LAKE PONTCHARTRAIN, LOUISIANA
AND
VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A
GENERAL DESIGN

LONDON AVENUE OUTFALL CANAL
SUPPLEMENT NO. 2

FRONTING PROTECTION
DRAINAGE PUMPING STATION NO. 3

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PROJECT HISTORY

1. Background

A. Two plans for hurricane protection for the London Avenue Outfall Canal are presented in the report entitled *Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 19A - General Design, London Avenue Outfall Canal, (January 1989)* (DM 19A). One plan concept, the one recommended, is to provide fronting protection at or near the lakefront end of the canal by the construction of a gated structure. The other plan, the one supported by the Board of Commissioners of the Orleans Levee District (OLD), will require upgrading the height of the existing 2.4 miles of parallel levees along both sides of the canal. The latter plan will also require flood-proofing 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 Station No. 3 (DPS#3) and Drainage Pumping Station No. 4 (DPS#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 will 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. The fronting protection project at Drainage Pumping Station No. 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 DPS#3 and is prepared in sufficient detail to provide an adequate basis for preparing the construction plans and specifications.

3. Project Location and Description

A. 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. The station houses three 1,000 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 pump being taken out of service at a time.

Typical operations consist of running one or more of the 20 cfs constant-duty pumps to accommodate dry weather flow. The constant duty pump discharge is directed into the Florida Ave. canal. As the need for pumping capacity increases, the 1,000 cfs pumps are primed by vacuum and brought on line. Similarly, the 500 cfs pumps are primed and started. Gates can operated to allow intake flow to bypass the station and flow into the Florida Ave. Canal via a bypass canal on the east side of the station.

C. The foundation of the station consists of a reinforced concrete slab supported by timber piles. The discharge basin slab is also pile supported. Water depth in the discharge basin is always approximately ten feet, subject to the tidal fluctuations of Lake Pontchartrain.

PROJECT PLAN

4. Flood Protection Plan

The Sewerage and Water Board of New Orleans has mandated that this flood protection project be accomplished by utilizing a sluice gate structure with concrete discharge tube structures. This recommended plan was agreed upon after several coordination meetings with the Sewerage and Water Board of New Orleans. The structure will be constructed approximately 125 feet north of the station and each of the five horizontal pumps will have an individual concrete discharge tube adjoining the sluice gate structure. There will be two sluice gates at the termination of each concrete discharge tube for a total of ten 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 fronting protection across the entire width of the canal, approximately 125 feet north of the existing station. Portions of the existing concrete discharge basin slab will be removed in the areas where the new sluice gate control structure is to be constructed (see Plates 3, 8 & 14). Pile-founded reinforced concrete T-walls and reinforced concrete capped steel sheet pile I-walls will tie the new protection to the existing protection.

Each horizontal pump will be provided with its own reinforced concrete discharge tube. Each reinforced concrete discharge tube will be fronted by two gates. Discharge tubes will be grouped together into two major discharge structures; one for the 500 cfs Pumps A & B and the second for the 1,000 cfs Pumps C, D, & E. A four gate control structure and a separate six gate control structure will be constructed at the ends of the two discharge structures for the 500 cfs and 1,000 cfs pumps, respectively (see Plates 3 through 6, 10 through 12, 18 & 19). The ten sluice gates will provide emergency closure capabilities in the event of pump failure. Power for all gate operators shall be supplied from the existing "T2" power panel within DPS#3.

Two T-walls monoliths will be constructed to connect the existing canal floodwalls to the ends of the new gate control structures. The gate control structures will be joined together at the center of the discharge basin by another T-wall monolith. Two I-wall monoliths will join the existing I-walls at the Norfolk-Southern railroad flood gates to the proposed construction.

A. <u>Sluice Gate Monolith for 500 cfs Pumps (G-1)</u>

A gated monolith will be utilized in front of the discharge area of 500 cfs Pumps A & B (see plates 3 through 6, 18 & 19). This monolith will house four 81" x 96" cast iron sluice gates. The structure will be reinforced concrete, with a top elevation of 13.90 NGVD (34.33 CD), founded on steel HP14x73 piles. Steel H-piles will be used in lieu of prestressed concrete piles because they tend to penetrate the soil with less diving effort and less vibration; thereby minimizing vibration effects to the existing pump station structure. The operating floor will be constructed from reinforced concrete with aluminum handrails and galvanized steel bar grate openings to allow access to the gate hoisting assemblies (see Plates 4 through 6). Placement of reinforcing steel, embedded steel items, construction joints and water stops will conform to construction industry standards. Expansion joints, where required between monoliths, will include ½ inch joint filler.

Stoplog slots for dewatering 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 1,000 cfs Pumps (G-2)

A gated monolith will be utilized in front of the discharge area of 1,000 cfs Pumps C, D & E (see Plates 3, 10 through 12, 18 & 19). This monolith will house six 108" x 96" cast iron sluice gates. The construction, operation and maintenance of this monolith is similar to the preceding paragraph A.

C. Concrete Discharge Tubes

Each horizontal pump will be provided its own reinforced concrete discharge tube approximately 90 feet long connecting the pump discharge pipe to the sluice gate structure (see Plates 3, 8 & 14). As previously stated, discharge tubes will combined into two major discharge structures; these structures will be founded on steel HP14x73 piles. The highest floor elevation inside the discharge tubes (hump) is El 6.61 NGVD (27.04 CD). 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 1,000 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 1,000 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. Gates

Six 108" x 96" cast iron sluice gates and four 81" x 96" cast iron sluice gates will be required. Each gate will be equipped with an electric motor operator and a manual operator to allow operation in the event of power failure. The sluice gates, motors and operators shall satisfy Sewerage and Water Board of New Orleans requirements.

E. T-Wall Monoliths

Three T-wall monoliths will be required (see Plates 3, 9, 18, 20, 22 & 24). One closure monolith will connect the four-gate structure and the six-gate structure at the center of the discharge basin. The other closure monoliths will connect 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 13.90 NGVD (34.33 CD), founded on steel HP14x73 piles, with PZ-22 or equal steel sheet pile seepage cut-off. Steel H-piles are used instead of prestressed concrete piles because of simpler handling and splicing requirements during placement and driving.

F. <u>I-Wall Monoliths</u>

Two I-wall monoliths will be required (see plates 3, 18 & 19). Each I-wall will connect existing I-walls on each side of the London Ave. Outfall Canal to the ends of the T-walls crossing the canal. The I-type floodwall will consist of steel sheet piles capped with a reinforced concrete wall. The top elevation will be 14.40 NGVD (34.83 CD). This elevation will be ½ foot higher than the adjacent HP pile founded T-wall monoliths to allow for I-wall settlement. Steel sheet piling sizes will include the existing and new PZ-22 or equal. Expansion joints in the floodwall will be spaced to occur at the steel sheet pile interlocks, approximately 30 feet apart, and at each change in I-wall direction.

G. <u>Walkways and Operating Floor</u>

The operating floor for sluice gate structures will be constructed from reinforced concrete with formed openings to allow access to the gate hoisting assemblies. All access openings shall have removable galvanized steel bar grate sections for access panel assemblies (see Plates 4 through 6, 10 through 12 & 15). Aluminum handrails and posts will be installed along the perimeter of each operating floor and along both sides of walkway joining the two sluice gate monoliths. Access to sluice gate monoliths will be via a stairway adjacent to concrete discharge structure for 500 cfs pumps A & B and via stairway between the 500 cfs pump and 1,000 cfs pump discharge structures. Placement of reinforcing steel, embedded steel items, construction joints and water stops will conform to construction industry standards. Expansion joints, where required between monoliths, will include ½ inch joint filler.

H. <u>Dewatering Bulkheads</u>

Provisions for dewatering the sluice gate monoliths are provided. Dewatering bulkheads constructed of single, solid panels (stop logs) designed to fit into the gate monolith stop log slots. The stoplog panels will be structural aluminum and will provide water retention to canal water El 4.00 NGVD (24.43 CD).

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). The top of the dam will be El 1.57 NGVD (22.00 CD) as mandated by the Sewerage and Water Board of New Orleans. The dam will consist of PZ-38 or equal cantilevered steel sheet piles with four 66" x 66" electrically operated butterfly gates valves. A flap plate, located at the top of the temporary dam, below the catwalk, will allow pump discharge flow to spill over the dam at El 1.57 NGVD (22.00 CD), but will check flow in the reverse direction. This will reduce the maximum pump head and at the same time provide protection of the dewatered work area in the event the flood side water surface rises above El 1.57 NGVD (22.00 CD). There will also be an access walkway at El 4.0 NGVD attached to the dam to allow manual operation of the butterfly and flap gates.

The butterfly gates will allow canal water to flood the dewatered work area (existing discharge basin) so that the pump discharge bells can be rapidly submerged. The pump discharge bells must be sealed in order to allow priming of the horizontal pumps. Once the pumping begins, if there is more flow than the open butterfly gates can accommodate, the excess flow will spill over the top of the temporary dam, through the flap gate.

J. Temporary Concrete Weir

A temporary weir will be constructed in the discharge basin for 500 cfs pumps A & B to keep these two discharge bells sealed at all times. In this way, these two horizontal pumps can be primed and loaded immediately to assist in filling up the dewatered work area, if required. In this manner, the work area can be flooded in less time than by just the four butterfly gates acting alone. The constant duty flow from the four 20 cfs pumps will maintain water within the weir, effectively maintaining the water seal on both discharge bells of 500 cfs pumps A & B. Excess constant duty flow will spill over the top of the weir and will be removed by site dewatering pumps.

K. Existing Canal Lining

Portions of the reinforced concrete lining of the London Ave. Outfall Canal will be removed to allow construction of the fronting protection. Those portions 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 the preliminary determination of size and proportions of various structural concrete components, reinforcing of these components, sizes and thickness of structural steel elements and preliminary pile layouts for the fronting protection structures and the horizontal pump discharge structures 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 these computations 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 to develop the final design of this project.

7. References

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

A. USACE 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 (ACI), 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 (AWS), Structural Welding Code, Steel, (AWS-D 1.1-88).

C. Computer Programs:

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

(2) WES Program X0030, "CFRAME"

(3) STAAD-III/ISDS, Finite Element Analysis 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 publications listed in previous paragraphs.

B. Material Weights

Densities for different materials used in the design computations are listed in Table 1 below:

Table 1 - Material Densities

Material	Unit Weight (lbs/cu.ft.)
Water	62.5
Normal Weight Concrete	150.0
Steel	490.0
Saturated Sand	122.0
Saturated Granular Backfill	122.0

C. Design Stresses

(1) Structural Steel: Allowable stresses shall be in accordance with AISC, Manual of Steel Construction,

Allowable Stress Design, as modified by EM 1110-1-2101.

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

(4) Reinforced Concrete

(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 0.25 ρb.

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

9. Loading Conditions

A. General

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

(1) Usual Loading Condition:

SPH with SWL @ El 11.9 NGVD (32.33 CD)

(2) Unusual Loading Condition:

SWL 2'-0" above SPH water level, i.e., @ El 13.9 NGVD (34.33 CD)

B. Discharge Tube Structures

The discharge tube structures for 500 cfs pumps A & B and 1,000 cfs pumps C, D & E were designed for both positive and negative hydrostatic pressures. The negative hydrostatic pressure resulting from vacuum 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 the impeller of 1,000 cfs 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.

C. Sluice Gate Structures

Sewerage and Water Board's normal operations require the sluice gates for the 500 cfs pumps A & B and 1,000 cfs pumps C, D & E at the north end of the discharge tube structures remain open at all times. In the event that one or more pumps become inoperable, and the water level in London Ave. Canal is at or above the high point in the discharge tube(s) EL 6.61 NGVD (27.04 CD), the gates for the affected discharge tube(s) will be closed to prevent backflow into the suction basin.

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

(1) Usual Conditions:

(a) Gates closed, canal SWL @ El 11.9 NGVD (32.33 CD) water level in discharge tube @ El 6.61 NGVD (27.04 CD), 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 CD) water level in discharge tube @ El 6.61 NGVD (27.04 CD), 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 CD), water level in discharge basin @ El -9.18 NGVD

(11.25 CD), operating wind, backfill in place; impervious sheet pile cut-off.

(b) Same as above but pervious sheet pile 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 CD), storm wind, backfill in place; impervious sheet pile cut-off.

(b) Canal SWL @ El 11.9 NGVD (32.33 CD), storm wind, backfill in place; pervious sheet pile cut-off.

(2) Unusual Conditions:

(a) Canal SWL @ El 13.9 NGVD (34.33 CD), storm wind, backfill in place; impervious sheet pile cut-off.

(b) Canal SWL @ El 13.9 NGVD (34.33 CD), storm wind, backfill in place; pervious sheet pile cut-off.

(c) Canal SWL @ El -9.18 NGVD (11.25 CD), operating wind, backfill in place; impervious sheet pile cut-off.

(d) Canal SWL @ El -9.18 NGVD (11.25 CD), operating wind, backfill in place; pervious sheet pile cut-off.

(3) Construction Condition:

(a) Completed T-wall in place, no backfill, no water in canal, no wind.

10. Structural Design

As indicated previously, analyses and design methods were simplified for the purpose of arriving at preliminary proportions, thicknesses and reinforcement 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. Sluice Gate Structures

Design computations were performed for Sluice Gate Structure G-2, at the discharge of 1,000 cfs pumps C, D & E only (see Plate 23). Sluice Gate Structure G-1, at the discharge of 500 cfs pumps A & B functionally identical, but smaller in plan and subjected to same loading intensities as Sluice Gate Structure G-2. Reinforcement and structural proportions identical to Sluice Gate Structure G-2 were assumed for Sluice Gate Structure G-1 without further computations (see plate 21). Pile layout, however, was determined separately for each foundation.

Different components of Sluice Gate Structure G-2 were designed as follows:

(1) Longitudinal Walls (North & South Walls):

Longitudinal walls were assumed to be continuous over transverse walls and simply supported at exterior transverse walls. Further, it was assumed that the longitudinal wall will transfer loads horizontally to the transverse walls. Reinforcement in these walls was determined based on the flexural stresses caused by out-of-plane loading on these walls.

(2) Transverse Walls:

Interior transverse walls were assumed to be fixed at the base slab, continuous over the slab @ El 4.82 NGVD (25.25 CD) 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 4.82 NGVD (25.25 CD) and the operating platform at El 16.82 NGVD (37.25 CD). Both in-plane and out-of-plane loadings were used to determine flexural and shear stresses. The exterior transverse walls were also designed in similar fashion.

(3) Columns:

The column between the gates for 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, on flood side of the gates, was also designed in similar fashion.

(4) Base Slab:

The base slab was assumed to be simply supported at exterior transverse walls of the gate structure and continuous at interior walls and columns in the longitudinal direction. 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 reinforcement was provided based on these assumptions. Adequacy of the base slab to span longitudinally over the piles was also checked. Minimum flexural reinforcement was assumed to be sufficient in the transverse direction for the purpose of the preliminary design.

B. T-Wall Monoliths

T-wall monoliths were designed as pile supported cantilever retaining walls.

(1) Stem:

The T-wall stem was assumed to be 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 reinforcement.

(2) Base Slab:

The base slab was designed to transfer loads horizontally to the piles. It was assumed to be fixed at the face of stem with vertical components of pile loads acting as point loads at the pile locations.

C. Discharge Tube Structures

Design calculations were performed for the discharge tubes at 1,000 cfs pumps C, D & E only. The top, bottom and foundation slabs span shorter distances in the structure for 500 cfs Pumps A & B. Both structures are subject to the same loading intensities. Reinforcement and structural proportions identical to the structure for 1,000 cfs pumps C, D & E were selected for the structure at 500 cfs pumps A & B without further computations. Pile layout computations for each were performed separately (see Plates 25 & 26).

(1) Top Slab:

The top slab of the discharge tube structure was designed as a continuous slab simply supported at exterior walls the tubes and continuous over the 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:

Exterior and interior walls of the discharge tube structure were designed as compression members subjected to combined axial and bending stresses. Both in-plane and out-of-plane loadings were used to determine reinforcement.

(3) Bottom Slab:

The bottom slab of the discharge tube structure was assumed to behave in a manner similar to the top slab and was designed accordingly.

(4) Foundation Slab:

The foundation slab of the discharge tube structure was designed as a continuous slab over interior walls and simply supported at the exterior walls of the discharge tubes. Pile loads were assumed to act as point loads on the foundation slab at the pile locations. Out-of-plane loading was used to determine flexural and shear stresses.

D. Pile 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.

11. Temporary Steel Sheet Pile Dam

The temporary steel sheet pile dam at Treasure Street was designed as a free-standing cantilevered wall constructed of PZ-38 steel sheet piles having a section modulus of 46.8 cubic inches per foot of wall. The mud line was set at El -10.00

NGVD (10.43 CD) on both sides of the dam. Water surface elevations were set at El 1.57 NGVD (22.00 CD) and -10.00 NGVD (10.43 CD) on the flood side and protected side, respectively. A factor of safety of 1.0 was applied to soil shear strengths to determine the maximum bending moment and a factor of safety of 1.5 was applied to soil shear strengths to determine the design tip embedment.

