) existing horizontal pumps. Each pump will have an individual concrete discharge tube connecting it to the sluice gate structure. These structures will be reinforced concrete, founded on steel H-piles with steel sheet pile seepage cut-off.

(2) T-wall monoliths will adjoin the gated monoliths. These monoliths will be inverted T-type reinforced concrete structures, founded on steel H-piles with steel sheet pile seepage cut-off.

(3) I-type floodwalls consisting of steel sheet piles capped with a reinforced concrete wall will tie the existing I-walls to T-walls on each end of the sluice gate structure.

B. Location

The Orleans Parish Outfall Canals of Lake Pontchartrain, Louisiana and Vicinity Hurricane Protection are located in southeastern Louisiana on the south side of Lake Pontchartrain in Orleans Parish. There are three (3) outfall canals which transport storm water drainage from the major urbanized areas of Orleans Parish on the east bank of the Mississippi River. The London Avenue Outfall Canal lies to the east of the 17th Street and Orleans Avenue Canals. The three canals run parallel to each other and are oriented in the north/south direction. Drainage Pumping Station No. 3 is located at the southern end of the London Avenue Outfall where it commences at approximate Station -0+27.

C. <u>Concrete Investigation</u>

(1) Concrete quantities and qualities.

Structural Feature	Concrete Quantity (Cu.Yds.)	Compressive Strength (28*days,psi)	Nominal Max** Size Aggregate (Inches)	Air*** Content (percent)
Stab.Slab, Unreinforced	60	2000	1.5	4 to 7
6" Paving, Reinforced	1350	3000	1.5	4 to 7
Monoliths, Reinforced	4850	4000	1.5	4 to 7

* 90 days if pozzolan used
** smaller sizes may be used if economically justified
*** depends on Nominal Maximum Size Aggregate (NMSA) also 4 to 7 percent for 1-inch NMSA

Based on service and environment conditions, a water-cement ratio of 0.58 will not be exceeded for durability requirements. The slump will range from 1 to 4 inches.

(2) <u>Environmental conditions.</u> The concrete will not be subjected to any critical environmental or functional conditions.

D. <u>Cementitious Materials Investigation</u>

(1) <u>Cement</u>

(a) <u>Special requirements</u>. Because of the nature of local aggregates, low alkali cement must be used. False set requirements will be necessary if an on-site batch plant is used, however a local ready mix plant will likely be chosen by the Contractor.

(b) <u>Availability</u>. Cement meeting Type I or II requirements of ASTM C 150 in addition to the above special requirements is locally available from Citadel Cement, LaFarge Co., Dundee Holnam Cement Co., Louisiana Industries, and others.

(c) Type and justification. Because of the availability of Type II cement at no additional cost and lower heat of hydration, Type II cement will be specified.

(d) Testing requirements. Testing requirements of CW-03301, paragraph 3.1.2.3 will be imposed in the specifications in lieu of paragraph 5.1.2.

(2) Pozzolan. Fly ash meeting the requirements of ASTM C 618, Types C or F, including the optional chemical and physical requirements 1A and 2A, respectively, will be allowed. The percentage of fly ash in the Contractor's furnished mix design will be limited to not greater than 35 percent of absolute volume. Its recommended use is based on potential cost savings. Also using fly ash could potentially reduce heat of hydration and permeability, and improve sulfate resistance. Type C fly ash obtained from Bayou Ash was satisfactorily used on the Old River Control Auxiliary Structure and is currently being satisfactorily used in the production of articulated concrete mats at St. Francisville, LA. Bayou Ash is located near New Roads, LA, approximately 120 miles from New Orleans, LA.

E. Aggregate Investigation

(1) Sand and gravel. The sources listed in Table 1 are a few of the area companies on the USACE pretested list that seem capable of furnishing sand and gravel for the project.

Test reports can be found in TM 6-370 and Old River Control, LA, Auxiliary Structure Sources of Construction Materials, DM No. 14 dated 3 Oct 80. Transportation of aggregates would probably be by truck, except for Lambert Gravel which has also indicated that barging from their source is possible.

TABLE 1

Company Name	Nearest Town (LA)	Project to Pit Distance (Miles)	Pit Location Lat (deg)	Pit Location Long (deg)	TM 6-370 Vol/Area	TM 6-370 Index Number
Lambert Gravel	Bains	130	30	91	4A/9A	1
La.Industries	Enon	70	30	90	4A/9A	9
Rebel Sand & Gravel	Watson	102	30	90	3A/7A	16
Standard Gravel	Enon	70	30	90	4A/9A	28
T.L. James & Co.	Pearl River	45	30	89	4A/9A	11

F. Concrete Batch Plant And Truck Mixer Investigation

(1) On-site batch plant. The largest single concrete placement appears to be the discharge tube base slab for pumps A and B which is approximately 420 cubic yards. The concrete batch plant needs to have a capacity of at least 75 cubic yards per hour in order to prevent cold joints during placement.

(2) Off-site batch plant. Ready mix concrete meeting the requirements of this project and produced from batch plants meeting the guidelines of Cast-in-place Structural Concrete (CW-03301) can be obtained from the sources listed in Table 2:

TABLE 2

Company Name	Distance to Project (miles)	Plant Capacity (CY/HR)	Plant Type	Number of Truck Mixers	Cooling Method
La. Indus. (Plant 4) (Euphrosine St.)	5	100	Semi	23	ice
LaFarge (Airline Hwy)	20	180	Auto	52	ice or chilled water
Carlo Ditta (S.Peter St.)	10	120	Auto	36	ice
Peter Judlin (Old Gentilly Rd.)	7	100	Auto	18	ice

G. Thermal Considerations. The largest single concrete placement will be the 3.25-foot thick discharge tube base slab for pumps A and B. Its volume is approximately 420 cubic yards. The placing temperature of the base slab concrete will not be allowed to exceed 85 degrees F, while for other elements, the maximum will be 90 degrees F.

ENVIRONMENTAL

37. General

The London Avenue Outfall Canal is a man-made canal approximately 4.0 miles in length, with an average bottom and top width of 100 to 160, respectively. Drainage Pumping Station No. 3 lies at the head of the canal near N. Broad Avenue. The canal is paralleled by earthen levees topped with floodwalls or floodwalls alone from Drainage Pumping Station No. 3 to Leon C. Simon Boulevard on the east and to Robert E. Lee Boulevard on the West. From these two boulevards to Lakefront Drive there is an earthen levee on both sides of the canal.

38. Existing Conditions

Water quality in the canal is generally poor and normally exceeds criteria for propagation of fish and wildlife. The canal provides minimal value as habitat for fishery resources.

Fishing is primarily limited to the lakefront area.

Esthetics is generally poor due to the poorly maintained areas around the pumping station and the condition of the floodwall.

No cultural resources or endangered species are recorded in the vicinity of the proposed work.

Noise levels in the area are within the range expected for residential areas. Residents in the project area will not be displaced by the construction work.

39. Environmental Effects

The ambient noise level would be increased during construction with some residences close to the construction site experiencing noise levels that could interfere with sleeping, conversation and some recreational activities. These levels will occur only for the period of construction and will be limited to daylight hours. There will be some temporary disruption in normal traffic patterns during construction, but will be limited again to daylight hours. No displacement of residences will be necessary.

40. Environmental Compliance

The final Environmental Impact Statement (EIS), for Lake Pontchartrain, Louisiana, and Vicinity Hurricane Protection Project, was filed with the President's Council on Environmental Quality on 17 January 1975. A Final Supplement to this EIS was filed with the Environmental Protection Agency (EPA) in December of 1984. The Final Supplement assessed the impacts associated with increased levee height for a high level plan of protection.

The impacts of providing protection along the outfall canals were not addressed in the original EIS or the subsequent supplement. However, an Environmental Assessment (EA), addressing the impacts associated with providing hurricane induced flood protection, for the London Avenue Canal, was prepared on 7 October 1988. Based on this EA, a determination was made that the hurricane protection provided along this canal would not have a significant impact upon the human environment. A Finding of No Significant Impact (FONSI) was signed 27 October 1988. This completes the environmental compliance for construction of this feature.

ESTIMATE OF COST

41. General

Based on March, 1996 price levels, the estimated first cost for constructing the fronting protection at Pumping Station No. 3 is $9,843,882.00. Engineering and Design and Construction Management are estimated to be $931,477.00 and $1,080,767.00. Table 3 presents the itemized first cost for the fronting protection at Drainage Pumping Station No. 3.

TABLE 3

DPS NO. 3
FRONTING PROTECTION
ESTIMATED CONSTRUCTION COST ESTIMATE

March, 1996 Level

ITEM NO.	DESCRIPTION OF ITEM	UNIT	QUANTITY	UNIT PRICE	AMOUNT
1.	Mobilization	Lump	L.S.	$324,252	$ 324,252.00
2.	Backfill	C.Y.	4,000	$15.00	$ 60,000.00
3.	Temporary Sheet Pile Dam	Lump	L.S.	$500,000	$ 500,000.00
4.	Removal of Concrete Lining	Lump	L.S.	$160,000	$ 160,000.00
5.	Temporary Steel Sheeting	S.F.	12,000	$16.00	$ 192,000.00
6.	Excavation	C.Y.	1,000	$6.00	$ 6,000.00
7.	Pile Load Test	Each	2	$20,000	$ 40,000.00
8.	Reinforced Concrete Paving 6"	Sq.Yd.	1,350	$32.00	$ 43,200.00
9.	Sheet Piling Cut Off Wall	S.F.	4,796	$20.00	$ 95,920.00
10.	Foundation Piling HP 14 x 73	L.F.	23,096	$27.00	$ 623,592.00
11.	Concrete	C.Y.	4,828	$400.00	$1,931,200.00

ITEM NO.	DESCRIPTION OF ITEM	UNIT	QUANTITY	UNIT PRICE	AMOUNT
12.	Reinforcing Steel	Lbs.	965,600	$0.65	$ 627,640.00
13.	Structural Steel	Lbs.	13,456	$1.25	$ 16,820.00
14.	Groundwater Drainage	Lump	L.S.	$250,000	$ 250,000.00
15.	Roadway Work	Lump	L.S.	$80,000	$ 80,000.00
16.	Detour Signs, Barricades	Lump	L.S.	$15,000	$ 15,000.00
17.	<u>UTILITIES:</u>				
a)	S&WB Elec. Manhole	Each	2	$6,000	$ 12,000.00
b)	Relocation of S&WB's Elec. Feeder Cables	L.F.	500	$100	$ 50,000.00
c)	Temporary Relocation of 48"Ø S.F.M.	L.F.	515	$350	$ 180,250.00
d)	Permanent Relocation of 48"Ø S.F.M.	L.F.	355	$300	$ 106,500.00
18.	Aluminum Handrail & Posts	L.F.	550	$25	$ 13,750.00
19.	Steel Grating	S.F.	940	$20	$ 18,800.00
20.	Galvanizing Charge	Lbs.	25,126	$0.30	$ 7,538.00
21.	48"Ø Flush Pipe	L.F.	120	$100	$ 12,000.00
22.	2 - Steel Pipes 11'-9" I.D.	Lbs.	70,219	$1.00	$ 70,000.00
23.	3-Reducers 13'-4" to 11'-9"	Lbs.	85,654	$1.80	$ 154,177.00

ITEM NO.	DESCRIPTION OF ITEM	UNIT	QUANTITY	UNIT PRICE	AMOUNT
24.	Bellows or Dresser Couplings	Each	10	$2,000	$ 20,000.00
25.	Aluminum M.H. w/ladder	Each	5	$3,000	$ 15,000.00
26.	81" x 96" Sluice Gates	Each	4	$75,000	$ 300,000.00
27.	108" x 96" Sluice Gates	Each	6	$95,000	$ 570,000.00
28.	Elec. Motor Operator	Each	10	$3,000	$ 30,000.00
29.	Stems	Each	10	$1,500	$ 15,000.00
30.	Vacuum Pump Upgrade-2	Each	2	$80,000	$ 160,000.00
31.	Electrical Controls (Interior)	Lump	L.S.	$80,000	$ 80,000.00
	SUB-TOTAL				$6,780,639.00
	10% CONTINGENCIES				$ 678,064.00
					$7,458,703.00
	INFLATION 5%				$ 372,935.00
FRONTING PROTECTION TOTAL					$7,831,638.00