Analysis were performed by Eustis Engineers, Inc., using the "CWALSHT" computer program furnished by the USACE to determine the estimated deflection of the steel sheet piles. The results of these analysis indicate a maximum deflection of 4.96 inches at the top of the wall and 2.64 inches at the mud line. Results of the analysis are included in Appendix B.

12. 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 will be provided.

METHOD OF CONSTRUCTION

13. General

All construction will be performed in dry conditions behind the temporary sheet pile dam. The contractor is expected to vacate the work area during all rain events in which the pumps must be operated. Only one 1,000 cfs pump will be permitted to be taken out of service at any time during the entire construction process. Due to this constraint, the discharge structure for the three 1,000 cfs pumps will be constructed in segments; provisions for construction joints and reinforcement splicing will be included in the final design. The construction easement shall include the vacant property west of the existing discharge basin. All electrical utility and plumbing piping relocations will be coordinated with the Sewerage and Water Board.

14. Suggested General Construction Sequence

A. Construct the temporary cantilevered steel sheet pile dam and butterfly control gates across the London Ave. canal.

(1) Construct a cantilevered steel sheet pile dam at USACE Sta. 2+58 E/BL (Treasure Street) across the canal with sill El 1.57 NGVD (22.0 CD) with four 66" square butterfly gates; then dewater area between the 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 the bottom lip of the highest discharge hood, to keep both hoods sealed at all times. This is required to allow the dewatered work area to be flooded within a 15 minute period so that pumps C D & E can be primed by vacuum. Pumps A & B are to be kept in service until all three 1,000 cfs pumps have been returned to service with their respective concrete discharge tubes.

(3) Butterfly valves are to be opened to flood the work area to seal remaining pump C D & E discharge hoods, within a 15 minute period; and will be left open until pumping has stopped and all station pumps are shut down.

B. Relocate existing utilities in conflict with the proposed construction.

(1) Relocate the 48"ϕ sewer force main to new permanent location along the south side of the Norfolk-Southern RR bridge right-of-way. The relocated pipe will be 48"ϕ fiberglass pipe, supported on concrete beams connected to concrete cap bents.

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

C. Construct the new sluice gate control structure monoliths G-1 and G-2 for 500 cfs pumps A & B and 1,000 cfs pumps C, D & E, respectively. Construct T-wall monoliths T-1 and T-2.

(1) Break out the bottom slab of the existing discharge basin to allow construction of the two sluice gate control structures and three T-wall monoliths across full the width of canal.

(2) Drive all steel HP14x73 foundation piling, place reinforcing steel and cast reinforced concrete base slabs of the two sluice gate control structures and two T-wall monoliths.

(3) Construct the walls of two sluice gate control structures. The sluice gates will be operational at the completion of this step. Construct the stem walls of the two T-wall monoliths.

D. Construct the fist portion of the discharge tube structure for 1,000 cfs pumps, the discharge tube for Pump C.

(1) Take 1,000 cfs Pump C out of service.

(2) Remove the discharge piping, including discharge hood, from the pump flange inside the pump house.

(3) Close the new sluice gates for Pump C at the new sluice gate control structure.

(4) Drive temporary steel sheeting on east side of the proposed concrete discharge tube and the space between the discharge hoods for Pump D and Pump C. The steel sheeting will connect to the pump house building and to the sluice gate control structure at respective its ends (see Plate 29).

(5) Construct that portion of the reinforced concrete discharge structure (single discharge tube) for Pump C and install the steel transition section between the pump flange and the concrete discharge tube.

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

(7) Remove the temporary steel sheeting on the east and west sides of the new concrete discharge tube for Pump C.

E. Construct the second portion of the discharge tube structure for 1,000 cfs pumps, the discharge tube for Pump D.

(1) Take 1,000 cfs Pump D out of service.

(2) Remove the discharge piping, including discharge hood, from the pump flange inside the pump house.

(3) Close the new sluice gates for Pump D at the new sluice gate control structure.

(4) Drive temporary steel sheeting between the discharge hoods for Pump D and Pump E, to permit construction of the discharge tube for Pump D. The steel sheeting will connect to the pump house building and to the sluice gate control structure at respective its ends (see Plate 29).

(5) Construct that portion of the reinforced concrete discharge structure (single discharge tube) for Pump D and install the steel transition section between the pump flange and the concrete discharge tube.

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

(7) Remove the temporary steel sheeting on the west side of the new concrete discharge tube for Pump D.

F. Construct the third and last portion of the discharge tube structure for 1,000 cfs pumps, the discharge tube for Pump E.

(1) Take 1,000 cfs Pump E out of service.

(2) Remove the discharge piping, including discharge hood, from the pump flange inside the pump house.

(3) Close the new sluice gates for Pump E at the new sluice gate control structure.

(4) Complete the reinforced concrete discharge structure (single discharge tube) for Pump E and install the steel transition section between the pump flange and the concrete discharge tube.

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

G. Perform relocation of constant-duty pump discharge and modifications to the Marigny and London Ave. gates.

(1) Close the new sluice gates for Pump A and Pump B in sluice gate control structure.

(2) Take 500 cfs Pumps A & B out of service.

(3) Remove the discharge piping, including discharge hood, from the pump flange inside the pump house.

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

(5) Construct a temporary low sill dam on east side of the Marigny Gate to prevent backflow from the Florida Avenue Canal. Remove the existing butterfly gate (Marigny Gate). Seal the existing gate opening with a concrete retaining wall and

provide a sleeve for the constant-duty pump discharge piping.

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

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

(8) Remove the existing west retaining wall from the London Ave. Gate to the pump house building (see Plate 2).

H. Construct the discharge tube structure for the two 500 cfs pumps.

(1) Construct the reinforced concrete discharge tube structure for 500 cfs Pumps A & B and install the steel transition sections between the pump flanges and the concrete discharge tubes.

(2) Restore Pumps A & B to service.

I. Construct T-Wall monolith T-3 on the west side of the London Ave. canal.

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

(2) Drive the new steel HP14x73 piles at the required locations.

(3) Construct the reinforced concrete T-wall foundation slab and stem wall.

(4) Restore the removed concrete canal bottom slab.

J. Construct the two I-wall monoliths, on each side of the discharge basin and tie into the existing flood protection I-walls.

K. Remove and salvage the temporary sheet pile dam and 66" square butterfly gate valves at Treasure Street and repair the removed concrete canal bottom lining.

ACCESS ROADS

15. Vehicular Access

Vehicular access to the project site is available via public streets. Public streets adjacent to the project site are Abundance Street., 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 Street 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 1,300 ft. north of the station.

RELOCATIONS

16. 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...".

17. Utility Relocation

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

A. 48" Diameter Sewer Force Main

The 48" diameter sewer force main will be relocated and reconstructed of 48"ϕ fiberglass pipe in its permanent location at the south side of the right-of-way of the Norfolk-Southern railroad tracks, in order to clear the area where the sluice gate structure will be constructed. This relocation will require approximately 355 feet of pipe (see Plate 3).

As mandated by the Sewerage and Water Board of New Orleans, 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 red-dyed concrete encased PVC conduit duct banks with blank spares, and in accordance with the requirements of the Sewerage and Water Board of New Orleans with respect to relocation and routing.

(3) All new cable will be provided for feeders 400, 340 and 432. The new cable shall be 500 MCM lead covered, three conductor, 15-KV, EPR cable conforming to the specifications of the Sewerage and Water Board of New Orleans. Feeder 506 will be of the same materials, 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 building will be temporarily re-routed along the eastern half of the wall to east end of the pump house 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 through conduits embedded in the sluice gate control structures and duct banks, as required by Sewerage and Water Board of New Orleans (see Plate 28).

C. Telephone Cable

The existing South Central Bell aerial telephone cable serving DPS#3 will relocated. Telephone cables currently enter the building on the north wall of the station near Pump C. These lines are in direct conflict with the proposed work, as they span the work area over the existing discharge basin.

D. Power Poles

The three existing S&WB power poles which are located within in the levee to be degraded along the west side of the discharge basin will temporarily relocated and eventually removed. The electrical and communication lines carried over the railroad tracks by these poles will be re-routed 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 Street will be de-energized during the construction and dismantling of the temporary sheet pile dam. The proximity of these overhead electrical lines to the pile driving leads during installation of the sheet piles causes an unsafe condition, should these lines remain energized. Construction and dismantling of the temporary sheet pile dam is to be coordinated with NOPSI.

MECHANICAL

18. General

The design of the mechanical systems for the fronting protection will include provisions for ten electrically operated sluice gate assemblies with manual override and one electrically operated valve with manual override to flush out the Florida Ave. Canal. The temporary sheet pile dam will also have four electrically operated butterfly gates with manual override to flood the work area to allow priming the 1,000 cfs pumps.

The design of mechanical systems 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.

19. 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 sluice gates will be by individual electric actuators. Each gate will require approximately ten minutes to fully open or close. Typically, in an emergency, all gate actuators will be operated simultaneously. 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. Three portable air motors or one portable generator with sufficient power to operate three gates simultaneously will be provided. The maximum worst-case sluice gate operating time, from fully open to fully closed positions, is approximately 40 minutes for all ten gates.

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 while opening or closing. 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.

20. Vacuum Pumps

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

Connection of the new vacuum pumps to existing vacuum lines will be accomplished so as not to render the existing vacuum system inoperable. Only one existing vacuum pump may be taken out of service at a time. Existing valves will be closed and lines plugged where they will be cut to maintain the vacuum system in an operating condition at all times.

ELECTRICAL DESIGN

21. General

The design of the electrical system for the ten gate motors and controllers will include provisions for power and control. The electrical design is based on space, conduit routing, power source and availability criteria established by the Sewerage and Water Board of New Orleans; and on the use of equipment and material that are available as standard products of the electrical industry. Gate operation procedures will require that gates be operated individually. 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.

22. Power Sources & Distribution

A. General

The station primary power supply for the main pumps is a 6,600-V, 25-Hz, 3-phase service generated by the Sewerage and

Water Board of New Orleans. Lighting and convenience outlets are supplied with 120-V, 60-Hz electrical service. The Sewerage and Water Board of New Orleans requests that power to all other motors be furnished as 240-V, 25-Hz, 3-phase electrical service.

B. Electrical Loads

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

(2) Sluice Gate Operators

(a) Power for the ten sluice gate operators will be provided from the existing "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 240-V, 25-Hz, 3-phase motors.

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

(3) Temporary Butterfly Gates

(a) Power for the four butterfly gate operators for the temporary sheet pile dam will 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, will be 120-V, 60-Hz, controlled by a photo-cell contactor arrangement. Lamps will be high pressure sodium vapor (HPS), 250-W, pole mounted.

(4) Fresh Water Flush Valve

One gate operator will be required for the 4'ϕ fresh water (lake intake) flush valve. Power for the gate operator for the fresh water flush valve will be the same as that for the sluice gate operators in Item (2) above.

(5) Voltage Drop Requirements

Conductors will be sized to limit voltage drops from exceeding 3 percent at the furthest utilization point of each circuit.

23. Conduit and Boxes

A. Conduit

All above ground and interior wiring will 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 affording 3" minimum cover. In some areas, as directed by the Sewerage and Water Board of New Orleans, feeder cables will be run in concrete duct banks.

B. Pull and Junction Boxes

All pull boxes 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.

24. Gate Motor Operator Controls

Local gate operator controls will be located on the operator and will include stop/open/close push-buttons with end-of-travel pilot lights.

Remote gate operator controls will be located on the DPS#3 Auxiliary System Control Console and will consist of open/close push-buttons and end-of-travel pilot lights for each operator.

HYDROLOGY AND HYDRAULICS

25. 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.

26. 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 of New Orleans mandated that the flood protection be accomplished by utilizing a sluice gate structure with concrete discharge tubes. The elevations of the top of the temporary sheet pile dam and the hump inside the concrete discharge tubes were both set by the Sewerage and Water Board of New Orleans, from their many decades of experience in operating DPS#3 and the other drainage pumping stations which discharge into canals leading into Lake Pontchartrain.

GEOLOGY

27. General

A. Scope

The geology presented herein is based on the geology presented in *Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, 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 supplements the data from DM 19A. It is intended to present herein a general project overview of the pertinent geologic data and its interpretation.

B. Physiography and Topography

The project site is located within the Central Gulf Coastal Plain region, on the flanks of the Mississippi River Deltaic Plain, and is 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 -15.00 NGVD (5.43 CD) in Lake Pontchartrain to 20.00

NGVD (40.43 CD) 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" diameter, undisturbed soil boring, 125 feet in depth, was made on August 4, 1994 under A-E contract. The USACE drilled two undisturbed borings in 1971. Two A-E contract borings were made in 1985. Information from all five subsurface investigations was utilized in the analyses. The results of all five borings are presented in The Soil Boring Profiles (Plate 31) in order to present the most geologically complete representation. All borings encountered artificial fill and Holocene soils. Those borings exceeding 70 foot depths 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.

28. Regional Geology

Reference *Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 19A - General Design, London Avenue Outfall Canal, (January 1989)* for information on regional geology.

29. 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 NGVD (-44.57 CD). The depth to the 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. Detailed Holocene Environmental Descriptions

(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 -50.00 NGVD (-29.57 CD). 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. Detailed Pleistocene Soil Descriptions

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 in the Soil Boring Profiles 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.

30. 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

31. 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 herein. 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.

32. 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.

33. Field Exploration

One A-E undisturbed sample type soil test boring, 125 feet in depth, was made on August 4, 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" diameter thin-wall 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" 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.

34. 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.

35. Description of Subsoil Conditions

A. Topography

On the west side of the canal, Boring 1-LUW is at El 3.5 and Boring 1 (1985) is at El 4.00 NGVD (24.43 CD). On the east side of canal, Boring 36 is at El 10.00 (30.43 CD) and Boring 2-LUE is at El 7.00 NGVD (27.43 CD). At the southern end of the canal, Boring 1 (1994) is at El 0.00 (20.43 CD).

B. Geology

Recent Holocene deposits overlie older Pleistocene deposits. Upper Holocene soils are deltaic plain deposits that overlie near-shore Gulf deposits. Near-shore 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 -13.00 NGVD (7.43 CD) to -17.00 NGVD (3.43 CD). 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 -23.50 NGVD (-3.07 CD) to -27.50 NGVD (-7.07 CD). The third stratum consists of prodeltaic deposits of CH soil to El -40.00 NGVD (-19.57 CD) to -43.00 NGVD (-22.57 CD). Deposits to these depths form the deltaic plain. Deltaic plain deposits appear normally consolidated. The deltaic plain is underlain by near-shore Gulf deposits of SP, SM, SC and CL soils to El -57.00 NGVD (-36.57 CD) to -62.00 NGVD (-41.57 CD). Beneath this, near-shore Gulf deposits of predominantly CH soil continue to El -63.50 NGVD (-43.07 CD) to -67.5 NGVD (-47.07 CD). Near-shore Gulf deposits appear slightly precompressed.

(2) Pleistocene:

The geologically identified Pleistocene formation begins at El -63.50 NGVD (-43.07 CD) to -67.50 NGVD (-47.07 CD). 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

-88.50 NGVD (-68.07 CD). Pleistocene deposits continue to the final boring depths of 75 to 125 feet below the existing ground surface El -71.50 NGVD (-51.07 CD) to -125.00 NGVD (-104.57 CD).

D. Groundwater

Observations of the groundwater were made during the field investigation on August 4, 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 -6.00 NGVD (14.43 CD). 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.

36. Foundation Analysis

A. Furnished Information

A temporary sheet pile dam with four 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 -1.00 NGVD (19.43 CD) and hurricane level is El 13.90 NGVD (34.33 CD). The bottom of the discharge basin is at El -9.18 NGVD (11.25 CD).

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

(1) 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 -10.00 NGVD (10.43 CD). 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) 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 -80.43 NGVD (-60.00 CD). 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.

(4) 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 percent 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.

(5) 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. Dynamic pile testing is recommended for steel H-piles used on the project.

Information derived from static and dynamic testing can be used during construction to evaluate the capacity of new piles driven in close proximity to existing timber piles or in locations where old piles have been pulled or cut off. This method will verify if any reduction in static capacity will occur to new steel H-piles. DPT should be performed with a pile driving analyzer (PDA) to monitor driving stresses during installation, evaluate the static capacity and evaluate pile integrity, and monitor efficiency of the energy transferred to the pile by the selected hammer. 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.

(6) Vibrations:

Pile Driving and other operations can cause vibrations which may affect nearby structures, pavements and underground utilities. The contractor shall take precautions to perform the work in a prudent manner in order to minimize the possibility of producing damaging vibrations, and to reduce the effect of such vibrations.

Peak particle velocities of 0.25 inches per second as measured by seismograph are generally regarded as a vibration level uncomfortable to human perception. Peak particle velocities of 0.50 inches per second (measured at the structure in question) may induce damage to the structure.

In past experience with steel H-pile driving operations utilizing both impact and vibratory hammers, Eustis Engineers, Inc. has recorded maximum peak particle velocities of 0.70 inches per second, measured 30 feet from the source and 0.47 inches per second, measured 50 feet from the source. Structures of concern at this site include the existing pumping station which is a close as 21 feet from pile driving operations.