ENGINEERING AND DESIGN:		
COST OF DESIGN MEMORANDUM	$ 305,690.00	
COST OF PLANS & SPECIFICATIONS	$ 464,126.00	
SUB-TOTAL	$ 769,816.00	
ENGINEERING DURING CONSTRUCTION	$ 76,982.00	
SUB-TOTAL	$ 846,798.00	
10% CONTINGENCIES	$ 84,679.00	
E & D TOTAL	$ 931,477.00	
CONSTRUCTION MANAGEMENT & TESTING:		
LUMP SUM	$ 939,797.00	
15% CONTINGENCIES	$ 140,970.00	
CM TOTAL	$1,080,767.00	
GRAND TOTAL		$9,843,882.00

42. Schedule for Design and Construction. The sequence for design and construction is shown in Table 4.

TABLE 4
SCHEDULE FOR DESIGN AND CONSTRUCTION

	DESIGN		CONSTRUCTION		
ACTIVITY	START	COMPLETE	ADVER.	AWARD	COMPLETE
P&S	Jan.95	Dec.95	Jan.96	March 96	March 98

43. Federal and Non-Federal Cost Breakdown. The breakdown of Federal and non-Federal costs needed to construct the Fronting Protection at Drainage Pumping Station No. 3 described in Supplement No. 2 to GDM 19A is shown in Table 5 below:

TABLE 5
FEDERAL AND NON-FEDERAL COST BREAKDOWN
MARCH 1996 PRICE LEVELS

Item	Federal	Non-Federal	Total
Relocations and Fronting Protection	$6,890,717	$2,953,165	$9,843,882

44. Non-Project Related Estimated Costs

The S&WB has requested that various non-project related improvements be performed at the station and site while the Contractor is on the site. The breakdown of items and estimated costs are shown on Table 6 below:

TABLE 6

NON-PROJECT RELATED ESTIMATED COSTS
ITEMS S1 - S4

ITEM NO.	DESCRIPTION OF ITEM	UNIT	QUANTITY	UNIT PRICE	AMOUNT
S-1	Concrete Deck over Suction Basin	Lump	L.S.	$50,000	$ 50,000.00
S-2	Roll-Up Shutters	Lump	L.S.	$10,000	$ 10,000.00
S-3	Forced Ventilation	Lump	L.S.	$15,000	$ 15,000.00
S-4	Modifications to Marigny Ave. Canal	Lump	L.S.	$50,000	$ 50,000.00
			SUB-TOTAL		$ 125,000.00
			CONTINGENCY 10%		$ 12,500.00
					$ 137,500.00
			INFLATION 5%		$ 6,875.00
			TOTAL		$ 144,375.00

OPERATIONS AND MAINTENANCE

45. General. All operations and maintenance (O&M) costs for this project will be S&WB responsibility. The estimated O&M costs are shown in Table 7 below:

TABLE 7

OPERATIONS AND MAINTENANCE

Item	Annual Cost*
Sluice Gate Maintenance	$4,600
Gated Monolith Maintenance	$1,500
I/T Wall Maintenance	$2,200
Subtotal	$8,300
Contingency	$1,245
TOTAL	$9,545

*The above annual cost estimates do not include replacement costs or increases due to inflation.

46. Funds Required by Fiscal Year. To maintain the schedule for design and construction for the Fronting Protection at Drainage Pumping Station No. 3 as shown in Table 4, funds will be required by fiscal year as shown in Table 8 below:

TABLE 8

TOTAL FEDERAL AND NON-FEDERAL FUNDING BY FISCAL YEAR

FY 95	$ 846,798.00
FY 96	$1,181,266.00
FY 97	$6,595,401.00
FY 98	$1,220,417.00
TOTAL	$9,843,882.00

47. Recommendation. The plan of improvement recommended herein calls for construction of a sluice gate structure across entire width of canal just north of Drainage Pumping Station No. 3 incorporating the use of I-walls and T-walls. New concrete discharge tubes will connect the sluice gate structure to the individual pump discharges. The plan of improvement presented in this supplemental design memorandum is to sufficient detail to proceed to plans and specifications. Approval of this supplemental design memorandum is recommended.

APPENDIX A

LEGEND AND NOTES FOR LOG OF BORING AND TEST RESULTS

PP Pocket penetrometer resistance in tons per square foot

TV Torvane shear strength in tons per square foot

SPT Standard Penetration Test. Number of blows of a 140-lb. hammer dropped 30 inches required to drive 2-in O.D., 1.4-in. I.D. sampler a distance of one foot into the soil, after first seating it 6 inches

SPLR Type of Sampling   Shelby   SPT   Auger   No Sample

SYMBOL Clay   Silt   Sand   Humus   Predominant type shown heavy; Modifying type shown light

DENSITY Unit weight in pounds per cubic foot

USC Unified Soil Classification

TYPE

UC	Unconfined compression shear
OB	Unconsolidated undrained triaxial compression shear on one specimen confined at the approximate overburden pressure
UU	Unconsolidated undrained triaxial compression shear
CU	Consolidated undrained triaxial compression shear
DS	Direct-shear
CON	Consolidation
PD	Particle size distribution
k	Coefficient of permeability in centimeters per second
SP	Swelling pressure in pounds per square foot

ϕ Angle of internal friction in degrees

c Cohesion in pounds per square foot

Other laboratory test results reported on separate figure

Ground Water Measurements   Initial   Final

GENERAL NOTES

(1) At the time the borings were made, ground water levels were measured below existing ground surface. These observations are shown on the boring logs. However, ground water levels may vary due to seasonal and other factors. If important to construction, the depth to ground water should be determined by those persons responsible for construction, immediately prior to beginning work.

(2) While the individual logs of borings are considered to be representative of subsurface conditions at their respective locations on the dates shown, it is not warranted that they are representative of subsurface conditions at other locations and times.

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

LONDON AVENUE OUTFALL CANAL, FRONTAL PROTECTION AT PUMPING STATION NO. 3 (SHEET 1 Of 3)
NEW ORLEANS, LOUISIANA

Ground Elev.: Datum: Gr. Water Depth: See Text Job No.: 13065 Date Drilled: 8/04/94 Boring: 1 Refer To "Legends & Notes"

Scale In Feet	PP	SPT	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density Dry	Density Wet	Shear Tests Type	Ø	C	Atterberg Limits LL	PL	PI		Other Tests
	0.90		Medium stiff gray silty clay w/shells, sand, brick fragments & roots	CL	1	0-1											
			Decayed wood	Wd	2	1-2											
	1.20		Stiff gray & tan silty clay w/decayed wood, sand, organic matter & brick fragments	CL	3	2-3	32	89	117	UC	--	1320					
5	1.20		Medium stiff tan & gray clay w/organic matter & trace of silt	CH	4	5-6	36	84	114	UC	--	735					
	0.40				5	8-9	42	78	110	UC	--	735					
10	0.20		Loose gray clayey silt w/organic matter & roots	ML	6	11-12	33	90	119	OB	--	340					
15	0.20		Medium stiff gray clay w/organic matter	CH	7	14-15	103	43	88	UC	--	505					
	0.30		Very loose gray clayey silt w/clay layers	ML	8	18-19	43	77	110	OB	--	210					
20		19	Medium dense gray fine sand	SP	9	21-22											
25		7	Medium stiff gray clay w/silt layers	CH	10	24-25											
		5			11	27-28	59										
30																	
35	0.20		Soft gray clay w/silt lenses	CH	12	33-34	68	60	100	UC	--	380	95	28	67		
40	0.20		Soft dark gray clay	CH	13	38-39	69	59	100	UC	--	495					
45			Soft gray sandy clay w/clayey sand layers & shell fragments	CL	14	43-44	33	89	119	OB	--	340					
			w/shell fragments		15	48-49											
50			Medium dense gray silty sand	SM													

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

LONDON AVENUE OUTFALL CANAL, FRONTAL PROTECTION AT PUMPING STATION NO. 3

NEW ORLEANS, LOUISIANA

Ground Elev.: Datum: Gr. Water Depth: See Text Job No.: 13065 Date Drilled: 8/04/94 Boring: 1 Refer To "Legends & Notes"

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density		Shear Tests			Atterberg Limits			% Passing No. 200 Sieve	Other Tests
										Dry	Wet	Type	Ø	C	LL	PL	PI		
		25			Medium dense gray silty sand	SM	16	51-52											PD
55		13					17	54-55											
60		6			Medium stiff gray clay w/silt layers	CH	18	59-60											
					Soft gray clay w/shell fragments & silty sand layers	CH	19	63-64	52	73	111	OB	--	485	85	26	59		
65					Medium stiff gray & tan silty clay w/fine sand layers	CL													
							20	68-69	27										
70																			
	2.20				Stiff greenish-gray & tan silty clay	CL	21	73-74	28	95	121	OB	--	1665					
75					Stiff greenish-gray & tan clay	CH													
	2.80						22	78-79	39	82	114	UC	--	1655					
80																			
	1.25						23	83-84	36	88	119	UC	--	1325	57	21	36		
85																			
	2.20						24	87-88	33	89	118	UC	--	1205					
90		40			Compact tan & gray clayey silt	ML	25	89-90										88.5	
95		19			Medium dense greenish-gray fine sand	SP	26	94-95											
100		10			Stiff gray clay	CH	27	99-100	50										

EUSTIS ENGINEERING COMPANY, INC.

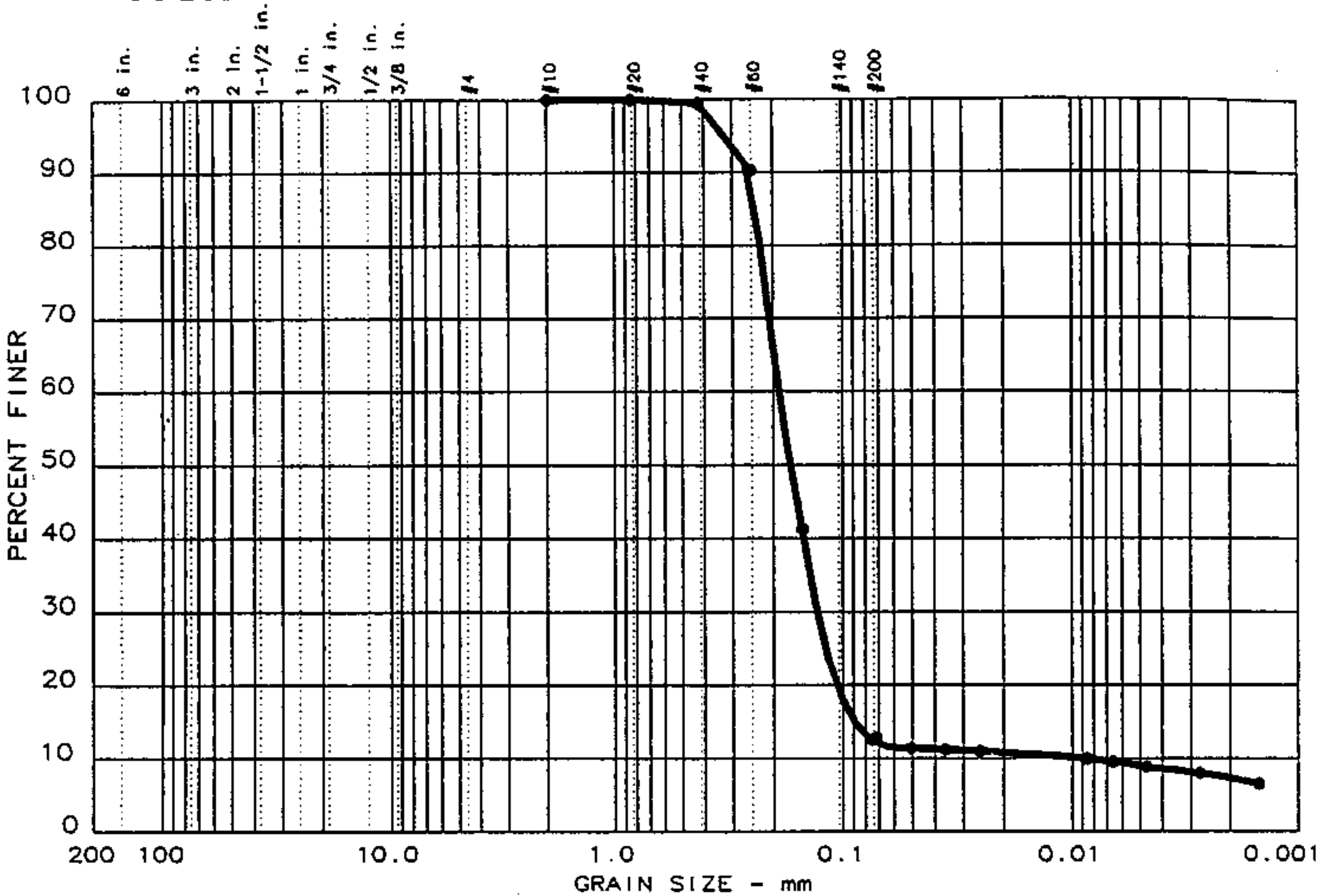
LOG OF BORING AND TEST RESULTS

LONDON AVENUE OUTFALL CANAL, FRONTAL PROTECTION AT PUMPING STATION NO. 3 (SHEET 3 Of 3)
NEW ORLEANS, LOUISIANA

Ground Elev.: Datum: Gr. Water Depth: See Text Job No.: 13065 Date Drilled: 8/04/94 Boring: 1 Refer To "Legends & Notes"

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density Dry	Density Wet	Shear Tests Type	Shear Tests Ø	Shear Tests C	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
					Stiff gray clay	CH	28	102–103											
105		50=10"			Very compact gray sandy silt	ML	29	104–105											PD
110		50=8"					30	109–110											PD
115		30			Medium dense gray fine sand w/clay layers	SP	31	114–115											
120		23					32	119–120											
					Stiff gray silty clay	CL													
125		14					33	124–125	41										
130																			
135																			
140																			
145																			
150																			

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.0	87.4	3.6	9.0	SM		

SIEVE inches size	PERCENT FINER ●		

	GRAIN SIZE		
D_{60}	0.19		
D_{30}	0.13		
D_{10}	0.00		

	COEFFICIENTS		
C_c	10.16		
C_u	21.5		

SIEVE number size	PERCENT FINER ●		
10	100.0		
20	99.9		
40	99.5		
60	90.3		
100	41.2		
200	12.6		

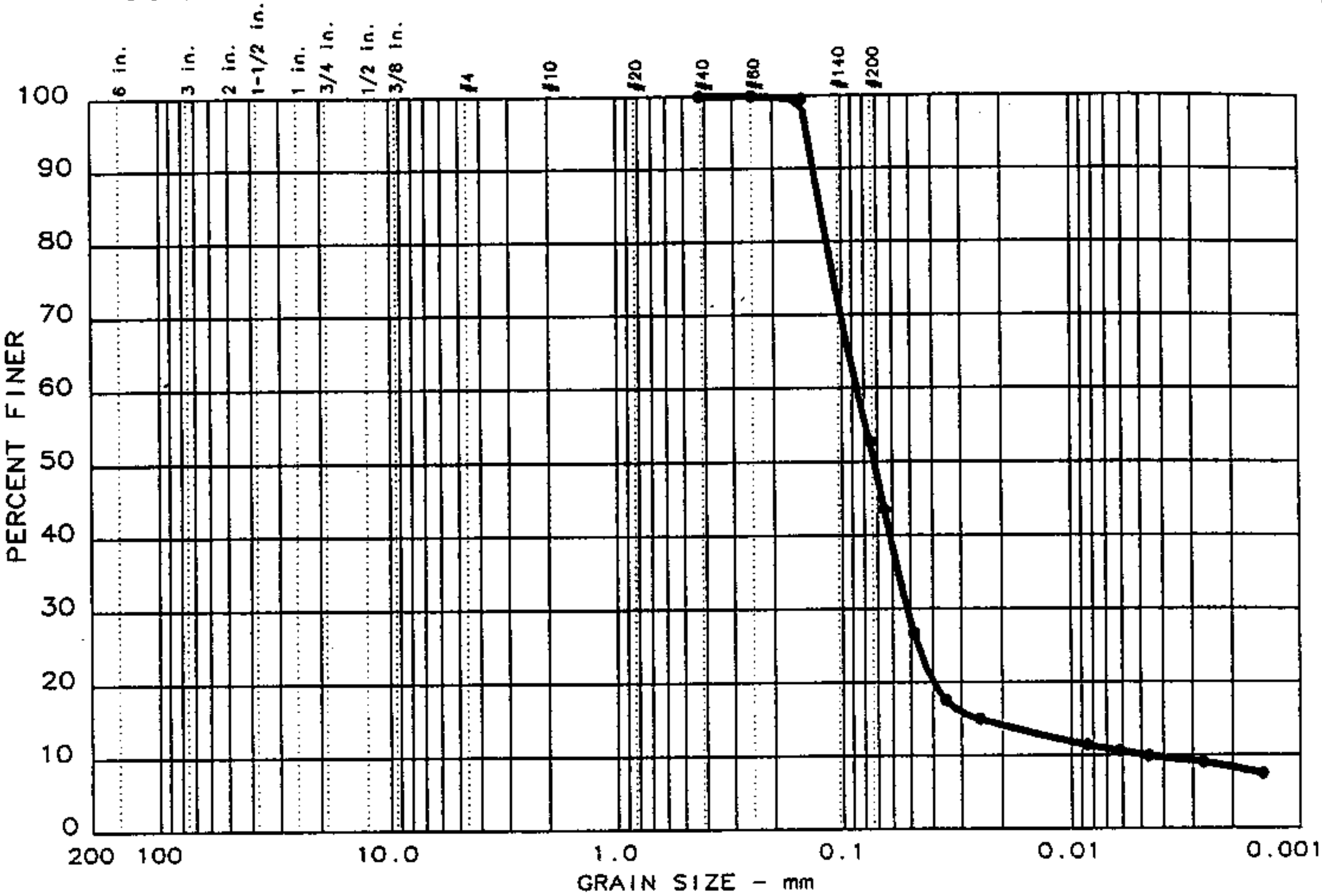
Sample information:
● Boring 1, Sample 16
Gray Silty Sand
w/tr shell frag & om

Remarks:
Sample depth 51'-52'

Eustis Engineering Company, Inc.

Project No.: 13065
Project: London Avenue Canal - Pump Station #3
Date: 8-15-94 Data Sheet No. ____

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.0	47.3	42.6	10.1	ML		

SIEVE inches size	PERCENT FINER ●		

	GRAIN SIZE		
D_{60}	0.09		
D_{30}	0.05		
D_{10}	0.00		

	COEFFICIENTS		
C_c	6.54		
C_u	18.0		

SIEVE number size	PERCENT FINER ●		
40	100.0		
60	99.9		
100	99.5		
200	52.7		

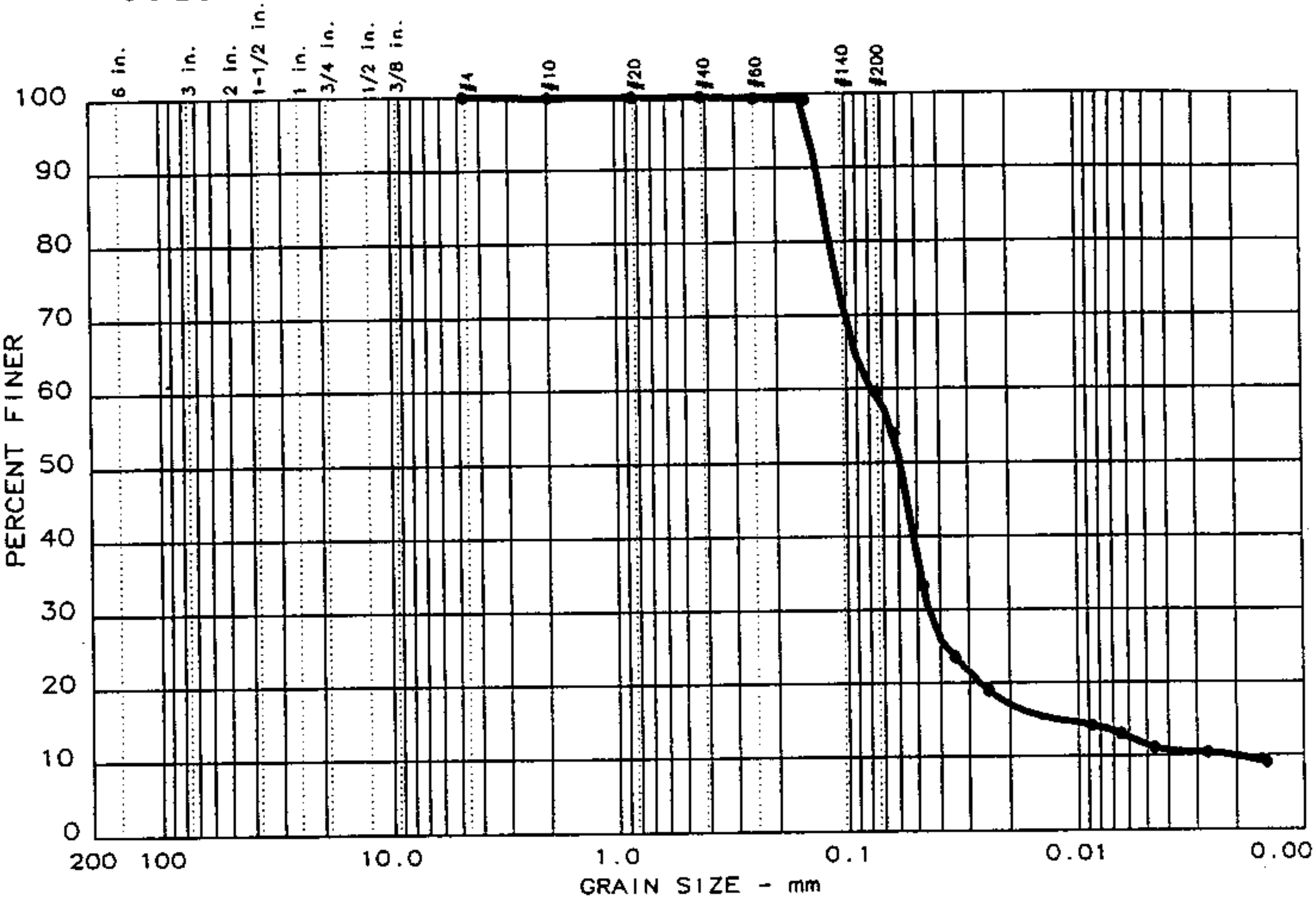
Sample information:
● Boring 1, Sample 29
Gray Sandy Silt

Remarks:
Sample depth 104'-105'

Eustis Engineering Company, Inc.

Project No.: 13065
Project: London Avenue Canal - Pump Station #3
Date: 8-15-94
Data Sheet No. ____

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.0	40.5	48.0	11.5	ML		

SIEVE inches size	PERCENT FINER ●		
	GRAIN SIZE		
D_{60}	0.08		
D_{30}	0.04		
D_{10}	0.00		
	COEFFICIENTS		
C_c	13.49		
C_u	40.7		

SIEVE number size	PERCENT FINER ●		
4	100.0		
10	99.8		
20	99.8		
40	99.7		
60	99.4		
100	99.1		
200	59.5		

Sample information:
● Boring 1, Sample 30
Gray Sandy Silt
w/tr clay & shell frag

Remarks:
Sample depth 109'-110'

Eustis Engineering Company, Inc.