(7) 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*. DPT is recommended 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:

Sand fill will vary in height from 12.25 feet at the T-walls to 8.25 feet towards the pumping station. Settlement analyses were made at three points within the fill area; at the midpoint of the T-wall monolith the estimated settlement is will be 1¼ inches. Near the midpoint of the sides of the fill, adjacent to the discharge tube structures, the estimated settlement is 1¾ inches. At the center of the fill area, estimated settlement is 6½ inches at the surface. For steel H-pile foundations embedded in the underlying Pleistocene formation at tip El -80.43 NGVD (-60.00 CD), no reduction in pile capacity is necessary for the down-drag settlement effects. This estimate of settlement does not include the elastic deformation of the piles. 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 -10.00 NGVD (10.43 CD) and -90.00 NGVD (-69.57 CD). 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 sheet pile dam will be constructed across London Avenue Canal at Treasure Street. The top of the dam will be at sill El 1.57 NGVD (22.00

CD)and the dam will have four (4) butterfly gates. The purpose 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 also 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 -10.00 NGVD (10.43 CD) on both sides of the dam. The water surface was assumed at El 1.57 NGVD (22.00 CD) on the flood side and El -10.00 NGVD (10.43 CD) on the protected side. The results show a maximum bending moment of 67,283 foot-pounds occurs at El -25.46 NGVD (-5.03 CD) using a factor of safety of 1.0 applied to the soil shear strengths. Using a factor of safety of 1.5, the sheet pile wall for the temporary dam should be driven to tip El -54.52 NGVD (-34.09 CD). 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 -13.00 NGVD (7.43 CD) and -27.50 NGVD (-7.07 CD) do not exceed El -15.00 NGVD (5.53 CD). Hydrostatic pressures in the cohesionless near-shore Gulf deposits between El -40.00 NGVD (-19.57 CD) and -62.00 NGVD (-41.57 CD) are assumed not to exceed El 4.50 NGVD (24.93 CD). In order to achieve these hydrostatic pressures, it will be necessary to install a dewatering and hydrostatic 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 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 sheet pile wall using Q-case soil conditions and the Corps' "CWALSHT" program. The bottom of the cofferdam excavation was assumed at El -11.28 NGVD (9.15 CD) and the water on the flood side was assumed at El 1.57 NGVD (22.00 CD). 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 NGVD (-7.63 CD). A factor of safety of 1.5 was applied to the soil shear strengths to determine the top embedment. The analyses indicate sheet piles 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 NGVD (29.00 CD). The still water level (SWL) or flowline was furnished at El 11.90 NGVD (32.33 CD). The flowline plus 2 feet of freeboard will result in El 13.90 NGVD (34.33 CD). The top of wall will be constructed to El 14.40 NGVD (34.83 CD) 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 sheet pile wall should be installed to tip El -0.80 NGVD (19.63 CD). 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 sheet pile penetration for seepage cut-off beneath the T-wall and sluice gate structure. Using Harr's method, it is recommended that a 25-ft sheet pile 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 NGVD (11.25 CD), this will result in a tip at El -34.18 NGVD (-13.75 CD). 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

It is noted that the work will be performed in close proximity to the pumping station and other infrastructure. Methods of construction for work in areas of this type will be in accordance with the generally accepted practice for work in such confined areas, and shall be strictly adhered to during construction.

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

37. 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 control structure will be placed in front of the discharge area for the five existing horizontal pumps. Each pump will have an individual concrete discharge tube connecting it to the sluice gate structure. These structures will constructed of 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-wall 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 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. Concrete Investigation

(1) Concrete quantities and qualities will be in accordance with Table 2 below:

Table 2 - Concrete Quantities and Qualities

Structural Feature	Concrete Quantity (Cu. Yds.)	Compressive Strength* (28 days, psi)	Max. Size Aggregate** (Inches)	Air Content*** (Percent)
Stabilization Slab	60.00	2,000.00	1.50	4 to 7
6" Paving, Reinforced	1,350.00	3,000.00	1.50	4 to 7
Monoliths, Reinforced	4,850.00	4,000.00	1.50	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) Environmental conditions

The concrete will not be subjected to any critical environmental or functional conditions.

D. Cementitious Materials Investigation

(1) Cement

(a) Special Requirements:

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) Availability:

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 3 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 October 30, 1980. Transportation of aggregates would probably be by truck, except for Lambert Gravel which has also indicated that barging from their source is possible.

Table 3 - Approved Aggregate Sources

Aggregate Source	Nearest Town	Project to Pit Distance (Miles)	Pit Location Lat./Long.	TM 6-370 Vol/Area	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 & Company	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 4 below:

Table 4 - Approved Ready Mix Concrete Sources

Ready-Mix Concrete Sources	Distance to Project (Miles)	Plant Capacity (Cu.Yd/Hr.)	Plant Type	Number of Truck Mixers	Cooling Method
La. Industries (Plant No.4) (Euphrosine St.)	5	100	Semi	23	Ice
LaFarge (Airline Hwy.)	20	180	Auto	52	Ice or Chilled Water
Carlo Ditta	10	120	Auto	36	Ice
Peter Judlin (Old Gentily 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° F, while for other elements, the maximum will be 90° F.

ENVIRONMENTAL

38. General

The London Avenue Outfall Canal is a man-made canal approximately 4.0 miles in length, with an average bottom width of 100 feet and average top width of 160 feet. Drainage Pumping Station No. 3 lies at the head of the canal near N. Broad Avenue. The canal is paralleled by combined earthen levee/floodwalls or floodwalls alone from Drainage Pumping Station No. 3 to Leon C. Simon Boulevard on the east canal bank and to Robert E. Lee Boulevard on the west canal bank.

From these two boulevards north, to Lakeshore Drive the canal is contained by earthen levees on both banks.

39. 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 are generally poor due to the poorly maintained areas around the pumping station and the appearance of the existing floodwalls.

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.

40. 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 these disruptions will be limited again to daylight hours. No displacement of residences will be necessary.

41. 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 January 17, 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. An Initial Site Assessment for Hazardous, Toxic and Radioactive Waste (HTRW) was prepared on August 4, 1993. The conclusion of this HTRW assessment was that the potential for encountering hazardous or toxic waste in the construction corridor was unlikely.

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 October 7, 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 October 27, 1988. This completes the environmental compliance for construction of this feature.

ESTIMATE OF COST

42. General

The estimated first cost for construction of the fronting protection presented herein at Pumping Station No. 3 is $6,399,034.00, based on current unit prices. Engineering design and construction management fees are estimated to be $931,478.00 and ,$1,080,767.00 respectively. Tables 5, 6, 7, 8 and 9 below present the itemized first cost of the various components for the fronting protection at Drainage Pumping Station No. 3.

Table 5 - Relocations: Estimated Construction Cost
Fronting Protection - Drainage Pumping Station No. 3

Item No.	Item Description	Unit	Quantity	Unit Price	Amount
	RELOCATIONS:				
1	S&WB Electric Manhole	Each	2.0	$6,000.00	$12,000.00
2	Relocation of S&WB Electric Feeders	Ln.Ft.	500.0	$80.00	$40,000.00
3	Permanent Relocation of 48"ϕ Sewer Force Main	Ln.Ft.	355.0	$350.00	$124,250.00
				SUBTOTAL - Relocations:	$176,250.00
				Contingencies(10%):	$17,625.00
				TOTAL - Relocations:	**$193,875.00**

Table 6 - Floodwalls and Levees: Estimated Construction Cost
Fronting Protection - Drainage Pumping Station No. 3

Item No.	Item Description	Unit	Quantity	Unit Price	Amount
	<u>FLOODWALLS AND LEVEES:</u>				
1	Mobilization	Lump	Lump Sum	$250,000.00	$250,000.00
2	Backfill	Cu.Yd.	4,000.0	$15.00	$60,000.00
3	Temporary Sheet Pile Dam	Lump	Lump Sum	$500,000.00	$500,000.00
4	Removal of Exist. Concrete Lining	Lump	Lump Sum	$160,000.00	$160,000.00
5	Temporary Steel Sheeting	Sq.Ft.	12,000.0	$10.00	$120,000.00
6	Excavation	Cu.Yd.	1,000.0	$6.00	$6,000.00
7	Pile Load Test	Each	2.0	$20,000.00	$40,000.00
8	6" Thk. Reinforced Concrete Paving	Sq.Yd.	1,350.0	$32.00	$43,200.00
9	Sheet Pile Cut-Off Wall	Sq.Ft.	4,796.0	$15.00	$71,940.00
10	Foundation Piling (HP 14x73)	Ln.Ft.	23,096.0	$30.00	$692,880.00
11	Reinforced Conc. (Base Slabs)	Cu.Yd.	1,620.2	$455.00	$737,191.00
12	Reinforced Conc. (Weir)	Cu.Yd.	44.5	$430.00	$19,135.00
13	Reinforced Conc. (Walls)	Cu.Yd.	1,602.5	$480.00	$769,200.00
14	Reinforced Conc. (Roofs)	Cu.Yd.	1,560.8	$505.00	$788,204.00
15	Structural Steel	Lbs.	13,456.0	$1.25	$16,820.00
16	Dewatering System	Lump	Lump Sum	$60,000.00	$60,000.00
17	Roadway Work	Lump	Lump Sum	$80,000.00	$80,000.00
18	Temporary Detours and Barricades	Lump	Lump Sum	$5,000.00	$5,000.00

Item No.	Item Description	Unit	Quantity	Unit Price	Amount
19	Aluminum Handrail and Posts	Ln.Ft.	550.0	$25.00	$13,750.00
20	Steel Bar Grating	Sq.Ft.	940.0	$20.00	$18,800.00
21	Galvanizing Charge	Lbs.	25,126.0	$0.30	$7,537.80
22	48"ϕ Flush Pipe	Ln.Ft.	120.0	$100.00	$12,000.00
23	2 - Steel Pipes (11'-9" ID)	Lbs.	70,219.0	$1.00	$70,219.00
24	3 - Reducers (13'-4" x 11'-9")	Lbs.	85,654.0	$1.80	$154,177.20
25	Bellows or Dresser Couplings	Each	10.0	$2,000.00	$20,000.00
26	Aluminum Manhole w/Ladder	Each	5.0	$3,000.00	$15,000.00
27	Sluice Gates (81"x96")	Each	4.0	$60,000.00	$240,000.00
28	Sluice Gates (108"x96")	Each	6.0	$85,000.00	$510,000.00
29	Vacuum Pump Upgrade	Each	2.0	$80,000.00	$160,000.00
				SUBTOTAL - Floodwalls & Levees:	$5,641,054.00
				Contingency (10%):	$564,105.00
				TOTAL - Floodwalls & Levees :	**$6,205,159.00**

Table 7 - Engineering and Design
Fronting Protection - Drainage Pumping Station No. 3

Engineering Fees - Design Memorandum:	$305,690.00
Engineering Fees - Plans & Specifications:	$464,126.00
Engineering Fees - Construction Engineering:	$76,982.00
SUBTOTAL - Engineering and Design:	$846,798.00
Contingency (10%):	$84,680.00
TOTAL - Engineering and Design:	**$931,478.00**

Table 8 - Construction Management and Testing
Fronting Protection - Drainage Pumping Station No. 3

Construction Management and Design Fees:	$939,797.00
Contingency (15%):	$140,970.00
TOTAL - Construction Management:	**$1,080,767.00**

Table 9 - Opinion of Probable Project Cost
Fronting Protection - Drainage Pumping Station No. 3

Total - Relocation Costs:	$193,875.00
Total - Floodwalls & Levees Costs:	$6,205,159.00
Total - Engineering and Design:	$931,478.00
Total - Construction Management and Testing:	$1,080,767.00
SUBTOTAL - Probable Project Cost:	$8,411,279.00
Contingency (10%):	$841,128.00
TOTAL - Probable Project Cost:	**$9,252,407.00**

43. Comparison of Estimates

The current project cost estimate (LMV Form 17/PB-2A) $4,473,000.00 effective October 1, 1995 is for the relocations and levee floodwall features. The current estimate of $6,399,034.00 for these features represents an increase of $1,926,034.00 when compared to the LMV Form 17/PB-2A estimate. This increase in cost is primarily due to refinement of the design from a survey scope to a DM scope.

44. Schedule for Design and Construction

The sequence for design and construction is shown in Table 10 below:

Table 10 - Schedule for Design and Construction

	Design		Construction		
Activity	Start	Complete	Advertise	Award	Complete
Plans & Specs.	January '95	December '95	January '96	March '96	March '98

45. 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 DM 19A is shown in Table 11 below:

Table 11 - Federal and Non-Federal Cost Breakdown
(March 1996 Price Levels)

Item	Federal Costs	Non-Federal Costs	Total Costs
Relocations and Fronting Protection	$6,476,684.90	$2,775,722.10	$9,252,407.00

46. Non-Project Related Estimated Costs

The Sewerage and Water Board of New Orleans has requested that various non-project related improvements be performed at the station and site while a contractor is on the site. The breakdown of these items and their estimated costs are shown in Table 12 below:

Table 12 - Non-Project Related Estimated Costs
(Items S1 - S4)

Item No.	Item Description	Unit	Quantity	Unit Price	Amount
S-1	Concrete Deck Over Suction Basin	Lump	Lump Sum	$50,000.00	$50,000.00
S-2	Roll-Up Shutters	Lump	Lump Sum	$10,000.00	$10,000.00
S-3	Forced Air Ventilation	Lump	Lump Sum	$15,000.00	$15,000.00
S-4	Modifications to Marigny Ave. Canal	Lump	Lump Sum	$50,000.00	$50,000.00
				SUBTOTAL - Non-Project Related Costs:	$125,000.00
				Contingency (10%):	$12,500.00
				TOTAL - Non-Project Related Costs:	**$137,500.00**

OPERATIONS AND MAINTENANCE

47. General

All operations and maintenance (O&M) costs for this project will be the responsibility of the Sewerage and Water Board of New Orleans. The estimated O&M costs are shown in Table 13 below:

Table 13 - Operations and Maintenance

Maintenance Item	Annual Cost*
Sluice Gate Maintenance	$4,600.00
Gate Monolith Maintenance	$1,500.00
I-Wall/T-Wall Maintenance	$2,200.00
SUBTOTAL - O&M:	$8,300.00
Contingency:	$1,245.00
TOTAL - O&M:	**$9,545.00**

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

48. 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 10, funds will be required by fiscal year as shown in Table 14 below:

Table 14 - Total Required Funding
(Federal and Non-Federal Sources)

Fiscal Year	Funds Required
1995	$795,707.00
1996	$1,110,289.00
1197	$6,199,113.00
1998	$1,147,298.00
Total Required Funds:	**$9,252,407.00**

49. 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 of sufficient detail to proceed to plans and specifications. Approval of this supplemental design memorandum is recommended.

LIST OF ABBREVIATIONS

COMMON ABBREVIATIONS USED IN THIS REPORT

Abbreviation	Description
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
AWS	American Welding Society
CD	Cairo Datum
cfs	Cubic Feet/Second
DM 19A	Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 19A - General Design, London Avenue Outfall Canal, (January 1989)
DPS#3	Drainage Pumping Station No. 3
DPT	Dynamic Pile Testing
EA	Environmental Assessment
EIS	Environmental Impact Statement
El	Elevation
EM	Engineering Manual
EPA	Environmental Protection Agency
FONSI	Finding of No Significant Impact
HPS	High Pressure Sodium Vapor
IEEE	Institute of Electrical and Electronic Engineers
NEC	National Electrical Code
NEMA	National Electrical Manufacturers Association
NGVD	National Geodetic Vertical Datum
NOPSI	New Orleans Public Service, Inc.
OLD	Board of Commissioners of the Orleans Levee District
PDA	Pile Driving Analyzer
PSF	Pounds/Square Foot
psi	Pounds/Square Inch
SPH	Standard Project Hurricane
S&WB	Sewerage and Water Board of New Orleans
SWL	Still Water Level
USACE	Department of the Army, New Orleans District, Corps of Engineers
WES	Water Experiment Station

APPENDIX A
Soil Boring Logs & Laboratory Analysis

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

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	Shear Tests Ø	Shear Tests C	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits 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				
20	0.30		Very loose gray clayey silt w/clay layers	ML	8	18-19	43	77	110	OB	–	210				
		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			[illegible] sandy clay w/clayey sand layers & shell fragments	CL	14	43-44	33	89	119	OB	–	340				
			w/shell fragments		15	48-49										
			[illegible] 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 (SHEET 2 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		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 grey 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					Stiff greenish-grey & tan silty clay	CL													
	2.20						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 & grey 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.