Project No.: 13065
Project: London Avenue Canal - Pump Station #3
Date: 8-15-94 Data Sheet No. ——

APPENDIX B

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 29-MAR-1995 TIME: 10.46.40

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¤ INPUT DATA ¤
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I.--HEADING:
'LONDON AVE CANAL JOB 13065
'TEMPORARY DAM ACROSS CANAL Q-CASE

II.--CONTROL
CANTILEVER WALL DESIGN

LEVEL 1 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
LEVEL 1 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.00

III.--WALL DATA
ELEVATION AT TOP OF WALL = 1.57 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	-10.00

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	-10.00

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH- ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH- ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
95.00	95.00	.00	100.0	.00	.0	-12.00	.00	DEF	DEF
110.00	110.00	.00	500.0	.00	.0	-16.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-26.00	.00	DEF	DEF
101.00	101.00	.00	475.0	.00	.0	-31.00	.00	DEF	DEF
101.00	101.00	.00	525.0	.00	.0	-36.00	.00	DEF	DEF

101.00	101.00	.00	575.0	.00	.0	-41.00	.00	DEF	DEF
120.00	120.00	15.00	300.0	.00	.0	-60.00	.00	DEF	DEF
110.00	110.00	.00	750.0	.00	.0	-65.00	.00	DEF	DEF
119.00	119.00	.00	1650.0	.00	.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA

LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH- ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH- ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
95.00	95.00	.00	100.0	.00	.0	-12.00	.00	DEF	DEF
110.00	110.00	.00	500.0	.00	.0	-16.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-26.00	.00	DEF	DEF
101.00	101.00	.00	475.0	.00	.0	-31.00	.00	DEF	DEF
101.00	101.00	.00	525.0	.00	.0	-36.00	.00	DEF	DEF
101.00	101.00	.00	575.0	.00	.0	-41.00	.00	DEF	DEF
120.00	120.00	15.00	300.0	.00	.0	-60.00	.00	DEF	DEF
110.00	110.00	.00	750.0	.00	.0	-65.00	.00	DEF	DEF
119.00	119.00	.00	1650.0	.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 1.57 (FT)
LEFTSIDE ELEVATION = -10.00 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 29-MAR-1995 TIME: 10.47.03

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¤ SUMMARY OF RESULTS FOR ¤
¤ CANTILEVER WALL DESIGN ¤
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I.--HEADING

'LONDON AVE CANAL JOB 13065
'TEMPORARY DAM ACROSS CANAL Q-CASE

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

```
WALL BOTTOM ELEV. (FT)     :        -39.17
      PENETRATION (FT)     :         29.17

MAX. BEND. MOMENT (LB-FT)  :        67283.
      AT ELEVATION (FT)    :        -25.46

MAX. SCALED DEFL. (LB-IN3) :    5.4733E+10
      AT ELEVATION (FT)    :          1.57
```

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS BY CLASSICAL METHODS

DATE: 29-MAR-1995 TIME: 10.47.03

COMPLETE RESULTS FOR
CANTILEVER WALL DESIGN

I.--HEADING

'LONDON AVE CANAL JOB 13065
'TEMPORARY DAM ACROSS CANAL Q-CASE

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
1.57	0.	0.	5.4733E+10	.00

.57	10.	31.	5.2508E+10	62.50
-.43	83.	125.	5.0283E+10	125.00
-1.43	281.	281.	4.8059E+10	187.50
-2.43	667.	500.	4.5834E+10	250.00
-3.43	1302.	781.	4.3612E+10	312.50
-4.43	2250.	1125.	4.1391E+10	375.00
-5.43	3573.	1531.	3.9174E+10	437.50
-6.43	5333.	2000.	3.6964E+10	500.00
-7.43	7594.	2531.	3.4763E+10	562.50
-8.43	10417.	3125.	3.2575E+10	625.00
-9.43	13865.	3781.	3.0405E+10	687.50
-10.00	16134.	4183.	2.9178E+10	723.13
-10.00	16134.	4183.	2.9178E+10	523.13
-10.43	17980.	4405.	2.8259E+10	509.15
-11.00	20573.	4690.	2.7049E+10	490.63
-11.43	22635.	4898.	2.6144E+10	476.65
-12.00	25481.	5051.	2.4956E+10	58.13
-12.06	25782.	5052.	2.4833E+10	.00
-12.24	26717.	5035.	2.4450E+10	-181.15
-12.43	27645.	4985.	2.4069E+10	-362.30
-13.43	32442.	4599.	2.2041E+10	-409.80
-14.43	36828.	4166.	2.0069E+10	-457.30
-15.43	40757.	3684.	1.8161E+10	-504.80
-16.00	42793.	3493.	1.7105E+10	-166.78
-16.43	44290.	3489.	1.6323E+10	147.44
-17.43	47832.	3577.	1.4562E+10	29.10
-18.43	51404.	3547.	1.2883E+10	-89.24
-19.43	54887.	3399.	1.1294E+10	-207.58
-20.43	58162.	3132.	9.7986E+09	-325.92
-21.43	61112.	2747.	8.4040E+09	-444.25
-22.43	63617.	2243.	7.1150E+09	-562.59
-23.43	65559.	1622.	5.9359E+09	-680.93
-24.43	66821.	882.	4.8699E+09	-799.27
-25.43	67283.	23.	3.9193E+09	-917.61
-26.00	67141.	-529.	3.4294E+09	-1020.97
-26.43	66818.	-980.	3.0848E+09	-1073.43
-27.43	65295.	-2072.	2.3656E+09	-1111.93
-28.43	62660.	-3203.	1.7591E+09	-1150.43
-29.12	60186.	-4003.	1.4055E+09	-1176.88
-29.43	58875.	-4371.	1.2607E+09	-1176.88
-30.43	53916.	-5548.	8.6390E+08	-1176.88
-31.00	50557.	-6244.	6.7995E+08	-1263.13
-31.30	48602.	-6638.	5.9365E+08	-1335.73
-31.43	47753.	-6802.	5.6006E+08	-1265.76
-31.71	45770.	-7140.	4.8924E+08	-1108.26
-32.43	40410.	-7791.	3.3857E+08	-711.76
-33.43	32355.	-8226.	1.8681E+08	-157.76
-34.43	24143.	-8107.	9.0927E+07	396.24
-35.43	16326.	-7434.	3.6823E+07	950.24
-36.00	12261.	-6802.	1.9474E+07	1266.02
-36.43	9460.	-6206.	1.1069E+07	1504.24
-37.43	4098.	-4425.	1.8774E+06	2058.23
-38.43	795.	-2090.	6.4496E+04	2612.23
-39.17	1.	0.	0.0000E+00	3023.13

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

III.--SOIL PRESSURES

ELEVATION	< LEFTSIDE PRESSURE (PSF)>		<RIGHTSIDE PRESSURE (PSF)>	
(FT)	PASSIVE	ACTIVE	ACTIVE	PASSIVE
1.57	0.	0.	0.	0.
.57	0.	0.	0.	0.
-.43	0.	0.	0.	0.
-1.43	0.	0.	0.	0.
-2.43	0.	0.	0.	0.
-3.43	0.	0.	0.	0.
-4.43	0.	0.	0.	0.
-5.43	0.	0.	0.	0.
-6.43	0.	0.	0.	0.
-7.43	0.	0.	0.	0.
-8.43	0.	0.	0.	0.
-9.43	0.	0.	0.	0.
-10.00+	0.	0.	0.	0.
-10.00-	200.	0.	0.	200.
-10.43	214.	0.	0.	214.
-11.00	233.	0.	0.	233.
-11.43	246.	0.	0.	246.
-12.00+	265.	0.	0.	265.
-12.00-	1065.	0.	0.	1065.
-12.06	1068.	0.	0.	1068.
-12.24	1077.	0.	0.	1077.
-12.43	1085.	0.	0.	1085.
-13.43	1133.	0.	0.	1133.
-14.43	1180.	0.	0.	1180.
-15.43	1228.	0.	0.	1228.
-16.00+	1255.	0.	0.	1255.
-16.00-	628.	103.	103.	628.
-16.43	689.	114.	114.	689.
-17.43	831.	137.	137.	831.
-18.43	973.	160.	160.	973.
-19.43	1114.	184.	184.	1114.
-20.43	1256.	207.	207.	1256.
-21.43	1398.	230.	230.	1398.
-22.43	1539.	254.	254.	1539.
-23.43	1681.	277.	277.	1681.
-24.43	1823.	300.	300.	1823.
-25.43	1964.	324.	324.	1964.
-26.00+	2045.	337.	337.	2045.
-26.00-	1780.	0.	0.	1780.
-26.43	1797.	0.	0.	1797.
-27.43	1835.	0.	0.	1835.
-28.43	1874.	0.	0.	1874.
-29.12	1900.	0.	0.	1900.
-29.43	1912.	12.	12.	1912.
-30.43	1951.	51.	51.	1951.
-31.00+	1973.	73.	73.	1973.
-31.00-	2073.	0.	0.	2073.
-31.30	2084.	0.	0.	2084.
-31.43	2089.	0.	0.	2089.
-31.71	2100.	0.	0.	2100.
-32.43	2128.	28.	28.	2128.
-33.43	2166.	66.	66.	2166.
-34.43	2205.	105.	105.	2205.

-35.43	2243.	143.	143.	2243.
-36.00+	2265.	165.	165.	2265.
-36.00-	2365.	65.	65.	2365.
-36.43	2382.	82.	82.	2382.
-37.43	2420.	120.	120.	2420.
-38.43	2459.	159.	159.	2459.
-39.17	2497.	197.	197.	2497.
-40.43	2536.	236.	236.	2536.

```
1000 'LONDON AVE CANAL   JOB 13065
1010 'TEMPORARY DAM ACROSS CANAL     Q-CASE
1020 CONTROL  C  D    1.00     1.00
1030 WALL        1.57
1040 SURFACE RIGHTSIDE   1
1050        .00    -10.00
1060 SURFACE  LEFTSIDE   1
1070        .00    -10.00
1080 SOIL RIGHTSIDE  STRENGTH    9      .00      .00
1090   95.00   95.00   .00  100.00     .00      .00     -12.00     .00   .00   .00
1100  110.00  110.00   .00  500.00     .00      .00     -16.00     .00   .00   .00
1110  120.00  120.00 25.00      .00    .00      .00     -26.00     .00   .00   .00
1120  101.00  101.00   .00  475.00     .00      .00     -31.00     .00   .00   .00
1130  101.00  101.00   .00  525.00     .00      .00     -36.00     .00   .00   .00
1140  101.00  101.00   .00  575.00     .00      .00     -41.00     .00   .00   .00
1150  120.00  120.00 15.00  300.00     .00      .00     -60.00     .00   .00   .00
1160  110.00  110.00   .00  750.00     .00      .00     -65.00     .00   .00   .00
1170  119.00  119.00   .00 1650.00     .00      .00        .00     .00
1180 SOIL  LEFTSIDE  STRENGTH    9      .00      .00
1190   95.00   95.00   .00  100.00     .00      .00     -12.00     .00   .00   .00
1200  110.00  110.00   .00  500.00     .00      .00     -16.00     .00   .00   .00
1210  120.00  120.00 25.00      .00    .00      .00     -26.00     .00   .00   .00
1220  101.00  101.00   .00  475.00     .00      .00     -31.00     .00   .00   .00
1230  101.00  101.00   .00  525.00     .00      .00     -36.00     .00   .00   .00
1240  101.00  101.00   .00  575.00     .00      .00     -41.00     .00   .00   .00
1250  120.00  120.00 15.00  300.00     .00      .00     -60.00     .00   .00   .00
1260  110.00  110.00   .00  750.00     .00      .00     -65.00     .00   .00   .00
1270  119.00  119.00   .00 1650.00     .00      .00        .00     .00
1280 WATER ELEVATIONS     62.50     1.57  -10.00
1290 FINISH
```

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 29-MAR-1995 TIME: 10.37.31

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¤ INPUT DATA ¤
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I.--HEADING:
'LONDON AVE CANAL JOB 13065
'TEMPORARY COFFERDAMS AT DISCHARGE TUBES Q-CASE

II.--CONTROL
CANTILEVER WALL DESIGN

LEVEL 1 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
LEVEL 1 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.00

III.--WALL DATA
ELEVATION AT TOP OF WALL = 1.57 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	-11.28

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	-11.28

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	<--BOTTOM--> SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	<-SAFETY-> <-FACTOR-> PASS.
95.00	95.00	.00	100.0	.00	.0	-12.00	.00	DEF	DEF
110.00	110.00	.00	500.0	.00	.0	-16.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-26.00	.00	DEF	DEF
101.00	101.00	.00	475.0	.00	.0	-31.00	.00	DEF	DEF
101.00	101.00	.00	525.0	.00	.0	-36.00	.00	DEF	DEF

101.00	101.00	.00	575.0	.00	.0	-41.00	.00	DEF	DEF
120.00	120.00	15.00	300.0	.00	.0	-60.00	.00	DEF	DEF
110.00	110.00	.00	750.0	.00	.0	-65.00	.00	DEF	DEF
119.00	119.00	.00	1650.0	.00	.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA

LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH- ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH- ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
95.00	95.00	.00	100.0	.00	.0	-12.00	.00	DEF	DEF
110.00	110.00	.00	500.0	.00	.0	-16.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-26.00	.00	DEF	DEF
101.00	101.00	.00	475.0	.00	.0	-31.00	.00	DEF	DEF
101.00	101.00	.00	525.0	.00	.0	-36.00	.00	DEF	DEF
101.00	101.00	.00	575.0	.00	.0	-41.00	.00	DEF	DEF
120.00	120.00	15.00	300.0	.00	.0	-60.00	.00	DEF	DEF
110.00	110.00	.00	750.0	.00	.0	-65.00	.00	DEF	DEF
119.00	119.00	.00	1650.0	.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 1.57 (FT)
LEFTSIDE ELEVATION = -11.28 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS BY CLASSICAL METHODS