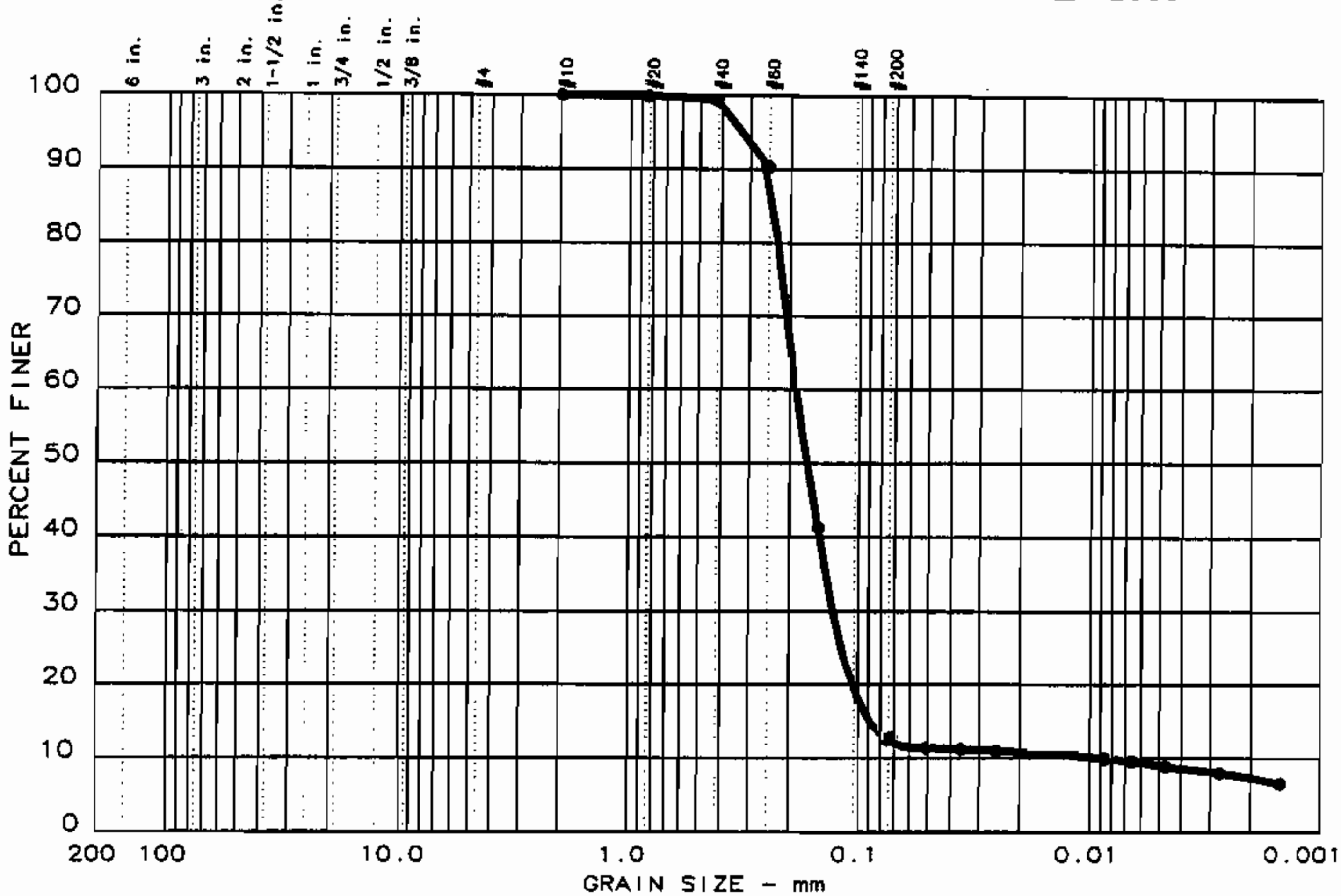
LOG OF BORING AND TEST RESULTS

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

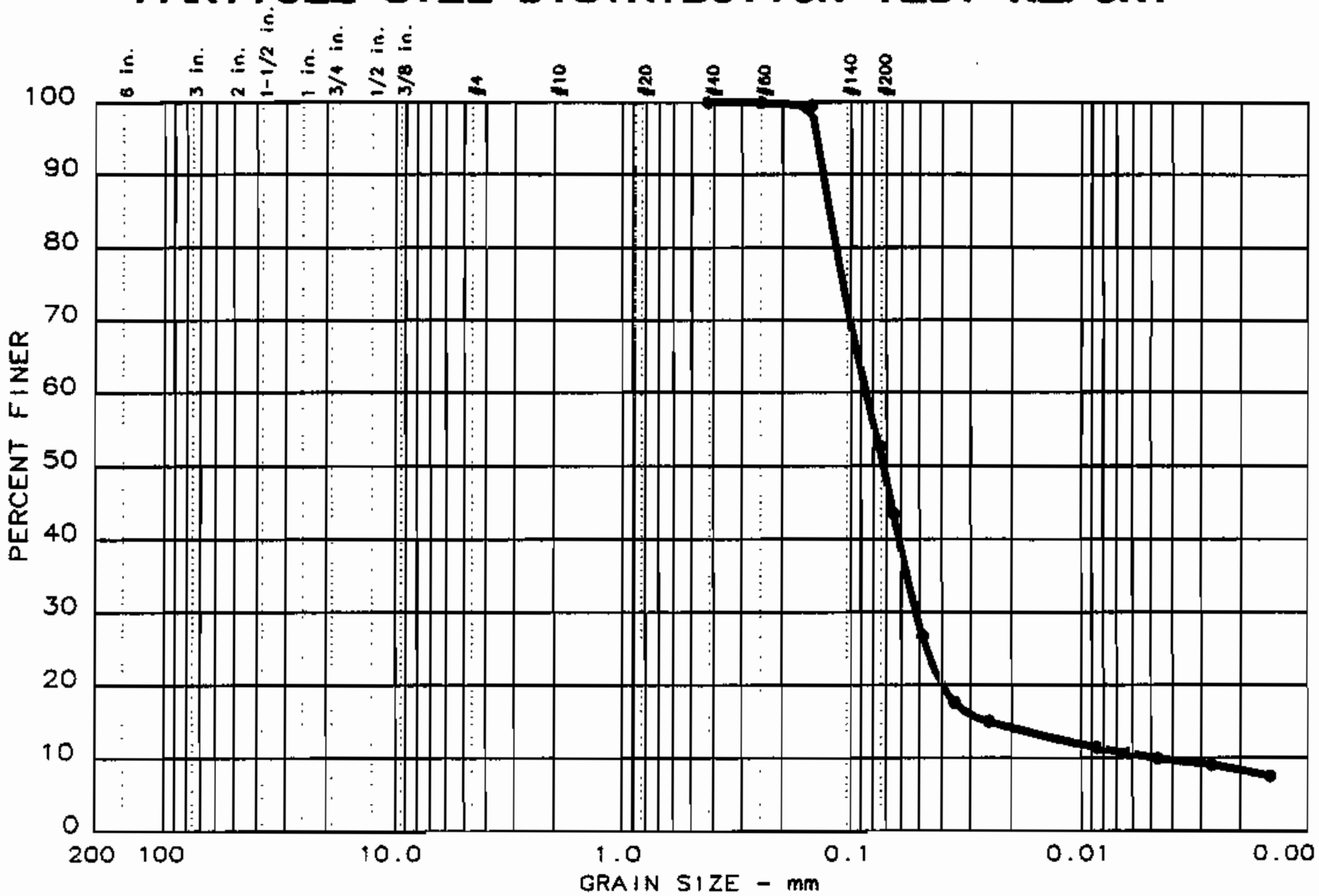
Remarks:
Sample depth 51'-52'

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'

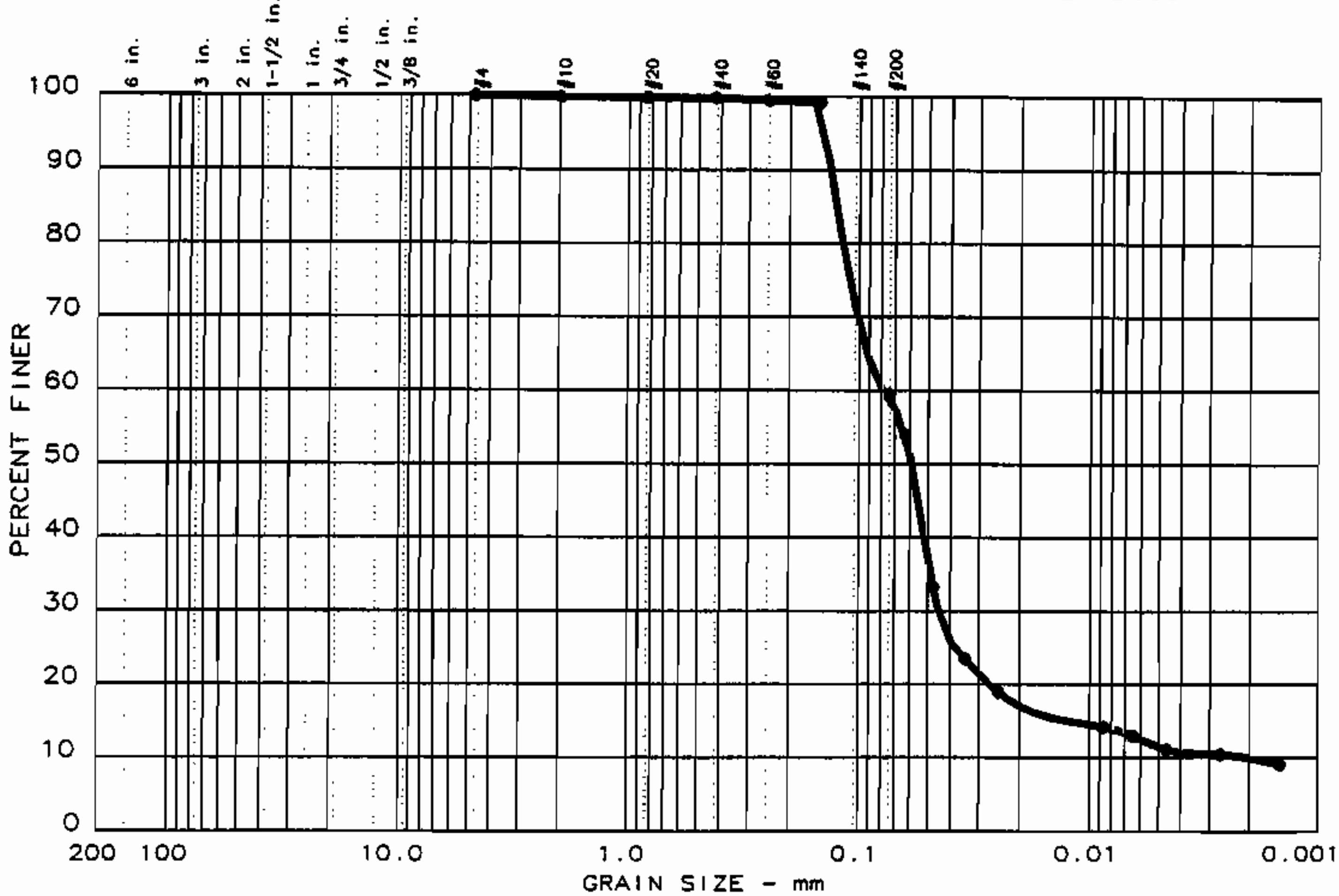
Eustis Engineering Company, Inc.

Project No.: 13065

Project: London Avenue Canal - Pump St tion #3

Date: 8-15-94 Data Sheet [illegible]

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

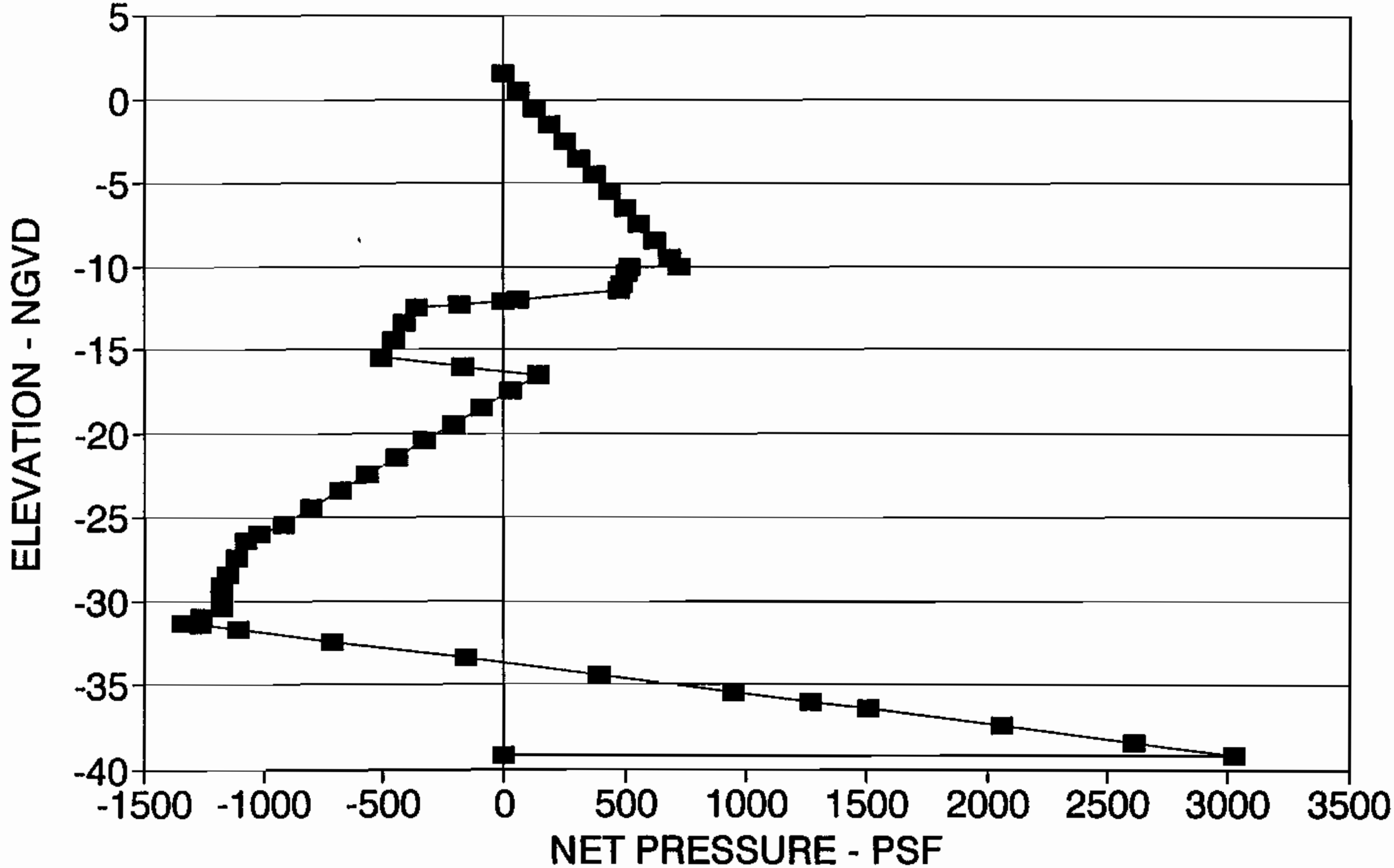
Date: 8-15-94

Data Sheet [illegible]

APPENDIX B
Temporary Dam Analysis/Cofferdam Analysis

LONDON AVENUE OUTFALL CANAL

TEMPORARY DAM ACROSS CANAL



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

DATE: 17-JUL-1995 TIME: 10.52.45

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¤ INPUT DATA ¤
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I.--HEADING:
'LONDON AVENUE CANAL JOB 13065
'TEMPORARY DAM ACROSS CANAL PZ-27

II.--CONTROL
CANTILEVER WALL ANALYSIS
SAME FACTOR OF SAFETY APPLIED TO ACTIVE AND PASSIVE PRESSURES.

III.--WALL DATA
ELEVATION AT TOP OF WALL = 1.57 (FT)
ELEVATION AT BOTTOM OF WALL = -39.17 (FT)
WALL MODULUS OF ELASTICITY = 2.90E+07 (PSI)
WALL MOMENT OF INERTIA = 184.20 (IN**4/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

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		
110.00	110.00	.00	500.0	.00	.0	-16.00	.00		
120.00	120.00	25.00	.0	.00	.0	-26.00	.00		
101.00	101.00	.00	475.0	.00	.0	-31.00	.00		
101.00	101.00	.00	525.0	.00	.0	-36.00	.00		
101.00	101.00	.00	575.0	.00	.0	-41.00	.00		

120.00	120.00	15.00	300.0	.00	.0	-60.00	.00		
110.00	110.00	.00	750.0	.00	.0	-65.00	.00		
119.00	119.00	.00	1650.0	.00	.0				

V.B.-- LEFTSIDE LAYER DATA

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		
110.00	110.00	.00	500.0	.00	.0	-16.00	.00		
120.00	120.00	25.00	.0	.00	.0	-26.00	.00		
101.00	101.00	.00	475.0	.00	.0	-31.00	.00		
101.00	101.00	.00	525.0	.00	.0	-36.00	.00		
101.00	101.00	.00	575.0	.00	.0	-41.00	.00		
120.00	120.00	15.00	300.0	.00	.0	-60.00	.00		
110.00	110.00	.00	750.0	.00	.0	-65.00	.00		
119.00	119.00	.00	1650.0	.00	.0				

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: 17-JUL-1995 TIME: 10.52.52

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

'LONDON AVENUE CANAL JOB 13065
'TEMPORARY DAM ACROSS CANAL PZ-27

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.

```
FACTOR OF SAFETY            :         1.00

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

MAXIMUM DEFLECTION (IN)     :   1.0246E+01
      AT ELEVATION (FT)     :         1.57
```

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

DATE: 17-JUL-1995 TIME: 10.52.52

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

'LONDON AVENUE CANAL JOB 13065
'TEMPORARY DAM ACROSS CANAL PZ-27

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	DEFLECTION (IN)	NET PRESSURE (PSF)
1.57	0.	0.	1.0246E+01	.00
.57	10.	31.	9.8296E+00	62.50
-.43	83.	125.	9.4132E+00	125.00
-1.43	281.	281.	8.9967E+00	187.50
-2.43	667.	500.	8.5803E+00	250.00
-3.43	1302.	781.	8.1642E+00	312.50
-4.43	2250.	1125.	7.7485E+00	375.00
-5.43	3573.	1531.	7.3335E+00	437.50
-6.43	5333.	2000.	6.9197E+00	500.00

-7.43	7594.	2531.	6.5077E+00	562.50
-8.43	10417.	3125.	6.0981E+00	625.00
-9.43	13865.	3781.	5.6919E+00	687.50
-10.00	16134.	4183.	5.4623E+00	723.13
-10.00	16134.	4183.	5.4623E+00	523.13
-10.43	17980.	4405.	5.2902E+00	509.15
-11.00	20573.	4690.	5.0637E+00	490.63
-11.43	22635.	4898.	4.8943E+00	476.65
-12.00	25481.	5051.	4.6718E+00	58.13
-12.06	25782.	5052.	4.6488E+00	.00
-12.24	26717.	5035.	4.5771E+00	-181.15
-12.43	27645.	4985.	4.5058E+00	-362.30
-13.43	32442.	4599.	4.1261E+00	-409.80
-14.43	36828.	4166.	3.7570E+00	-457.30
-15.43	40757.	3684.	3.3998E+00	-504.80
-16.00	42793.	3493.	3.2020E+00	-166.78
-16.43	44290.	3489.	3.0558E+00	147.44
-17.43	47832.	3577.	2.7260E+00	29.10
-18.43	51404.	3547.	2.4118E+00	-89.24
-19.43	54887.	3399.	2.1142E+00	-207.58
-20.43	58162.	3132.	1.8343E+00	-325.92
-21.43	61112.	2747.	1.5733E+00	-444.25
-22.43	63617.	2243.	1.3320E+00	-562.59
-23.43	65559.	1622.	1.1112E+00	-680.93
-24.43	66821.	882.	9.1166E-01	-799.27
-25.43	67283.	23.	7.3370E-01	-917.61
-26.00	67141.	-529.	6.4199E-01	-1020.97
-26.43	66818.	-980.	5.7748E-01	-1073.43
-27.43	65295.	-2072.	4.4285E-01	-1111.93
-28.43	62660.	-3203.	3.2931E-01	-1150.43
-29.12	60186.	-4003.	2.6311E-01	-1176.88
-29.43	58875.	-4371.	2.3601E-01	-1176.88
-30.43	53916.	-5548.	1.6172E-01	-1176.88
-31.00	50557.	-6244.	1.2729E-01	-1263.13
-31.30	48602.	-6638.	1.1113E-01	-1335.73
-31.43	47753.	-6802.	1.0485E-01	-1265.76
-31.71	45770.	-7140.	9.1586E-02	-1108.26
-32.43	40410.	-7791.	6.3382E-02	-711.76
-33.43	32355.	-8226.	3.4971E-02	-157.76
-34.43	24143.	-8107.	1.7022E-02	396.24
-35.43	16326.	-7434.	6.8935E-03	950.24
-36.00	12261.	-6802.	3.6456E-03	1266.02
-36.43	9460.	-6206.	2.0721E-03	1504.24
-37.43	4098.	-4425.	3.5145E-04	2058.23
-38.43	795.	-2090.	1.2074E-05	2612.23
-39.17	1.	0.	0.0000E+00	3023.13

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 AVENUE CANAL   JOB 13065
1010 'TEMPORARY DAM ACROSS CANAL   PZ-27
1020 CONTROL  C  A
1030 WALL        1.57    -39.17   2.900E+07    184.20
1040 SURFACE RIGHTSIDE    1
1050         .00     -10.00
1060 SURFACE  LEFTSIDE    1
1070         .00     -10.00
1080 SOIL RIGHTSIDE  STRENGTH     9      .00
1090   95.00   95.00    .00  100.00     .00      .00   -12.00     .00
1100  110.00  110.00    .00  500.00     .00      .00   -16.00     .00
1110  120.00  120.00 25.00      .00     .00      .00   -26.00     .00
1120  101.00  101.00    .00  475.00     .00      .00   -31.00     .00
1130  101.00  101.00    .00  525.00     .00      .00   -36.00     .00
1140  101.00  101.00    .00  575.00     .00      .00   -41.00     .00
1150  120.00  120.00 15.00  300.00     .00      .00   -60.00     .00
1160  110.00  110.00    .00  750.00     .00      .00   -65.00     .00
1170  119.00  119.00    .00 1650.00     .00      .00
1180 SOIL  LEFTSIDE  STRENGTH     9      .00
1190   95.00   95.00    .00  100.00     .00      .00   -12.00     .00
1200  110.00  110.00    .00  500.00     .00      .00   -16.00     .00
1210  120.00  120.00 25.00      .00     .00      .00   -26.00     .00
1220  101.00  101.00    .00  475.00     .00      .00   -31.00     .00
1230  101.00  101.00    .00  525.00     .00      .00   -36.00     .00
1240  101.00  101.00    .00  575.00     .00      .00   -41.00     .00
1250  120.00  120.00 15.00  300.00     .00      .00   -60.00     .00
1260  110.00  110.00    .00  750.00     .00      .00   -65.00     .00
1270  119.00  119.00    .00 1650.00     .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: 20-JUL-1995 TIME: 9.59.27

INPUT DATA

I.--HEADING:
'LONDON AVENUE CANAL JOB 13065
'TEMPORARY DAM ACROSS CANAL PZ-38

II.--CONTROL
CANTILEVER WALL ANALYSIS
SAME FACTOR OF SAFETY APPLIED TO ACTIVE AND PASSIVE PRESSURES.

III.--WALL DATA
ELEVATION AT TOP OF WALL = 1.57 (FT)
ELEVATION AT BOTTOM OF WALL = -39.17 (FT)
WALL MODULUS OF ELASTICITY = 2.90E+07 (PSI)
WALL MOMENT OF INERTIA = 380.80 (IN**4/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

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		
110.00	110.00	.00	500.0	.00	.0	-16.00	.00		
120.00	120.00	25.00	.0	.00	.0	-26.00	.00		
101.00	101.00	.00	475.0	.00	.0	-31.00	.00		
101.00	101.00	.00	525.0	.00	.0	-36.00	.00		
101.00	101.00	.00	575.0	.00	.0	-41.00	.00		

120.00	120.00	15.00	300.0	.00	.0	-60.00	.00		
110.00	110.00	.00	750.0	.00	.0	-65.00	.00		
119.00	119.00	.00	1650.0	.00	.0				

V.B.-- LEFTSIDE LAYER DATA

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		
110.00	110.00	.00	500.0	.00	.0	-16.00	.00		
120.00	120.00	25.00	.0	.00	.0	-26.00	.00		
101.00	101.00	.00	475.0	.00	.0	-31.00	.00		
101.00	101.00	.00	525.0	.00	.0	-36.00	.00		
101.00	101.00	.00	575.0	.00	.0	-41.00	.00		
120.00	120.00	15.00	300.0	.00	.0	-60.00	.00		
110.00	110.00	.00	750.0	.00	.0	-65.00	.00		
119.00	119.00	.00	1650.0	.00	.0				

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: 20-JUL-1995 TIME: 9.59.35

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

'LONDON AVENUE CANAL JOB 13065
'TEMPORARY DAM ACROSS CANAL PZ-38

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.

```
FACTOR OF SAFETY            :         1.00

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

MAXIMUM DEFLECTION (IN)    :   4.9562E+00
      AT ELEVATION (FT)    :         1.57
```

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

DATE: 20-JUL-1995 TIME: 9.59.35

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

'LONDON AVENUE CANAL JOB 13065
'TEMPORARY DAM ACROSS CANAL PZ-38

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	DEFLECTION (IN)	NET PRESSURE (PSF)
1.57	0.	0.	4.9562E+00	.00
.57	10.	31.	4.7548E+00	62.50
-.43	83.	125.	4.5533E+00	125.00
-1.43	281.	281.	4.3519E+00	187.50
-2.43	667.	500.	4.1505E+00	250.00
-3.43	1302.	781.	3.9492E+00	312.50
-4.43	2250.	1125.	3.7481E+00	375.00
-5.43	3573.	1531.	3.5474E+00	437.50
-6.43	5333.	2000.	3.3472E+00	500.00

-7.43	7594.	2531.	3.1479E+00	562.50
-8.43	10417.	3125.	2.9497E+00	625.00
-9.43	13865.	3781.	2.7533E+00	687.50
-10.00	16134.	4183.	2.6422E+00	723.13
-10.00	16134.	4183.	2.6422E+00	523.13
-10.43	17980.	4405.	2.5589E+00	509.15
-11.00	20573.	4690.	2.4494E+00	490.63
-11.43	22635.	4898.	2.3675E+00	476.65
-12.00	25481.	5051.	2.2598E+00	58.13
-12.06	25782.	5052.	2.2487E+00	.00
-12.24	26717.	5035.	2.2140E+00	-181.15
-12.43	27645.	4985.	2.1795E+00	-362.30
-13.43	32442.	4599.	1.9959E+00	-409.80
-14.43	36828.	4166.	1.8173E+00	-457.30
-15.43	40757.	3684.	1.6446E+00	-504.80
-16.00	42793.	3493.	1.5489E+00	-166.78
-16.43	44290.	3489.	1.4781E+00	147.44
-17.43	47832.	3577.	1.3186E+00	29.10
-18.43	51404.	3547.	1.1666E+00	-89.24
-19.43	54887.	3399.	1.0227E+00	-207.58
-20.43	58162.	3132.	8.8729E-01	-325.92
-21.43	61112.	2747.	7.6101E-01	-444.25
-22.43	63617.	2243.	6.4429E-01	-562.59
-23.43	65559.	1622.	5.3751E-01	-680.93
-24.43	66821.	882.	4.4099E-01	-799.27
-25.43	67283.	23.	3.5490E-01	-917.61
-26.00	67141.	-529.	3.1054E-01	-1020.97
-26.43	66818.	-980.	2.7934E-01	-1073.43
-27.43	65295.	-2072.	2.1422E-01	-1111.93
-28.43	62660.	-3203.	1.5929E-01	-1150.43
-29.12	60186.	-4003.	1.2727E-01	-1176.88
-29.43	58875.	-4371.	1.1416E-01	-1176.88
-30.43	53916.	-5548.	7.8229E-02	-1176.88
-31.00	50557.	-6244.	6.1572E-02	-1263.13
-31.30	48602.	-6638.	5.3757E-02	-1335.73
-31.43	47753.	-6802.	5.0716E-02	-1265.76
-31.71	45770.	-7140.	4.4302E-02	-1108.26
-32.43	40410.	-7791.	3.0659E-02	-711.76
-33.43	32355.	-8226.	1.6916E-02	-157.76
-34.43	24143.	-8107.	8.2338E-03	396.24
-35.43	16326.	-7434.	3.3345E-03	950.24
-36.00	12261.	-6802.	1.7634E-03	1266.02
-36.43	9460.	-6206.	1.0023E-03	1504.24
-37.43	4098.	-4425.	1.7000E-04	2058.23
-38.43	795.	-2090.	5.8403E-06	2612.23
-39.17	1.	0.	0.0000E+00	3023.13

III.--SOIL PRESSURES

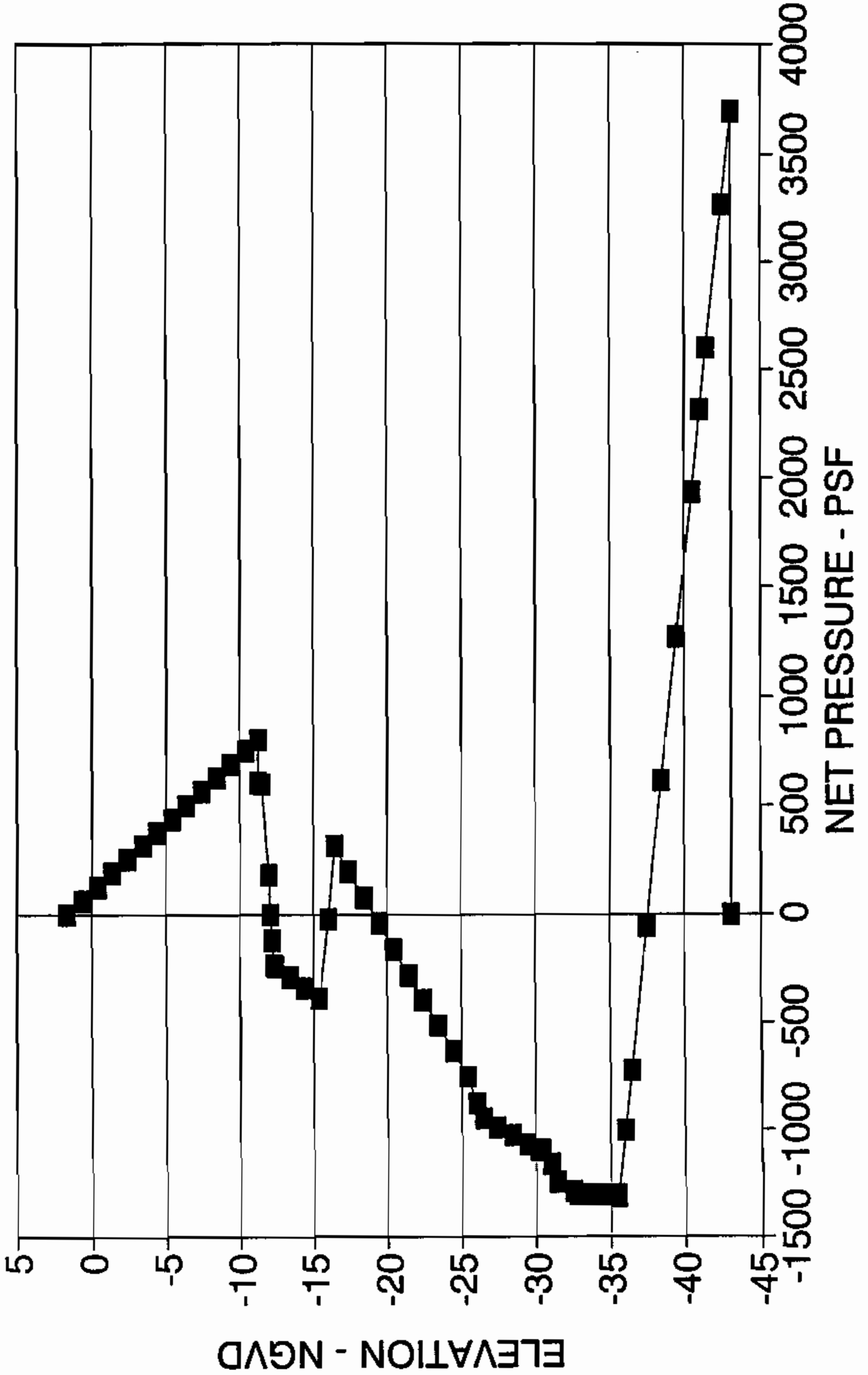
ELEVATION	< LEFTSIDE PRESSURE (PSF)>		<RIGHTSIDE PRESSURE (PSF)>	
(FT)	PASSIVE	ACTIVE	ACTIVE	PASSIVE
1.57	0.	0.	0.	0.
.57	0.	0.	0.	0.
-.43	0.	0.	0.	0.
-1.43	0.	0.	0.	0.
-2.43	0.	0.	0.	0.
-3.43	0.	0.	0.	0.
-4.43	0.	0.	0.	0.