DATE: 29-MAR-1995 TIME: 10.37.44

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¤ SUMMARY OF RESULTS FOR ¤
¤ CANTILEVER WALL DESIGN ¤
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I.--HEADING

'LONDON AVE CANAL JOB 13065
'TEMPORARY COFFERDAMS AT DISCHARGE TUBES Q-CASE

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

```
WALL BOTTOM ELEV. (FT)     :        -43.07
      PENETRATION (FT)     :         31.79

MAX. BEND. MOMENT (LB-FT) :         89333.
      AT ELEVATION (FT)    :        -28.06

MAX. SCALED DEFL. (LB-IN3):     8.7551E+10
      AT ELEVATION (FT)    :          1.57
```

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 29-MAR-1995 TIME: 10.37.44

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¤   COMPLETE RESULTS FOR    ¤
¤  CANTILEVER WALL DESIGN   ¤
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I.--HEADING

'LONDON AVE CANAL JOB 13065
'TEMPORARY COFFERDAMS AT DISCHARGE TUBES Q-CASE

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
1.57	0.	0.	8.7551E+10	.00

.57	10.	31.	8.4327E+10	62.50
-.43	83.	125.	8.1104E+10	125.00
-1.43	281.	281.	7.7881E+10	187.50
-2.43	667.	500.	7.4659E+10	250.00
-3.43	1302.	781.	7.1437E+10	312.50
-4.43	2250.	1125.	6.8218E+10	375.00
-5.43	3573.	1531.	6.5003E+10	437.50
-6.43	5333.	2000.	6.1794E+10	500.00
-7.43	7594.	2531.	5.8595E+10	562.50
-8.43	10417.	3125.	5.5408E+10	625.00
-9.43	13865.	3781.	5.2240E+10	687.50
-10.43	18000.	4500.	4.9096E+10	750.00
-11.28	22102.	5160.	4.6447E+10	803.13
-11.28	22102.	5160.	4.6447E+10	603.13
-11.43	22883.	5250.	4.5983E+10	598.25
-12.00	25950.	5472.	4.4226E+10	179.73
-12.12	26617.	5483.	4.3852E+10	.00
-12.20	27051.	5478.	4.3610E+10	-116.79
-12.28	27484.	5464.	4.3368E+10	-233.58
-12.43	28301.	5429.	4.2909E+10	-240.70
-13.43	33602.	5164.	3.9885E+10	-288.20
-14.43	38614.	4852.	3.6918E+10	-335.70
-15.43	43291.	4493.	3.4019E+10	-383.20
-16.00	45809.	4377.	3.2399E+10	-23.17
-16.43	47699.	4439.	3.1194E+10	313.05
-17.43	52275.	4693.	2.8451E+10	194.71
-18.43	57046.	4829.	2.5799E+10	76.37
-19.43	61894.	4846.	2.3245E+10	-41.96
-20.43	66699.	4745.	2.0798E+10	-160.30
-21.43	71344.	4525.	1.8467E+10	-278.64
-22.43	75711.	4188.	1.6259E+10	-396.98
-23.43	79680.	3732.	1.4181E+10	-515.32
-24.43	83134.	3157.	1.2241E+10	-633.65
-25.43	85955.	2464.	1.0445E+10	-751.99
-26.00	87230.	2000.	9.4874E+09	-877.36
-26.43	88007.	1607.	8.7972E+09	-951.83
-27.43	89131.	635.	7.3013E+09	-990.33
-28.43	89265.	-374.	5.9593E+09	-1028.83
-29.43	88370.	-1422.	4.7713E+09	-1067.33
-30.20	86962.	-2253.	3.9633E+09	-1096.88
-30.43	86408.	-2508.	3.7359E+09	-1096.88
-31.00	84797.	-3152.	3.2128E+09	-1162.33
-31.43	83332.	-3669.	2.8497E+09	-1244.33
-32.43	79034.	-4933.	2.1073E+09	-1282.83
-32.79	77149.	-5403.	1.8709E+09	-1296.88
-33.43	73455.	-6227.	1.5013E+09	-1296.88
-34.43	66580.	-7524.	1.0220E+09	-1296.88
-35.43	58408.	-8821.	6.5754E+08	-1296.88
-35.54	57463.	-8959.	6.2501E+08	-1315.51
-36.00	53178.	-9498.	4.9589E+08	-1007.26
-36.43	49009.	-9870.	3.9386E+08	-721.46
-37.43	38890.	-10259.	2.1476E+08	-56.81
-38.43	28713.	-9983.	1.0286E+08	607.84
-39.43	19145.	-9043.	4.0661E+07	1272.49
-40.43	10848.	-7438.	1.1728E+07	1937.14
-41.00	6944.	-6226.	4.5374E+06	2315.99
-41.43	4489.	-5169.	1.8197E+06	2601.79
-42.43	732.	-2235.	4.4183E+04	3266.44

-43.07	0.	0.	0.0000E+00	3693.28

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

III.--SOIL PRESSURES

ELEVATION (FT)	< LEFTSIDE PRESSURE (PSF) > PASSIVE	ACTIVE	<RIGHTSIDE PRESSURE (PSF) > ACTIVE	PASSIVE
1.57	0.	0.	0.	0.
.57	0.	0.	0.	0.
-.43	0.	0.	0.	0.
-1.43	0.	0.	0.	0.
-2.43	0.	0.	0.	0.
-3.43	0.	0.	0.	0.
-4.43	0.	0.	0.	0.
-5.43	0.	0.	0.	0.
-6.43	0.	0.	0.	0.
-7.43	0.	0.	0.	0.
-8.43	0.	0.	0.	0.
-9.43	0.	0.	0.	0.
-10.43	0.	0.	0.	0.
-11.28+	0.	0.	0.	0.
-11.28-	200.	0.	0.	200.
-11.43	205.	0.	0.	205.
-12.00+	223.	0.	0.	223.
-12.00-	1023.	0.	0.	1023.
-12.12	1029.	0.	0.	1029.
-12.20	1033.	0.	0.	1033.
-12.28	1037.	0.	0.	1037.
-12.43	1044.	0.	0.	1044.
-13.43	1091.	0.	0.	1091.
-14.43	1139.	0.	0.	1139.
-15.43	1186.	0.	0.	1186.
-16.00+	1213.	0.	0.	1213.
-16.00-	526.	87.	87.	526.
-16.43	587.	97.	97.	587.
-17.43	728.	120.	120.	728.
-18.43	870.	143.	143.	870.
-19.43	1012.	167.	167.	1012.
-20.43	1153.	190.	190.	1153.
-21.43	1295.	213.	213.	1295.
-22.43	1437.	237.	237.	1437.
-23.43	1578.	260.	260.	1578.
-24.43	1720.	283.	283.	1720.
-25.43	1862.	307.	307.	1862.
-26.00+	1943.	320.	320.	1943.
-26.00-	1738.	0.	0.	1738.
-26.43	1755.	0.	0.	1755.
-27.43	1793.	0.	0.	1793.
-28.43	1832.	0.	0.	1832.
-29.43	1870.	0.	0.	1870.
-30.20	1900.	0.	0.	1900.
-30.43	1909.	9.	9.	1909.
-31.00+	1931.	31.	31.	1931.
-31.00-	2031.	0.	0.	2031.
-31.43	2047.	0.	0.	2047.

-32.43	2086.	0.	0.	2086.
-32.79	2100.	0.	0.	2100.
-33.43	2124.	24.	24.	2124.
-34.43	2163.	63.	63.	2163.
-35.43	2201.	101.	101.	2201.
-35.54	2206.	87.	106.	2206.
-36.00+	2223.	123.	123.	2223.
-36.00-	2323.	23.	23.	2323.
-36.43	2340.	40.	40.	2340.
-37.43	2378.	78.	78.	2378.
-38.43	2417.	117.	117.	2417.
-39.43	2455.	155.	155.	2455.
-40.43	2494.	194.	194.	2494.
-41.00+	2516.	216.	216.	2516.
-41.00-	3102.	344.	344.	3102.
-41.43	3144.	358.	358.	3144.
-42.43	3241.	392.	392.	3241.
-43.07	3339.	426.	426.	3339.
-44.43	3437.	460.	460.	3437.

```
1000 'LONDON AVE CANAL      JOB 13065
1010 'TEMPORARY COFFERDAMS AT DISCHARGE TUBES     Q-CASE
1020 CONTROL  C  D    1.00     1.00
1030 WALL        1.57
1040 SURFACE RIGHTSIDE   1
1050           .00     -11.28
1060 SURFACE  LEFTSIDE   1
1070           .00     -11.28
1080 SOIL RIGHTSIDE  STRENGTH     9        .00        .00
1090   95.00   95.00    .00  100.00      .00        .00     -12.00     .00    .00    .00
1100  110.00  110.00    .00  500.00      .00        .00     -16.00     .00    .00    .00
1110  120.00  120.00 25.00      .00      .00        .00     -26.00     .00    .00    .00
1120  101.00  101.00    .00  475.00      .00        .00     -31.00     .00    .00    .00
1130  101.00  101.00    .00  525.00      .00        .00     -36.00     .00    .00    .00
1140  101.00  101.00    .00  575.00      .00        .00     -41.00     .00    .00    .00
1150  120.00  120.00 15.00  300.00      .00        .00     -60.00     .00    .00    .00
1160  110.00  110.00    .00  750.00      .00        .00     -65.00     .00    .00    .00
1170  119.00  119.00    .00 1650.00      .00        .00        .00     .00
1180 SOIL  LEFTSIDE  STRENGTH     9        .00        .00
1190   95.00   95.00    .00  100.00      .00        .00     -12.00     .00    .00    .00
1200  110.00  110.00    .00  500.00      .00        .00     -16.00     .00    .00    .00
1210  120.00  120.00 25.00      .00      .00        .00     -26.00     .00    .00    .00
1220  101.00  101.00    .00  475.00      .00        .00     -31.00     .00    .00    .00
1230  101.00  101.00    .00  525.00      .00        .00     -36.00     .00    .00    .00
1240  101.00  101.00    .00  575.00      .00        .00     -41.00     .00    .00    .00
1250  120.00  120.00 15.00  300.00      .00        .00     -60.00     .00    .00    .00
1260  110.00  110.00    .00  750.00      .00        .00     -65.00     .00    .00    .00
1270  119.00  119.00    .00 1650.00      .00        .00        .00     .00
1280 WATER ELEVATIONS     62.50     1.57  -11.28
1290 FINISH
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APPENDIX C

LONDON AVENUE OUTFALL CANAL
FRONTAL PROTECTION AT PUMPING STATION NO. 3
NEW ORLEANS, LOUISIANA

I-WALL ANALYSES

FURNISHED DATA: GROUND SURFACE EL. 8.57 BOTH SIDES
STILL WATER LEVEL (SWL) EL. 11.90
SWL PLUS 2 FEET FREEBOARD EL. 13.90
TOP OF WALL EL. 14.40
ELEVATIONS REFER TO N.G.V.D.

Q-CASE

F.S. = 1.5 WATER EL. 11.90 TIP EL. 5.94 Mmax = 560 ft-lbs

F.S. = 1.0 WATER EL. 13.90 TIP EL. 3.35 Mmax = 2369 ft-lbs

COMPUTED VALUE (CV) IS DEEPEST PENETRATION ABOVE.

COMPARE CV TO 3:1 AND 2.5:1 PENETRATION TO HEAD RATIOS.

3:1 PENETRATION TO HEAD RATIO

HEAD = 11.90 - 8.57 = 3.33 FEET (USING SWL)

PENETRATION = 3 x 3.33 = 9.99 FEET

TIP EL. -1.42

2.5:1 PENETRATION TO HEAD RATIO

HEAD = 13.90 - 8.57 = 5.33 FEET (USING SWL + 2 FEET FREEBOARD)

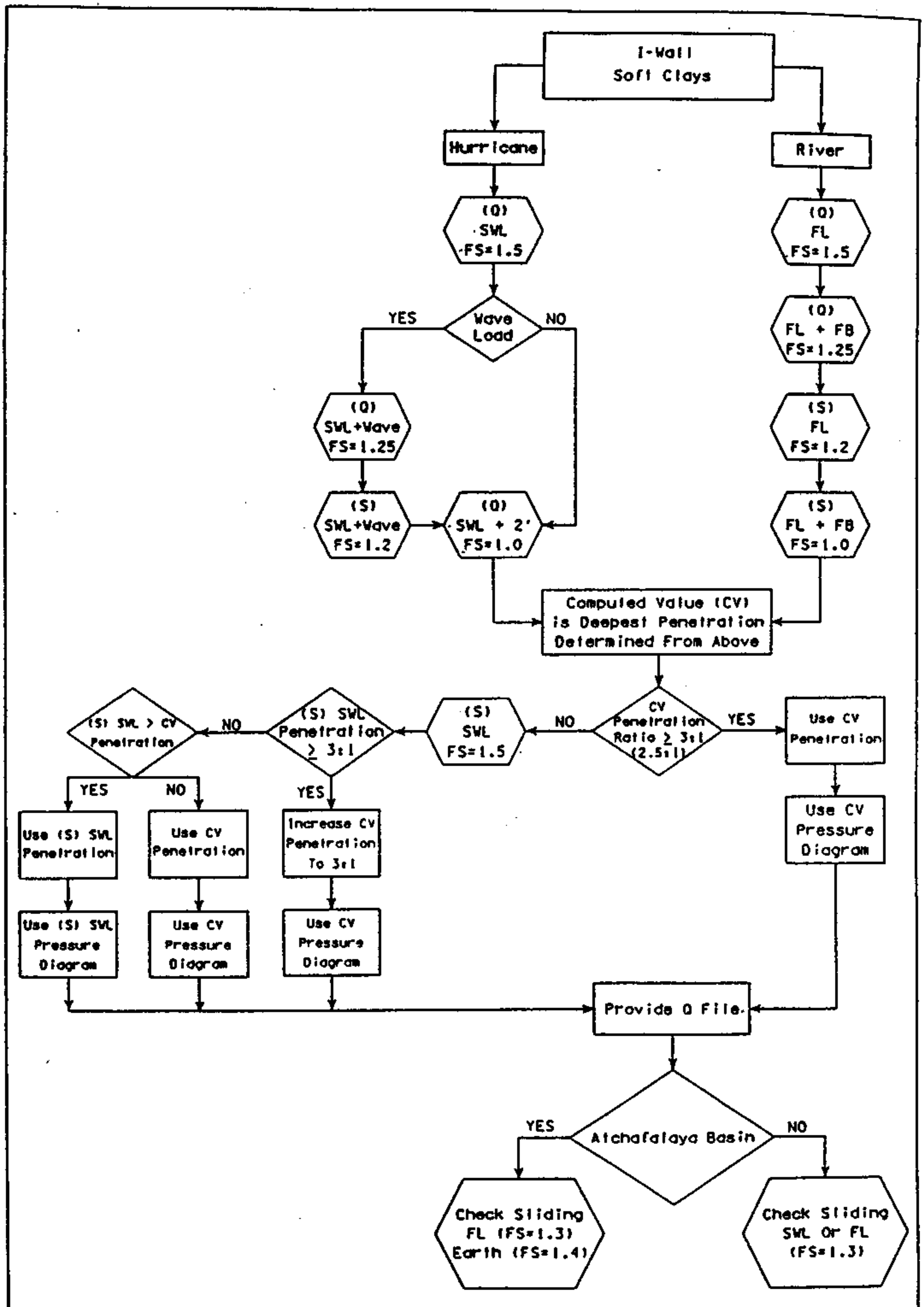
PENETRATION = 2.5 x 5.33 = 13.33 FEET

TIP EL. -4.76

SINCE CV LESS THAN 3:1 AND 2.5:1 RATIOS, CHECK S-CASE

F.S. = 1.5 WATER EL. 11.90 TIP EL.-0.80 Mmax = 2398 ft-lbs

∴ SINCE TIP EL. -0.80 LESS THAN PENETRATION FOR 3:1 RATIO AND GREATER THAN CV PENETRATION, USE TIP EL -0.80 AND PRESSURE DIAGRAM FOR S-CASE FOR DESIGN.



I-Wall
Soft Clays
Hurricane
River
(Q)
SWL
FS=1.5
(Q)
FL
FS=1.5
Wave
Load
YES
NO
(Q)
FL + FB
FS=1.25
(Q)
SWL+Wave
FS=1.25
(S)
FL
FS=1.2
(S)
SWL+Wave
FS=1.2
(Q)
SWL + 2'
FS=1.0
(S)
FL + FB
FS=1.0
Computed Value (CV)
is Deepest Penetration
Determined From Above
CV
Penetration
Ratio ≥ 3:1
(2.5:1)
YES
Use CV
Penetration
Use CV
Pressure
Diagram
NO
(S)
SWL
FS=1.5
(S) SWL
Penetration
≥ 3:1
NO
(S) SWL > CV
Penetration
YES
NO
YES
Use (S) SWL
Penetration
Use CV
Penetration
Increase CV
Penetration
To 3:1
Use (S) SWL
Pressure
Diagram
Use CV
Pressure
Diagram
Use CV
Pressure
Diagram
Provide Q File
Atchafalaya Basin
YES
NO
Check Sliding
FL (FS=1.3)
Earth (FS=1.4)
Check Sliding
SWL Or FL
(FS=1.3)

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 26-JAN-1995 TIME: 18.47.25

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¤ INPUT DATA ¤
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I.--HEADING:
'LONDON AVE OUTFALL CANAL FRONTAL PROTECTION
'I-WALL S-CASE

II.--CONTROL
CANTILEVER WALL DESIGN

LEVEL 1 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.50
LEVEL 1 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

III.--WALL DATA
ELEVATION AT TOP OF WALL = 14.40 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	8.57
100.00	8.57

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	8.57
100.00	8.57

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
110.00	115.00	23.00	.0	.00	.0	4.00	.00	DEF	DEF
110.00	115.00	23.00	.0	.00	.0	-6.00	.00	DEF	DEF
110.00	110.00	23.00	.0	.00	.0	-16.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-26.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-31.00	.00	DEF	DEF

101.00	101.00	23.00	.0	.00	.0	-36.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-41.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-60.00	.00	DEF	DEF
110.00	110.00	23.00	.0	.00	.0	-65.00	.00	DEF	DEF
119.00	119.00	23.00	.0	.00	.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA

LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
115.00	115.00	23.00	.0	.00	.0	4.00	.00	DEF	DEF
115.00	115.00	23.00	.0	.00	.0	-6.00	.00	DEF	DEF
110.00	110.00	23.00	.0	.00	.0	-16.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-26.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-31.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-36.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-41.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-60.00	.00	DEF	DEF
110.00	110.00	23.00	.0	.00	.0	-65.00	.00	DEF	DEF
119.00	119.00	23.00	.0	.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 11.90 (FT)
LEFTSIDE ELEVATION = -6.00 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 26-JAN-1995 TIME: 18.47.52

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¤ SUMMARY OF RESULTS FOR ¤
¤ CANTILEVER WALL DESIGN ¤
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I.--HEADING

'LONDON AVE OUTFALL CANAL FRONTAL PROTECTION
'I-WALL S-CASE

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

```
WALL BOTTOM ELEV. (FT)     :        -.80
     PENETRATION (FT)      :        9.37

MAX. BEND. MOMENT (LB-FT)  :        2398.
     AT ELEVATION (FT)     :        3.59

MAX. SCALED DEFL. (LB-IN3) :  2.4178E+08
     AT ELEVATION (FT)     :       14.40
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(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS BY CLASSICAL METHODS

DATE: 26-JAN-1995 TIME: 18.47.52

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¤ COMPLETE RESULTS FOR ¤
¤ CANTILEVER WALL DESIGN ¤
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I.--HEADING

'LONDON AVE OUTFALL CANAL FRONTAL PROTECTION
'I-WALL S-CASE

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
14.40	0.	0.	2.4178E+08	.00
13.40	0.	0.	2.1809E+08	.00
12.40	0.	0.	1.9440E+08	.00
11.90	0.	0.	1.8256E+08	.00
11.40	1.	8.	1.7071E+08	31.25
10.40	35.	70.	1.4703E+08	93.75
9.40	163.	195.	1.2342E+08	156.25
8.57	385.	347.	1.0404E+08	208.13
8.40	446.	380.	1.0011E+08	189.19
7.57	817.	499.	8.1347E+07	96.73
7.40	903.	514.	7.7608E+07	77.80

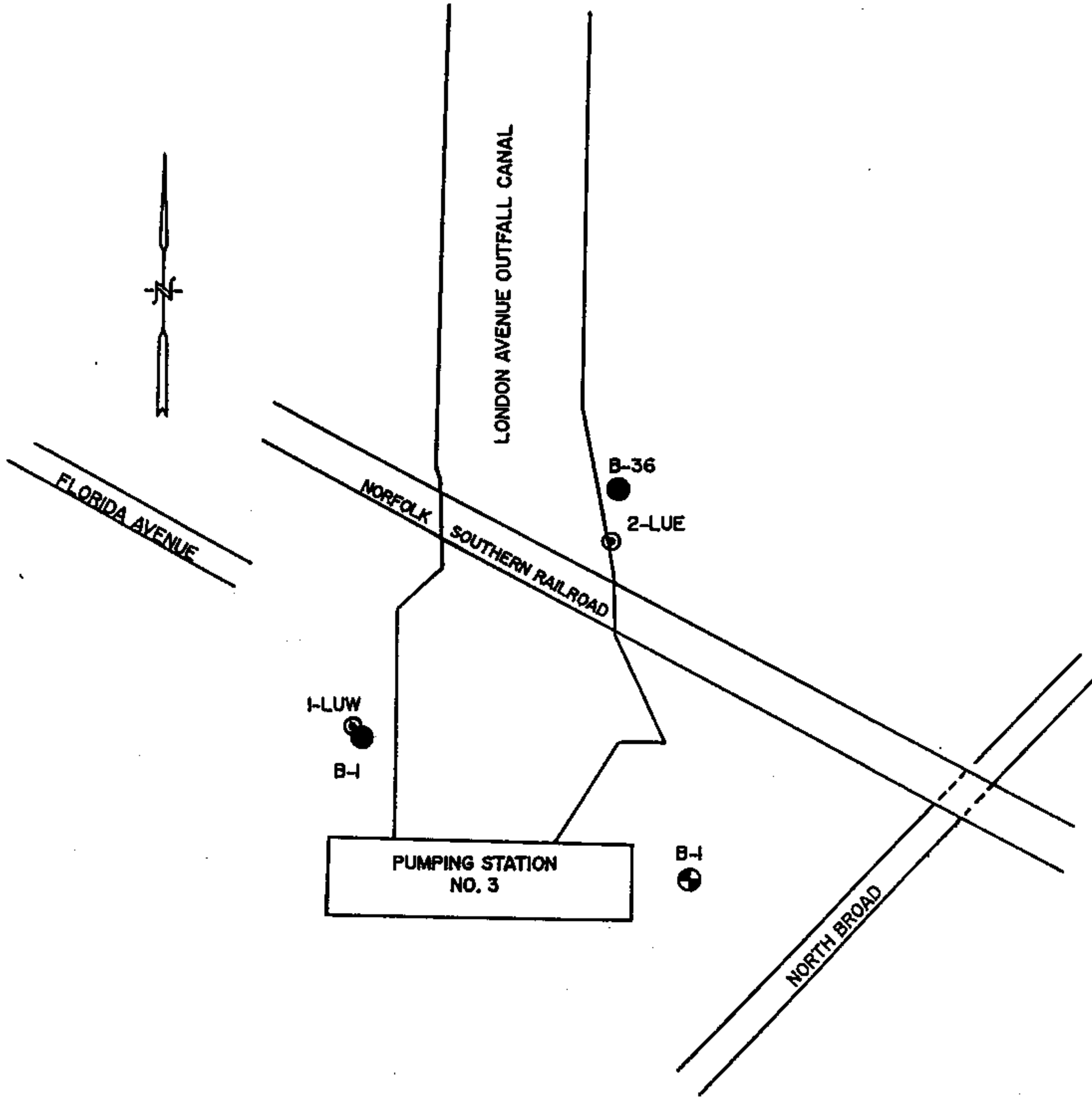
6.70	1274.	541.	6.2767E+07	.00
6.55	1356.	540.	5.9693E+07	-16.80
6.40	1437.	536.	5.6672E+07	-33.60
5.40	1937.	447.	3.8215E+07	-144.99
4.40	2293.	246.	2.3084E+07	-256.38
4.00	2370.	134.	1.8110E+07	-300.94
3.40	2392.	-66.	1.1879E+07	-367.77
2.40	2124.	-490.	4.7544E+06	-479.16
2.10	1952.	-640.	3.3451E+06	-513.01
1.40	1411.	-875.	1.2319E+06	-159.85
.40	541.	-781.	1.2386E+05	347.45
-.60	18.	-180.	1.0436E+02	854.76
-.80	0.	0.	0.0000E+00	955.45

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

III.--SOIL PRESSURES

ELEVATION	< LEFTSIDE PRESSURE (PSF)>		<RIGHTSIDE PRESSURE (PSF)>	
(FT)	PASSIVE	ACTIVE	ACTIVE	PASSIVE
14.40	0.	0.	0.	0.
13.40	0.	0.	0.	0.
12.40	0.	0.	0.	0.
11.90	0.	0.	0.	0.
11.40	0.	0.	0.	0.
10.40	0.	0.	0.	0.
9.40	0.	0.	0.	0.
8.57	0.	0.	0.	0.
8.40	34.	11.	5.	14.
7.57	201.	66.	27.	83.
7.40	235.	77.	32.	97.
6.70	376.	123.	51.	155.
6.55	406.	133.	55.	168.
6.40	436.	143.	59.	180.
5.40	637.	209.	86.	263.
4.40	838.	274.	113.	346.
4.00	919.	301.	124.	380.
3.40	1039.	340.	140.	429.
2.40	1241.	406.	168.	512.
2.10	1302.	426.	176.	538.
1.40	1442.	472.	195.	595.
.40	1643.	537.	222.	678.
-.60	1844.	603.	249.	762.
-.80	2045.	669.	276.	845.
-2.60	2246.	735.	303.	928.