-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 AVENUE CANAL    JOB 13065
1010 'TEMPORARY DAM ACROSS CANAL    PZ-38
1020 CONTROL  C  A
1030 WALL         1.57     -39.17   2.900E+07     380.80
1040 SURFACE RIGHTSIDE    1
1050          .00    -10.00
1060 SURFACE  LEFTSIDE    1
1070          .00    -10.00
1080 SOIL RIGHTSIDE  STRENGTH     9       .00
1090   95.00   95.00    .00  100.00      .00       .00    -12.00      .00
1100  110.00  110.00    .00  500.00      .00       .00    -16.00      .00
1110  120.00  120.00 25.00      .00      .00       .00    -26.00      .00
1120  101.00  101.00    .00  475.00      .00       .00    -31.00      .00
1130  101.00  101.00    .00  525.00      .00       .00    -36.00      .00
1140  101.00  101.00    .00  575.00      .00       .00    -41.00      .00
1150  120.00  120.00 15.00  300.00      .00       .00    -60.00      .00
1160  110.00  110.00    .00  750.00      .00       .00    -65.00      .00
1170  119.00  119.00    .00 1650.00      .00       .00
1180 SOIL  LEFTSIDE  STRENGTH     9       .00
1190   95.00   95.00    .00  100.00      .00       .00    -12.00      .00
1200  110.00  110.00    .00  500.00      .00       .00    -16.00      .00
1210  120.00  120.00 25.00      .00      .00       .00    -26.00      .00
1220  101.00  101.00    .00  475.00      .00       .00    -31.00      .00
1230  101.00  101.00    .00  525.00      .00       .00    -36.00      .00
1240  101.00  101.00    .00  575.00      .00       .00    -41.00      .00
1250  120.00  120.00 15.00  300.00      .00       .00    -60.00      .00
1260  110.00  110.00    .00  750.00      .00       .00    -65.00      .00
1270  119.00  119.00    .00 1650.00      .00       .00
1280 WATER ELEVATIONS     62.50     1.57  -10.00
1290 FINISH
```

LONDON AVENUE OUTFALL CANAL

TEMPORARY COFFERDAM FOR DISCHARGE TUBES



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

DATE: 19-JUL-1995 TIME: 10.14.19

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¤ INPUT DATA ¤
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I.--HEADING:
'LONDON AVENUE CANAL JOB 13065
'TEMPORARY COFFERDAM 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: 19-JUL-1995 TIME: 10.14.31

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

'LONDON AVENUE CANAL JOB 13065
'TEMPORARY COFFERDAM 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: 19-JUL-1995 TIME: 10.14.31

COMPLETE RESULTS FOR
CANTILEVER WALL DESIGN

I.--HEADING

'LONDON AVENUE CANAL JOB 13065
'TEMPORARY COFFERDAM 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	< 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.
-10.43	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.

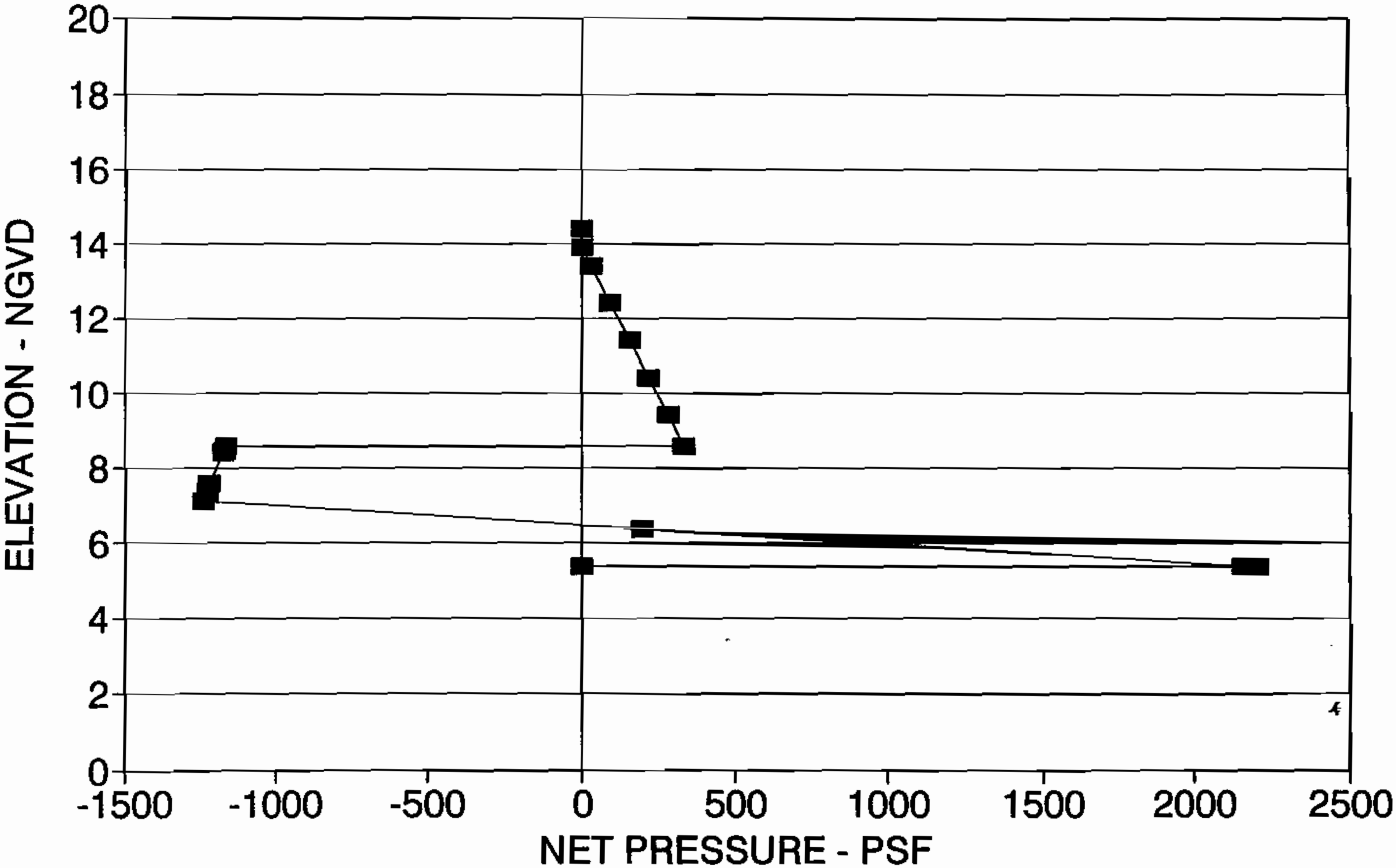
-32.43	2086.	0.	0.	2086.
-32.79	2100.	0.	0.	2100.
-33.43	2124.	24.	24.	2124.
-34.43	2163.	63.	63.	2163.
-35.43	2201.	101.	101.	2201.
-35.54	2206.	87.	106.	2206.
-36.00+	2223.	123.	123.	2223.
-36.00-	2323.	23.	23.	2323.
-36.43	2340.	40.	40.	2340.
-37.43	2378.	78.	78.	2378.
-38.43	2417.	117.	117.	2417.
-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 AVENUE CANAL    JOB 13065
1010 'TEMPORARY COFFERDAM    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
```

APPENDIX C
I-Wall Analysis/T-Wall Analysis

LONDON AVENUE OUTFALL CANAL

I-WALL



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. 6.66 Mmax = 460 ft-lbs

F.S. = 1.0 WATER EL. 13.90 TIP EL. 5.38 Mmax = 1911 ft-lbs

COMPUTED VALUE (CV) IS DEEPEST PENETRATION ABOVE.

COMPARE CV TO 3:1 PENETRATION TO HEAD RATIO.

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

SINCE CV LESS THAN 3:1 RATIO, 3:1 RATIO CONTROLS

∴ USE TIP EL. -1.42 AND PRESSURE DIAGRAM FOR Q-CASE F.S. = 1.0

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

DATE: 19-JUL-1995 TIME: 11.00.54

âêêêêêêêêêêêêêêê£
¤ INPUT DATA ¤
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I.--HEADING:
'LONDON AVENUE CANAL JOB 13065
' I-WALL Q-CASE SWL

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

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.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)	<--BOTTOM--> SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
115.00	115.00	.00	750.0	.00	.0	4.00	.00	DEF	DEF
115.00	115.00	.00	700.0	.00	.0	-6.00	.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.
115.00	115.00	.00	750.0	.00	.0	4.00	.00	DEF	DEF
115.00	115.00	.00	700.0	.00	.0	-6.00	.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 = 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: 19-JUL-1995 TIME: 11.01.05

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¤ SUMMARY OF RESULTS FOR ¤

¤ CANTILEVER WALL DESIGN ¤
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I.--HEADING

'LONDON AVENUE CANAL JOB 13065
' I-WALL 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)     :        6.66
     PENETRATION (FT)      :        1.91

MAX. BEND. MOMENT (LB-FT)  :        460.
     AT ELEVATION (FT)     :        8.14

MAX. SCALED DEFL. (LB-IN3):   8.9416E+06
     AT ELEVATION (FT)     :       14.40
```

(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: 19-JUL-1995 TIME: 11.01.05

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

'LONDON AVENUE CANAL JOB 13065
' I-WALL Q-CASE

II.--RESULTS

ELEVATION	BENDING MOMENT	SHEAR	SCALED DEFLECTION	NET PRESSURE

(FT)	(LB-FT)	(LB)	(LB-IN3)	(PSF)
14.40	0.	0.	8.9416E+06	.00
13.40	0.	0.	7.4501E+06	.00
12.40	0.	0.	5.9587E+06	.00
11.90	0.	0.	5.2130E+06	.00
11.40	1.	8.	4.4673E+06	31.25
10.40	35.	70.	2.9827E+06	93.75
9.40	163.	195.	1.5723E+06	156.25
8.57	385.	347.	6.1503E+05	208.13
8.57	385.	347.	6.1503E+05	-791.88
8.48	411.	279.	5.3776E+05	-796.34
8.40	432.	211.	4.6560E+05	-800.80
7.75	397.	-322.	9.7706E+04	-835.01
7.57	329.	-438.	5.1304E+04	-463.47
7.40	249.	-486.	2.3892E+04	-109.38
6.66	0.	0.	0.0000E+00	1427.53

(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.
8.57-	1000.	0.	0.	1000.
8.48	1010.	0.	0.	1004.
8.40	1020.	0.	0.	1009.
7.75	1094.	0.	0.	1043.
7.57	1115.	0.	0.	1053.
7.40	1135.	0.	0.	1061.
6.66	1250.	0.	0.	1114.
5.40	1365.	0.	0.	1166.

```
1000 'LONDON AVENUE CANAL   JOB 13065
1010 ' I-WALL   Q-CASE
1020 CONTROL  C  D    1.50    1.50
1030 WALL       14.40
1040 SURFACE RIGHTSIDE   1
1050        .00     8.57
1060 SURFACE  LEFTSIDE   1
1070        .00     8.57
1080 SOIL RIGHTSIDE  STRENGTH   10      .00      .00
1085  115.00  115.00   .00  750.00     .00      .00     4.00     .00   .00   .00
1090  115.00  115.00   .00  700.00     .00      .00    -6.00     .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   10      .00      .00
1185  115.00  115.00   .00  750.00     .00      .00     4.00     .00   .00   .00
1190  115.00  115.00   .00  700.00     .00      .00    -6.00     .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    11.90    -6.00
1290 FINISH
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PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 19-JUL-1995 TIME: 11.03.42

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¤ INPUT DATA ¤
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I.--HEADING:
'LONDON AVENUE CANAL JOB 13065
' I-WALL Q-CASE *SWL + 2' FB*

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 = 14.40 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

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

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.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.
115.00	115.00	.00	750.0	.00	.0	4.00	.00	DEF	DEF
115.00	115.00	.00	700.0	.00	.0	-6.00	.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.
115.00	115.00	.00	750.0	.00	.0	4.00	.00	DEF	DEF
115.00	115.00	.00	700.0	.00	.0	-6.00	.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 = 13.90 (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: 19-JUL-1995 TIME: 11.03.52

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¤ SUMMARY OF RESULTS FOR ¤

¤ CANTILEVER WALL DESIGN ¤
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I.--HEADING

'LONDON AVENUE CANAL JOB 13065
' I-WALL 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)     :        5.38
     PENETRATION (FT)      :        3.19

MAX. BEND. MOMENT (LB-FT)  :       1911.
     AT ELEVATION (FT)     :        7.82

MAX. SCALED DEFL. (LB-IN3):   6.2889E+07
     AT ELEVATION (FT)     :       14.40
```

(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: 19-JUL-1995 TIME: 11.03.52

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

'LONDON AVENUE CANAL JOB 13065
' I-WALL Q-CASE

II.--RESULTS

ELEVATION	BENDING MOMENT	SHEAR	SCALED DEFLECTION	NET PRESSURE

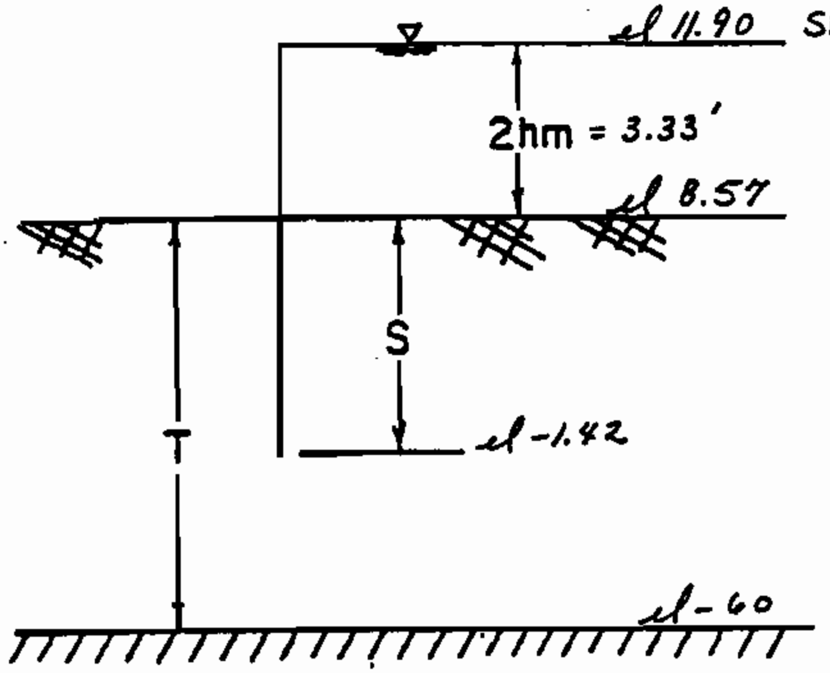
(FT)	(LB-FT)	(LB)	(LB-IN3)	(PSF)
14.40	0.	0.	6.2889E+07	.00
13.90	0.	0.	5.7787E+07	.00
13.40	1.	8.	5.2684E+07	31.25
12.40	35.	70.	4.2487E+07	93.75
11.40	163.	195.	3.2363E+07	156.25
10.40	447.	383.	2.2543E+07	218.75
9.40	949.	633.	1.3527E+07	281.25
8.57	1577.	888.	7.2676E+06	333.13
8.57	1577.	888.	7.2676E+06	-1166.88
8.48	1649.	788.	6.7189E+06	-1171.34
8.40	1711.	689.	6.1907E+06	-1175.80
7.57	1873.	-305.	2.2284E+06	-1219.38
7.40	1803.	-513.	1.6933E+06	-1228.30
7.13	1624.	-841.	1.0357E+06	-1242.23
6.40	800.	-1225.	1.4120E+05	197.60
5.40	1.	-47.	3.4938E-02	2157.60
5.38	0.	0.	0.0000E+00	2200.17

(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.
13.90	0.	0.	0.	0.
13.40	0.	0.	0.	0.
12.40	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.
8.57-	1500.	0.	0.	1500.
8.48	1510.	0.	0.	1504.
8.40	1520.	0.	0.	1509.
7.57	1615.	0.	0.	1553.
7.40	1635.	0.	0.	1561.
7.13	1665.	0.	0.	1575.
6.40	1750.	0.	0.	1614.
5.40	1865.	0.	0.	1666.
5.38	1980.	0.	0.	1719.
4.00+	2026.	0.	0.	1740.
4.00-	1926.	0.	0.	1640.