FIGURES 1 - 8



U.S.A.C.E. BORINGS (1971)

EUSTIS ENGINEERING BORINGS (1985)

UNDISTURBED BORING DRILLED 4 AUGUST 1994

LOCATION OF BORINGS

LONDON AVENUE OUTFALL CANAL
FRONTAL PROTECTION AT
PUMPING STATION NO. 3
NEW ORLEANS, LOUISIANA

EUSTIS ENGINEERING COMPANY, INC.

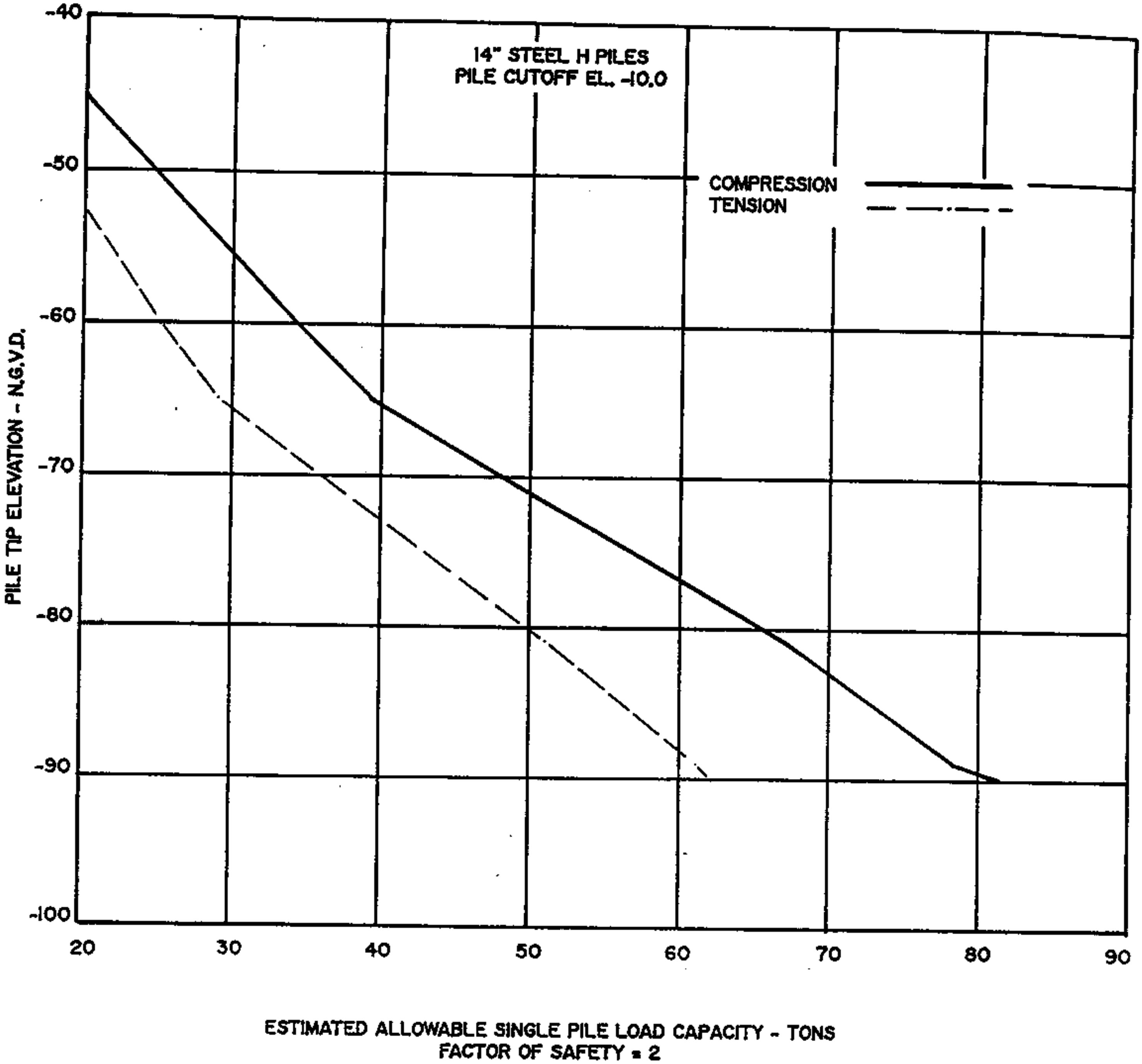
FIGURE 1

LONDON AVENUE OUTFALL CANAL
FRONTAL PROTECTION AT PUMPING STATION NO. 3
NEW ORLEANS, LOUISIANA

SOIL DESIGN PARAMETERS

ELEVATION FEET NGVD	UNIT WEIGHT PCF	(Q) UNDRAINED SHEAR STRENGTH		EFFECTIVE (S) SHEAR STRENGTH
		COESHION PSF	ANGLE OF INTERNAL FRICTION DEGREES	ANGLE OF INTERNAL FRICTION DEGREES
10 to 4	115	1,000	0	23
4 to -6	115	700	0	23
-6 to -16	110	500	0	23
-16 to -26	120	0	25	25
-26 to -41	101	450 to 600*	0	23
-41 to -60	120	300	15	25
-60 to -65	110	750	0	23
-65 to -81	119	1,650	0	23
-81 to -90	119	1,250	0	23

* Denotes shear strength at top and bottom of stratum increasing with depth.



PILE LOAD CAPACITIES

LONDON AVENUE OUTFALL CANAL
FRONTAL PROTECTION AT
PUMPING STATION NO. 3
NEW ORLEANS, LOUISIANA

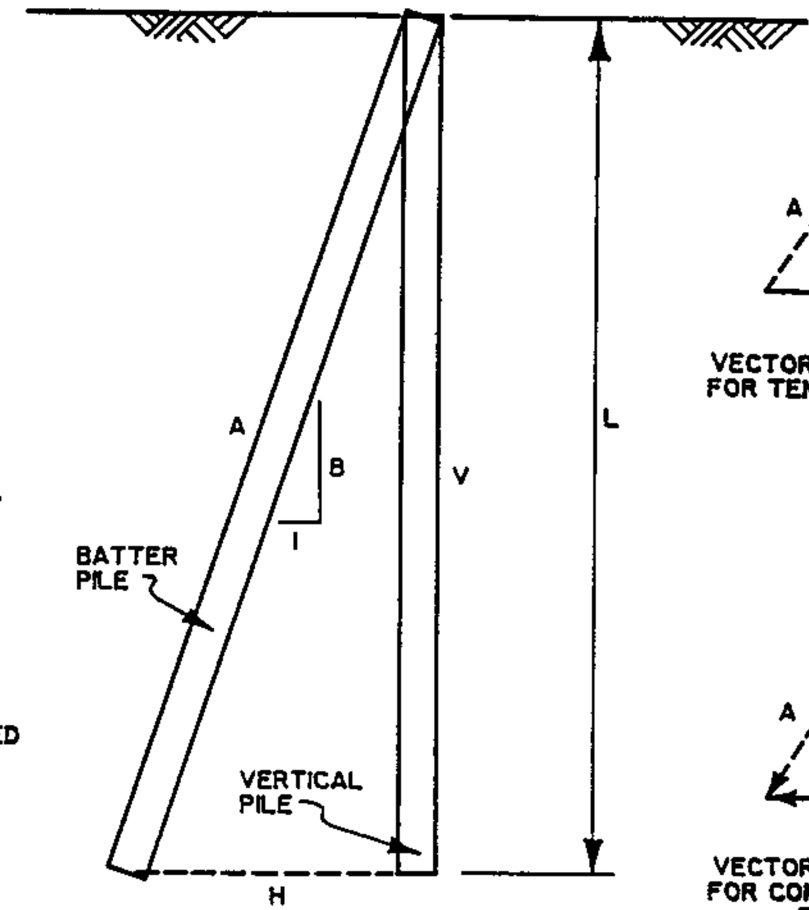
EUSTIS ENGINEERING COMPANY, INC.

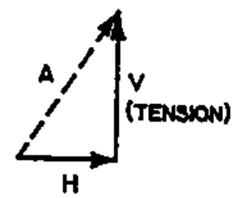
FIGURE 3

AXIAL AND HORIZONTAL RESISTANCE OF BATTER PILES

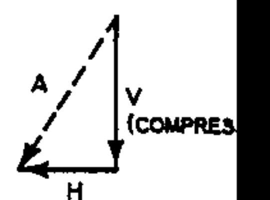
ESTIMATED FROM ALLOWABLE VERTICAL LOAD CAPACITY

L = VERTICAL COMPONENT OF BATTER PILE EMBEDMENT LENGTH.





VECTOR DIAGRAM FOR TENSION PILE



VECTOR DIAGRAM FOR COMPRESSION PILE

V = ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITY OF A PILE DRIVEN VERTICALLY WITH EMBEDMENT LENGTH, L.

B = BATTER OF PILE EXPRESSED AS A RATIO OF VERTICAL DISTANCE TO ONE FOOT HORIZONTAL DISTANCE.

H = HORIZONTAL RESISTANCE OF BATTER PILE ESTIMATED AS FOLLOWS:

$$H = \frac{V}{B}$$

A = ALLOWABLE AXIAL PILE LOAD CAPACITY OF A SINGLE BATTER PILE ESTIMATED AS FOLLOWS:

$$A = \sqrt{V^2 \left(1 + \frac{1}{B^2}\right)}$$

NOTE: THE AXIAL LOAD RESISTANCE OF A VERTICAL PILE, V, IS DEPENDENT ON THE TYPE OF LOADING--TENSION OR COMPRESSION. CAUTION SHOULD BE EXERCISED TO INSURE THAT THE CORRECT VERTICAL CAPACITY IS USED.

CAPACITY OF PILE GROUPS

The maximum allowable load carrying capacity of a pile group is no greater than the sum of the single pile load capacities, but may be limited to a lower value if so indicated by the result of the following formula.

$$Q_a = \frac{P \times L \times c}{(FSF)} + \frac{2.6\, q_u \,(1 + 0.2\, \frac{w}{b})\, A}{(FSB)}$$

In Which:

Q_a	=	Allowable load carrying capacity of pile group, lb
P	=	Perimeter distance of pile group, ft
L	=	Length of pile, ft
c	=	Average (weighted) cohesion or shear strength of material between surface and depth of pile tip, psf
q_u	=	Average unconfined compressive strength of material in the zone immediately below pile tips, psf (unconfined compressive strength = cohesion x 2)
w	=	Width of base of pile group, ft
b	=	Length of base of pile group, ft
A	=	Base area of pile group, sq ft
(FSF)	=	Factor of safety for the friction area = 2
(FSB)	=	Factor of safety for the base area = 3

The values of c and q_u used in this formula should be based on applicable soil data shown on the Log of Boring and Test Results for this report. In the application of this formula, the weight of the piles, pile caps and mats, considering the effect of buoyancy, should be included.