```
1000 'LONDON AVENUE CANAL   JOB 13065
1010 ' I-WALL   Q-CASE
1020 CONTROL  C  D    1.0    1.0
1030 WALL       14.40
1040 SURFACE RIGHTSIDE   1
1050       .00    8.57
1060 SURFACE  LEFTSIDE   1
1070       .00    8.57
1080 SOIL RIGHTSIDE  STRENGTH   10      .00      .00
1085  115.00  115.00   .00  750.00    .00      .00      4.00     .00   .00   .00
1090  115.00  115.00   .00  700.00    .00      .00     -6.00     .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   10      .00      .00
1185  115.00  115.00   .00  750.00    .00      .00      4.00     .00   .00   .00
1190  115.00  115.00   .00  700.00    .00      .00     -6.00     .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   13.90   -6.00
1290 FINISH
```



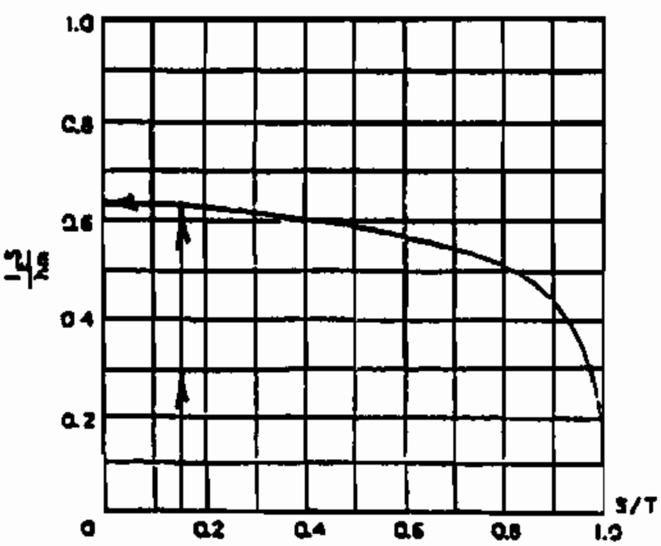
S = 9.99'   T = 68.57'   S/T = 0.15   hm = 1.67'

I_E S/hm FOR S/T = 0.62 I_E = 0.10

FOR   Icr. = γsub/γw = 58/62.4 = 0.93

F. S. = Icr/I_E = 9.3 > 4.0 OK

FOR SM & SP REC. F.S. = 4.0



PUMP STATION No. 3
I-WALL

CUTOFF WALL ANALYSIS

BY: HARR METHOD

U.S. ARMY ENGINEERING DISTRICT   NEW ORLEANS
CORPS OF ENGINEERS

Page

EUSTIS ENGINEERING COMPANY, INC.
Geotechnical Engineers
Metairie, Louisiana

Date 1-20-95
Job# 13065
By QRE
Checked By

Project PUMP STATION NO. 3
Subject SEEPAGE CUTOFF T-WALL

HARR'S METHOD

T-WALL @ STA. 1+57 TO STA. 2+07

SWL @ el 11.9 NGVD

GROUND WATER @ el -6.0 NGVD BORING 1 (1994)

HEAD = 11.9 + 6.0 = 17.9' = 2 h_m ∴ h_m = 8.95'

T = THICKNESS OF PERVIOUS STRATUM

SAY: FROM BOTTOM OF CANAL (el -9.18) to el -60 (CH SOIL)

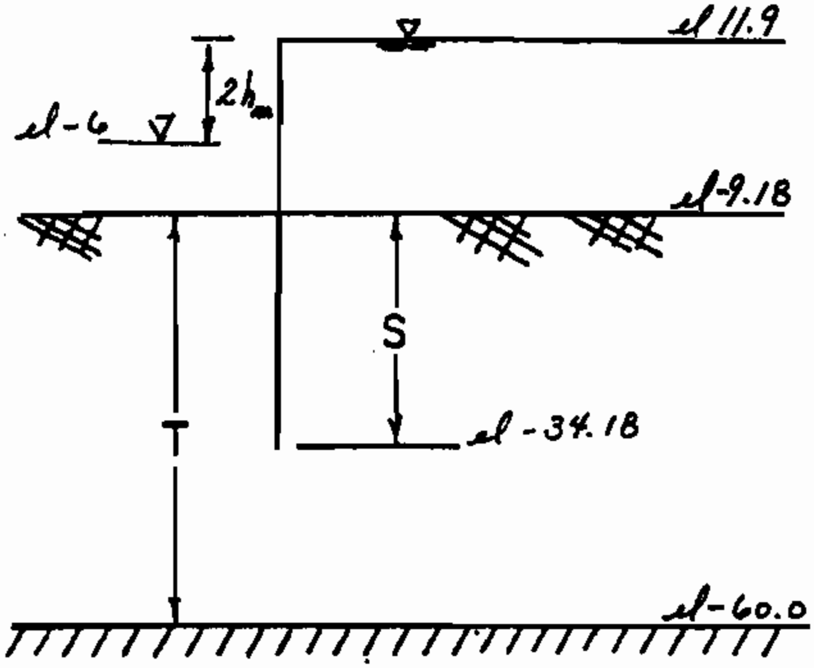
T = 60 - 9.18 = 50.82'

$I_{CR} = 58/62.5 = 0.9$

S	$I_E S/h_m$	S/T	I_E	F.S. = I_{CR}/I_E
10'	.636	.197	.57	1.63
20'	.62	.394	.277	3.35
25'	.60	.492	.215	4.33 ✓
30'	.60	.590	.179	5.20

RECOMMEND 25' SHEETPILE CUTOFF FOR T-WALLS & SLUICE GATES

TIP EL - 34.18 NGVD



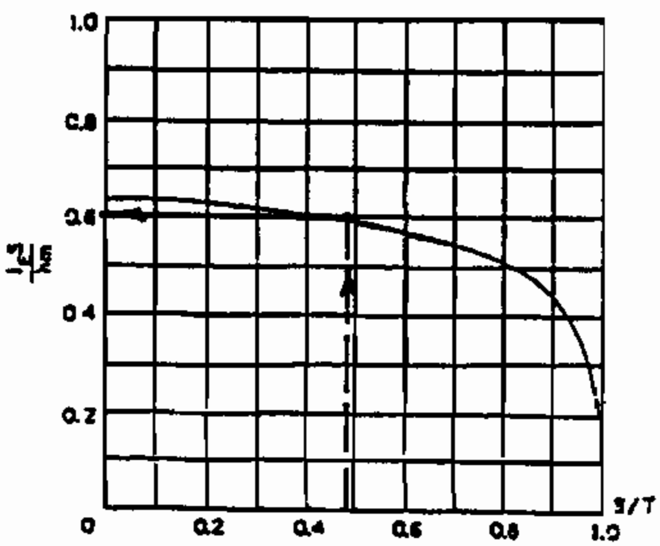
$S = 25'$ $T = 50.82'$ $S/T = 0.492$ $hm = 8.95'$

$I_E\, S/hm$ FOR $S/T = 0.60$ $I_E = 0.215$

FOR $Icr. = \gamma sub/\gamma w = 58/62.4 = 0.93$

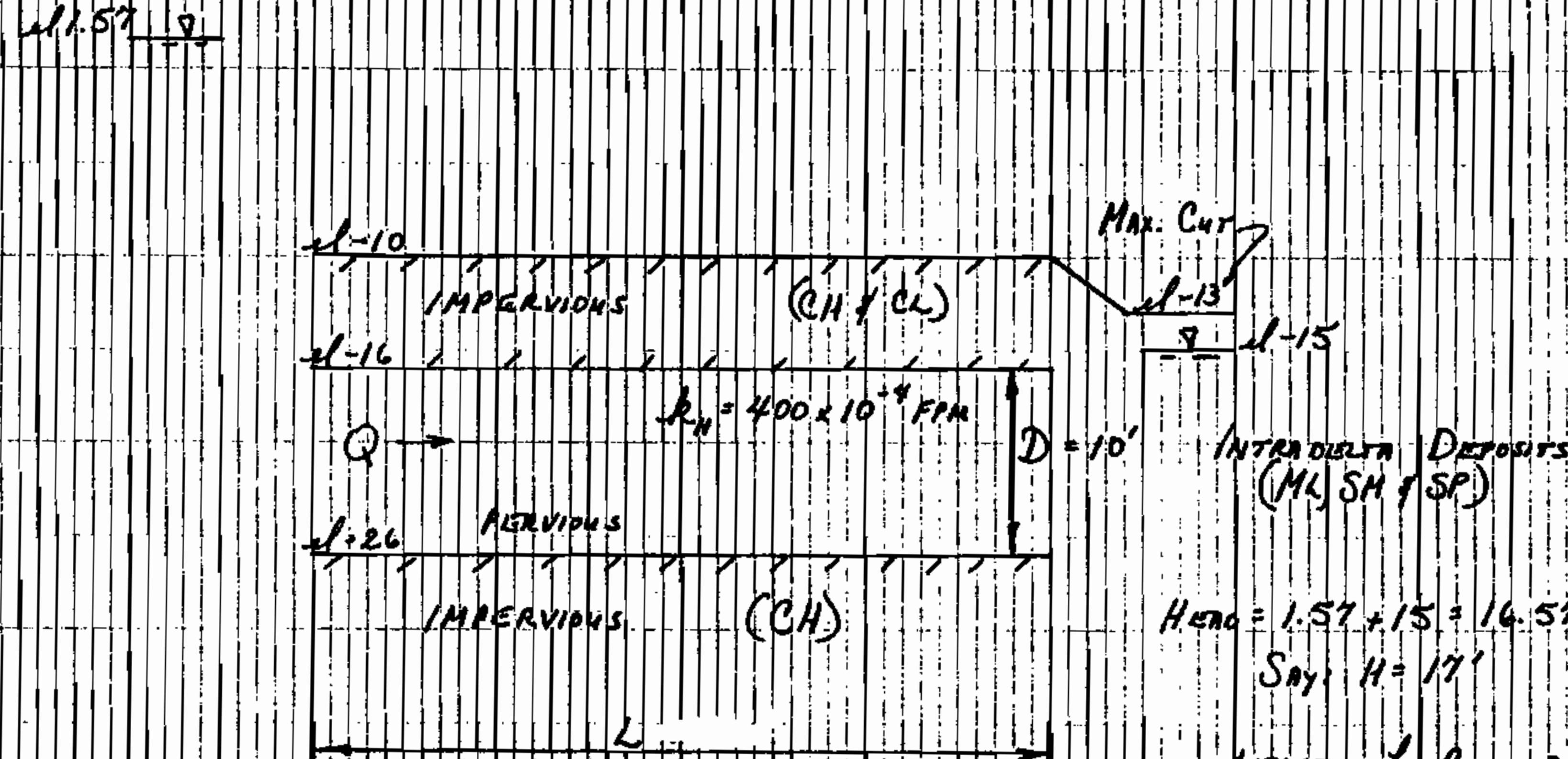
F. S. = $Icr/I_E = 4.33 > 4.0$ OK

FOR SM & SP REC. F.S. = 4.0



PUMP STATION No. 3
T-WALL
CUTOFF WALL ANALYSIS
BY: HARR METHOD
U. S. ARMY ENGINEERING DISTRICT NEW ORLEANS
CORPS OF ENGINEERS

London Avenue Outfall Canal



Head = 1.57 + 15 = 16.57'
Say: H = 17'

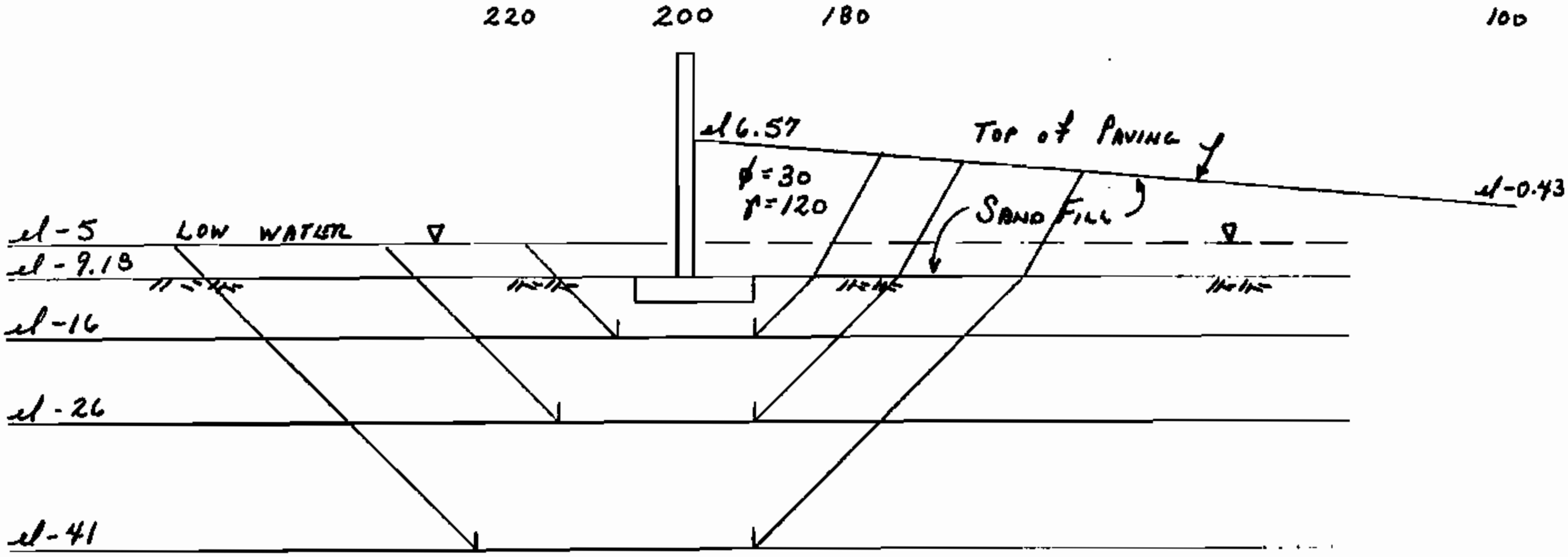
Length of Basin = 300'
L = 150' To Excavation

Rate of Flow $q = \frac{k\,D}{L} \times H$

$q = \frac{(400 \times 10^{-4}\ \text{FPM})(10')(17')}{150} = 0.045$ CFM/Foot

Width of Basin Varies from 90' to 180' Say: W = 135'

Flow $Q = q \times W = (.045)(135) = 6.12$ CFM



Failure Elevation	Factor of Safety
-16	1.04
-26	1.23
-41	1.40

*Note: F.S. = 1.3 Applied To Soil Shear Strengths

London Ave. Outfall Canal
Monolith T-1 & T-2

Job 13065 JRE 7-20-95

**** STABILITY WITH UPLIFT ****

FAILURE TOWARD CANAL DURING LOW WATER EL -5.0
T-WALL MONOLITHS T-1 AND T-2 (F.S.=1.3 APPLIED TO SHEAR STRENGTHS)
11 PROFILES
1VERTICALS
UPLIFT WITH 1 PIEZOMETRIC GRADE LINES

* * STRATUM 3 ACT. WEDGE LOC. 90191.0 EL. -16.0 PASS.WEDGE LOC. 207.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED
.0	-16.0	1800.	688.	385.	405.	385.
100.0	-16.0	1800.	688.	385.	405.	385.
200.0	-16.0	2515.	688.	385.	665.	385.
200.0	-16.0	2515.	688.	385.	665.	385.
SHEAR STRENGTHS ARE EQUAL 385.0 AT DIST. 200.1						
200.1	-16.0	1230.	688.	385.	198.	198.
200.2	-16.0	1011.	688.	385.	118.	118.
400.0	-16.0	1011.	688.	385.	118.	118.

ASSUMED CRIT. PASSIVE LOC. 207.0 EL. -16.0 DP 4886. RP 5251.

ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS
191.0	-16.0	24926.	11262.	0.	4317.	1.04

CRIT. ACTIVE LOC 191.0 EL -16.0 DA 24926. RA 11262.

DIS.	EL.	DP	RP	DB	RB	FS
207.0	-16.0	4886.	5251.	0.	4317.	1.04

* * STRATUM 4 ACT. WEDGE LOC. 90191.0 EL. -26.0 PASS.WEDGE LOC. 214.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED
.0	-26.0	3000.	1313.	614.	385.	385.

100.0	-26.0	3000.	1313.	614.	385.	385.
200.0	-26.0	3715.	1313.	874.	385.	385.
200.0	-26.0	3715.	1313.	874.	385.	385.
200.1	-26.0	2430.	1313.	407.	385.	385.

SHEAR STRENGTHS ARE EQUAL 385.0 AT DIST. 200.1

200.2	-26.0	2211.	1313.	327.	385.	327.
400.0	-26.0	2211.	1313.	327.	385.	327.

ASSUMED CRIT. PASSIVE LOC. 214.0 EL. -26.0 DP 21000. RP 11607.

ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS
191.0	-26.0	54185.	21207.	0.	8055.	1.23

CRIT. ACTIVE LOC 191.0 EL -26.0 DA 54185. RA 21207.

DIS.	EL.	DP	RP	DB	RB	FS
214.0	-26.0	21000.	11607.	0.	8055.	1.23

* * STRATUM 5 ACT. WEDGE LOC. 90191.0 EL. -31.0 PASS.WEDGE LOC. 217.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED
.0	-31.0	3505.	1625.	385.	423.	385.
100.0	-31.0	3505.	1625.	385.	423.	385.
200.0	-31.0	4220.	1625.	385.	423.	385.
200.0	-31.0	4220.	1625.	385.	423.	385.
200.1	-31.0	2935.	1625.	385.	423.	385.
200.2	-31.0	2716.	1625.	385.	423.	385.
400.0	-31.0	2716.	1625.	385.	423.	385.

ASSUMED CRIT. PASSIVE LOC. 217.0 EL. -31.0 DP 33319. RP 15257.

ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS
191.0	-31.0	72555.	24413.	0.	10010.	1.27

CRIT. ACTIVE LOC 191.0 EL -31.0 DA 72555. RA 24413.

DIS.	EL.	DP	RP	DB	RB	FS
217.0	-31.0	33319.	15257.	0.	10010.	1.27

* * STRATUM 6 ACT. WEDGE LOC. 90191.0 EL. -36.0 PASS.WEDGE LOC. 221.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED
.0	-36.0	4010.	1938.	423.	462.	423.
100.0	-36.0	4010.	1938.	423.	462.	423.
200.0	-36.0	4725.	1938.	423.	462.	423.
200.0	-36.0	4725.	1938.	423.	462.	423.
200.1	-36.0	3440.	1938.	423.	462.	423.
200.2	-36.0	3221.	1938.	423.	462.	423.
400.0	-36.0	3221.	1938.	423.	462.	423.

ASSUMED CRIT. PASSIVE LOC. 221.0 EL. -36.0 DP 48163. RP 19298.

ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS
191.0	-36.0	93282.	28014.	0.	12690.	1.33

CRIT. ACTIVE LOC 191.0 EL -36.0 DA 93282. RA 28014.

DIS.	EL.	DP	RP	DB	RB	FS
221.0	-36.0	48163.	19298.	0.	12690.	1.33

* * STRATUM 7 ACT. WEDGE LOC. 90191.0 EL. -41.0 PASS.WEDGE LOC. 224.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED
.0	-41.0	4515.	2250.	462.	712.	462.
100.0	-41.0	4515.	2250.	462.	712.	462.
200.0	-41.0	5230.	2250.	462.	864.	462.
200.0	-41.0	5230.	2250.	462.	864.	462.
200.1	-41.0	3945.	2250.	462.	591.	462.
200.2	-41.0	3726.	2250.	462.	545.	462.
400.0	-41.0	3726.	2250.	462.	545.	462.

ASSUMED CRIT. PASSIVE LOC. 224.0 EL. -41.0 DP 65532. RP 23717.

ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS
191.0	-41.0	116365.	32002.	0.	15246.	1.40

CRIT. ACTIVE LOC 191.0 EL -41.0 DA 116365. RA 32002.

DIS.	EL.	DP	RP	DB	RB	FS
224.0	-41.0	65532.	23717.	0.	15246.	1.40

* * STRATUM 8 ACT. WEDGE LOC. 90191.0 EL. -60.0 PASS.WEDGE LOC. 238.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED
.0	-60.0	6795.	3438.	945.	577.	577.
100.0	-60.0	6795.	3438.	945.	577.	577.
200.0	-60.0	7510.	3438.	1097.	577.	577.
200.0	-60.0	7510.	3438.	1097.	577.	577.
200.1	-60.0	6225.	3438.	824.	577.	577.
200.2	-60.0	6006.	3438.	777.	577.	577.
400.0	-60.0	6006.	3438.	777.	577.	577.

ASSUMED CRIT. PASSIVE LOC. 238.0 EL. -60.0 DP 157991. RP 54731.

ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS
191.0	-60.0	230352.	60097.	0.	27119.	1.96

CRIT. ACTIVE LOC 191.0 EL -60.0 DA 230352. RA 60097.

DIS.	EL.	DP	RP	DB	RB	FS
238.0	-60.0	157991.	54731.	0.	27119.	1.96

* * STRATUM 9 ACT. WEDGE LOC. 90191.0 EL. -65.0 PASS.WEDGE LOC. 241.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED
.0	-65.0	7345.	3750.	577.	1269.	577.
100.0	-65.0	7345.	3750.	577.	1269.	577.
200.0	-65.0	8060.	3750.	577.	1269.	577.
200.0	-65.0	8060.	3750.	577.	1269.	577.
200.1	-65.0	6775.	3750.	577.	1269.	577.
200.2	-65.0	6556.	3750.	577.	1269.	577.
400.0	-65.0	6556.	3750.	577.	1269.	577.

ASSUMED CRIT. PASSIVE LOC. 241.0 EL. -65.0 DP 189397. RP 60501.

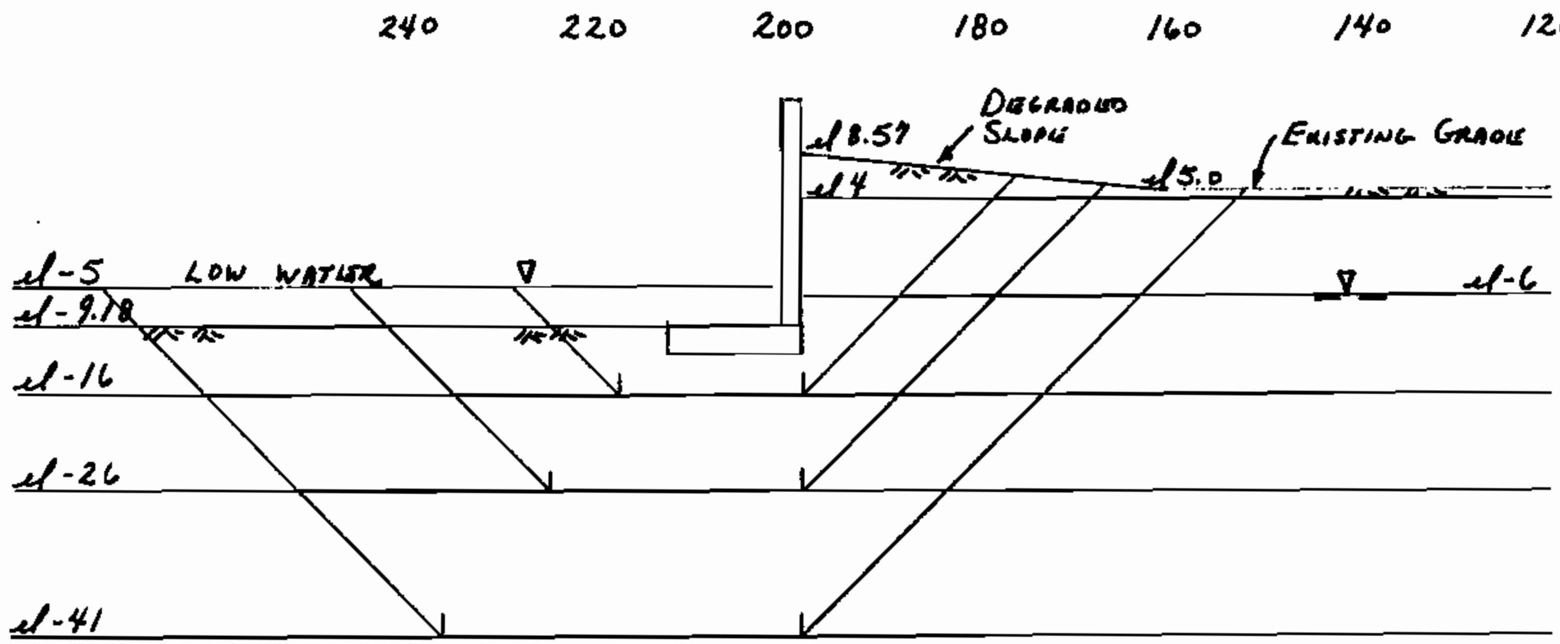
ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS
191.0	-65.0	266658.	65224.	0.	28850.	2.00

CRIT. ACTIVE LOC 191.0 EL -65.0 DA 266658. RA 65224.

DIS.	EL.	DP	RP	DB	RB	FS
241.0	-65.0	189397.	60501.	0.	28850.	2.00