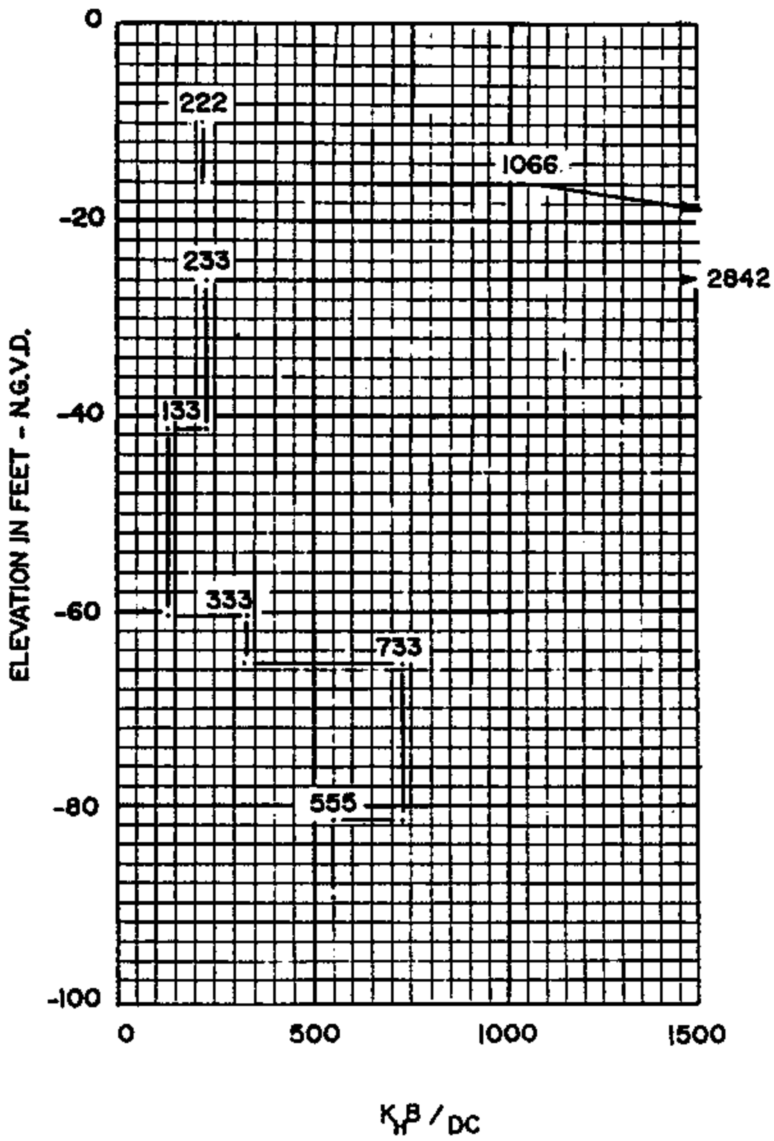
SPACING OF PILE GROUPS

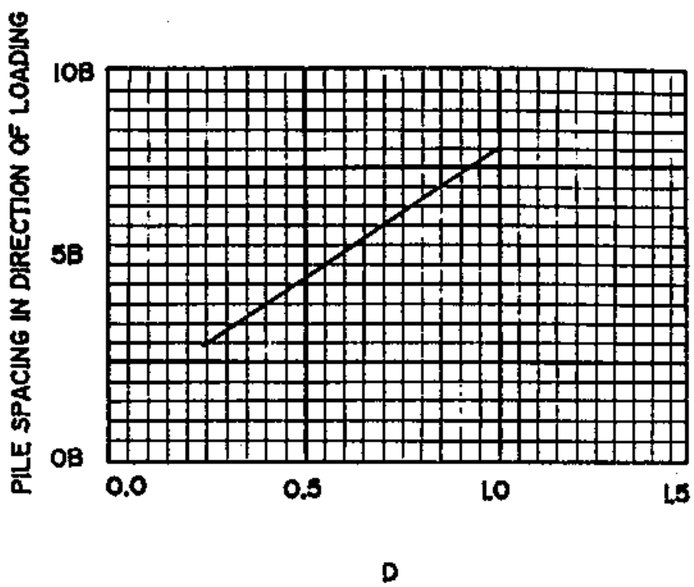
SPAC = $0.05\,(L_1) + 0.025\,(L_2) + 0.0125\,(L_3)$

In Which:

SPAC	=	Center to center of piles, feet
L_1	=	Pile penetration up to 100 feet
L_2	=	Pile penetration from 101 to 200 feet
L_3	=	Pile penetration beyond 200 feet

NOTE: Minimum pile spacing = 3 feet or 3 pile diameters, whichever is greater





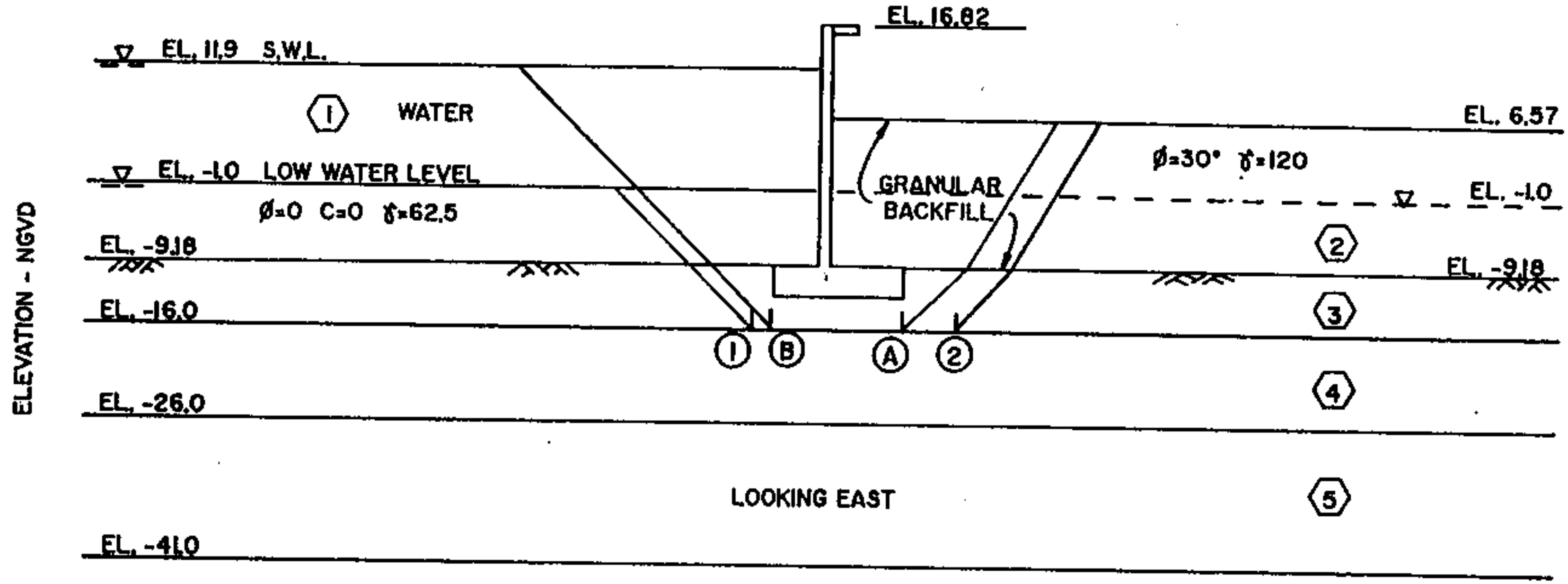
WHERE :

C = 0.5 FOR CYCLIC LOADING
C = 1.0 FOR INITIAL LOADING
B = PILE WIDTH OR DIAMETER - INCHES
D = GROUP EFFECT REDUCTION FACTOR
K_H = MODULUS OF HORIZONTAL SUBGRADE REACTION - LBS/IN³

SUBGRADE MODULI

LONDON AVENUE OUTFALL CANAL
FRONTAL PROTECTION AT
PUMPING STATION NO. 3
NEW ORLEANS, LOUISIANA

T - WALL @ STA. 0+62 TO STA. 0+87



SCALE : 1" = 20'

FAILURE SURFACE	SUMMATION OF FORCES LBS/FT DRIVING	RESISTING	FACTOR OF SAFETY
A 1	20,525	26,975	1.31
B 2	-5,089	47,590	O.K.*

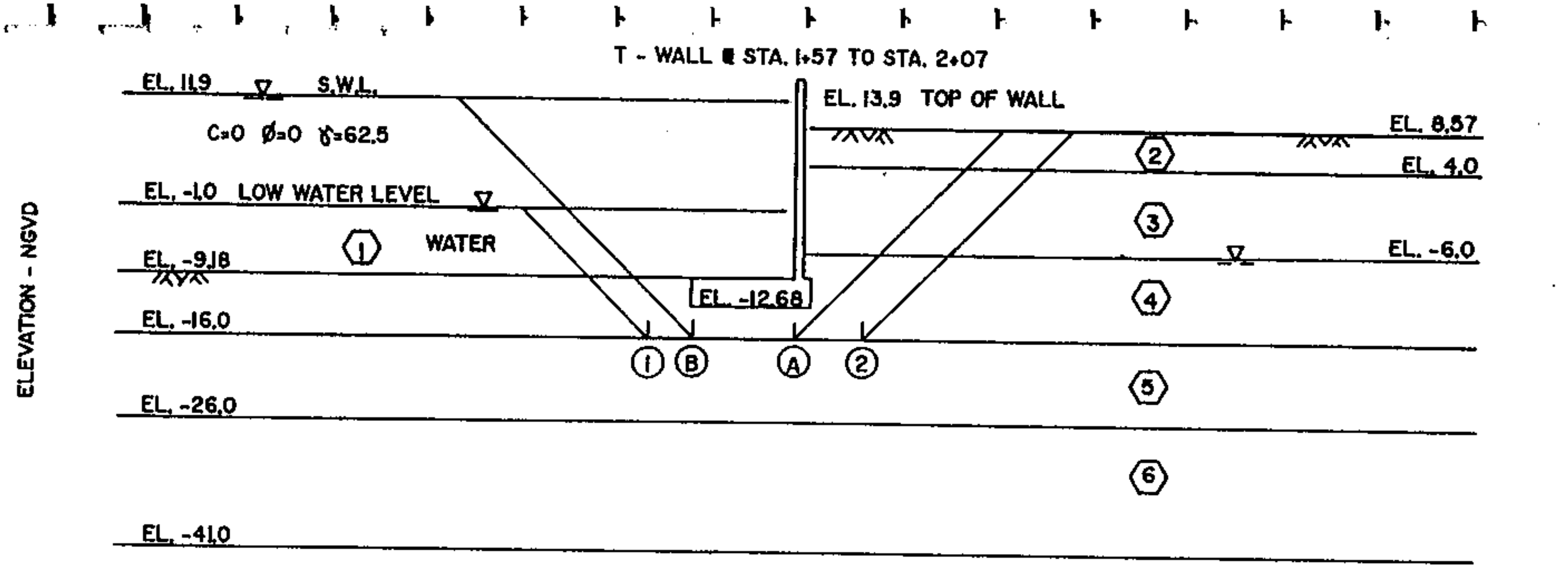
⬡ STRATA NUMBER, SEE FIGURE 2 FOR SOIL DESIGN PARAMETERS BELOW EL. -9.18 NGVD

* PASSIVE DRIVING FORCES > ACTIVE DRIVING FORCES

STABILITY ANALYSES

LONDON AVENUE OUTFALL CANAL
FRONTAL PROTECTION AT
PUMPING STATION NO. 3
NEW ORLEANS, LOUISIANA

EUSTIS ENGINEERING COMPANY, INC.



LOOKING SOUTH
SCALE : 1" = 20'

FAILURE SURFACE	SUMMATION OF FORCES LBS/FT DRIVING	RESISTING	FACTOR OF SAFETY
Ⓐ ①	25,323	41,880	1.66
Ⓑ ②	-8,945	46,159	O.K.*

⬡ STRATA NUMBER, SEE FIGURE 2 FOR SOIL DESIGN PARAMETERS

* PASSIVE DRIVING FORCES > ACTIVE DRIVING FORCES

STABILITY ANALYSES

LONDON AVENUE OUTFALL CANAL
FRONTAL PROTECTION AT
PUMPING STATION NO. 3
NEW ORLEANS, LOUISIANA

EUSTIS ENGINEERING COMPANY, INC.

FIGURE 8