```
AILURE TOWARD CANAL DURING LOW WATER EL -5.0
 -WALL MONOLITHS T-1 AND T-2   (F.S.=1.3 APPLIED TO SHEAR STRENGTHS)
20 10 1.0 200 1 1
 0 1 2 1
 00
 0 62.5  0    0
24 120   0    0
 0 110  385  385
 0 120   0    0
 0 101  365  385
 0 101  404  423
 0 101  442  462
12 120  231  231
 0 110  577  577
 0 119 1269 1269
 ,-0.43  100,-0.43  200,6.57  200.1,-5.0
400,-5.0  9999.9,0
 ,-0.43  100,-0.43  200,6.57  200.1,-5.0
 00.2,-9.18  400,-9.18  9999.9,0
0,-9.18  200.2,-9.18  400,-9.18  9999.9,0
 ,-16  400,-16  9999.9,0
 ,-26  400,-26  9999.9,0
0,-31  400,-31  9999.9,0
0,-36  400,-36  9999.9,0
 ,-41  400,-41  9999.9,0
 ,-60  400,-60  9999.9,0
0,-65  400,-65  9999.9,0
 ,-81  400,-81  9999.9,0
 ,-5  200,-5  200.1,-5  200.2,-5  400,-5  9999.9,0
1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
   90191  -16  207  -16  1
 07
4  90191  -26  214  -26  1
214
   90191  -31  217  -31  1
 17
6  90191  -36  221  -36  1
 21
   90191  -41  224  -41  1
224
8  90191  -60  238  -60  1
 38
   90191  -65  241  -65  1
241
```



FAILURE ELEVATION	FACTOR of SAFETY
-16	1.10
-26	1.26
-41	1.34

* NOTE: F.S. = 1.3 APPLIED TO SOIL SHEAR STRENGTHS

LONDON AVENUE OUTFALL CANAL
MONOLITH T-3

JOB 13065 JRE 7-20-95

**** STABILITY WITH UPLIFT ****

FAILURE TOWARD CANAL DURING LOW WATER EL -5.0
T-WALL MONOLITH T-3 (F.S.=1.3 APPLIED TO SOIL SHEAR STRENGTHS)
12 PROFILES
1VERTICALS
UPLIFT WITH 1 PIEZOMETRIC GRADE LINES

* * STRATUM 4 ACT. WEDGE LOC. 90200.0 EL. -16.0 PASS.WEDGE LOC. 217.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED
.0	-16.0	2365.	625.	385.	633.	385.
160.0	-16.0	2365.	625.	385.	633.	385.
200.0	-16.0	2728.	625.	385.	765.	385.
200.0	-16.0	2728.	625.	385.	765.	385.
200.1	-16.0	2157.	688.	385.	534.	385.
SHEAR STRENGTHS ARE EQUAL 385.0 AT DIST. 200.1						
200.2	-16.0	1210.	687.	385.	190.	190.
200.3	-16.0	1149.	687.	385.	168.	168.
200.4	-16.0	1011.	687.	385.	118.	118.
400.0	-16.0	1011.	687.	385.	118.	118.

ASSUMED CRIT. PASSIVE LOC. 217.0 EL. -16.0 DP 4886. RP 5251.

ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS
200.0	-16.0	30846.	21218.	0.	2061.	1.10

CRIT. ACTIVE LOC 200.0 EL -16.0 DA 30846. RA 21218.

DIS.	EL.	DP	RP	DB	RB	FS
217.0	-16.0	4886.	5251.	0.	2061.	1.10

* * STRATUM 5 ACT. WEDGE LOC. 90200.0 EL. -26.0 PASS.WEDGE LOC. 224.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST. ELEV. WT. UPLIFT STR 1 STR 2 STR USED

.0	-26.0	3565.	1250.	843.	385.	385.
160.0	-26.0	3565.	1250.	843.	385.	385.
200.0	-26.0	3928.	1250.	975.	385.	385.
200.0	-26.0	3928.	1250.	975.	385.	385.
200.1	-26.0	3357.	1313.	744.	385.	385.
200.2	-26.0	2410.	1313.	400.	385.	385.

SHEAR STRENGTHS ARE EQUAL 385.0 AT DIST. 200.3

200.3	-26.0	2349.	1313.	377.	385.	377.
200.4	-26.0	2211.	1313.	327.	385.	327.
400.0	-26.0	2211.	1313.	327.	385.	327.

ASSUMED CRIT. PASSIVE LOC. 224.0 EL. -26.0 DP 21000. RP 11607.

ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS
200.0	-26.0	62401.	32648.	0.	7872.	1.26

CRIT. ACTIVE LOC 200.0 EL -26.0 DA 62401. RA 32648.

DIS.	EL.	DP	RP	DB	RB	FS
224.0	-26.0	21000.	11607.	0.	7872.	1.26

* * STRATUM 6 ACT. WEDGE LOC. 90200.0 EL. -31.0 PASS.WEDGE LOC. 228.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED
.0	-31.0	4070.	1563.	385.	423.	385.
160.0	-31.0	4070.	1563.	385.	423.	385.
200.0	-31.0	4433.	1563.	385.	423.	385.
200.0	-31.0	4433.	1563.	385.	423.	385.
200.1	-31.0	3862.	1626.	385.	423.	385.
200.2	-31.0	2915.	1625.	385.	423.	385.
200.3	-31.0	2854.	1625.	385.	423.	385.
200.4	-31.0	2716.	1625.	385.	423.	385.
400.0	-31.0	2716.	1625.	385.	423.	385.

ASSUMED CRIT. PASSIVE LOC. 228.0 EL. -31.0 DP 33319. RP 15257.

ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS

200.0	-31.0	81752.	35645.	0.	10780.	1.27

CRIT. ACTIVE LOC 200.0 EL -31.0 DA 81752. RA 35645.

DIS.	EL.	DP	RP	DB	RB	FS
228.0	-31.0	33319.	15257.	0.	10780.	1.27

* * STRATUM 7 ACT. WEDGE LOC. 90200.0 EL. -36.0 PASS.WEDGE LOC. 231.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED
.0	-36.0	4575.	1875.	423.	462.	423.
160.0	-36.0	4575.	1875.	423.	462.	423.
200.0	-36.0	4938.	1875.	423.	462.	423.
200.0	-36.0	4938.	1875.	423.	462.	423.
200.1	-36.0	4367.	1938.	423.	462.	423.
200.2	-36.0	3420.	1938.	423.	462.	423.
200.3	-36.0	3359.	1938.	423.	462.	423.
200.4	-36.0	3221.	1938.	423.	462.	423.
400.0	-36.0	3221.	1938.	423.	462.	423.

ASSUMED CRIT. PASSIVE LOC. 231.0 EL. -36.0 DP 48163. RP 19297.

ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS
200.0	-36.0	103417.	39033.	0.	13113.	1.29

CRIT. ACTIVE LOC 200.0 EL -36.0 DA 103417. RA 39033.

DIS.	EL.	DP	RP	DB	RB	FS
231.0	-36.0	48163.	19297.	0.	13113.	1.29

* * STRATUM 8 ACT. WEDGE LOC. 90200.0 EL. -41.0 PASS.WEDGE LOC. 235.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED

.0	-41.0	5080.	2188.	462.	846.	462.
160.0	-41.0	5080.	2188.	462.	846.	462.
200.0	-41.0	5443.	2188.	462.	923.	462.
200.0	-41.0	5443.	2188.	462.	923.	462.
200.1	-41.0	4872.	2251.	462.	788.	462.
200.2	-41.0	3925.	2250.	462.	587.	462.
200.3	-41.0	3864.	2250.	462.	574.	462.
200.4	-41.0	3726.	2250.	462.	545.	462.
400.0	-41.0	3726.	2250.	462.	545.	462.

ASSUMED CRIT. PASSIVE LOC. 235.0 EL. -41.0 DP 65532. RP 23717.

ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS
200.0	-41.0	127436.	43053.	0.	16170.	1.34

CRIT. ACTIVE LOC 200.0 EL -41.0 DA 127436. RA 43053.

DIS.	EL.	DP	RP	DB	RB	FS
235.0	-41.0	65532.	23717.	0.	16170.	1.34

* * STRATUM 9 ACT. WEDGE LOC. 90200.0 EL. -60.0 PASS.WEDGE LOC. 248.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED
.0	-60.0	7360.	3375.	1078.	577.	577.
160.0	-60.0	7360.	3375.	1078.	577.	577.
200.0	-60.0	7723.	3375.	1155.	577.	577.
200.0	-60.0	7723.	3375.	1155.	577.	577.
200.1	-60.0	7152.	3438.	1020.	577.	577.
200.2	-60.0	6205.	3438.	819.	577.	577.
200.3	-60.0	6144.	3438.	806.	577.	577.
200.4	-60.0	6006.	3438.	777.	577.	577.
400.0	-60.0	6006.	3438.	777.	577.	577.

ASSUMED CRIT. PASSIVE LOC. 248.0 EL. -60.0 DP 157992. RP 54731.

ACTIVE WEDGE DATA

DIST.	ELEV.	DA	RA	DB	RB	FS
200.0	-60.0	246255.	73857.	0.	27696.	1.77

CRIT. ACTIVE LOC 200.0 EL -60.0 DA 246255. RA 73857.

DIS.	EL.	DP	RP	DB	RB	FS
248.0	-60.0	157992.	54731.	0.	27696.	1.77

* * STRATUM 10 ACT. WEDGE LOC. 90200.0 EL. -65.0 PASS.WEDGE LOC. 252.0 EL.

ASSUMED FAILURE SURFACE DATA

DIST.	ELEV.	WT.	UPLIFT	STR 1	STR 2	STR USED
.0	-65.0	7910.	3688.	577.	1269.	577.
160.0	-65.0	7910.	3688.	577.	1269.	577.
200.0	-65.0	8273.	3688.	577.	1269.	577.
200.0	-65.0	8273.	3688.	577.	1269.	577.
200.1	-65.0	7702.	3751.	577.	1269.	577.
200.2	-65.0	6755.	3750.	577.	1269.	577.
200.3	-65.0	6694.	3750.	577.	1269.	577.
200.4	-65.0	6556.	3750.	577.	1269.	577.
400.0	-65.0	6556.	3750.	577.	1269.	577.

ASSUMED CRIT. PASSIVE LOC. 252.0 EL. -65.0 DP 189398. RP 60501.

ACTIVE WEDGE DATA

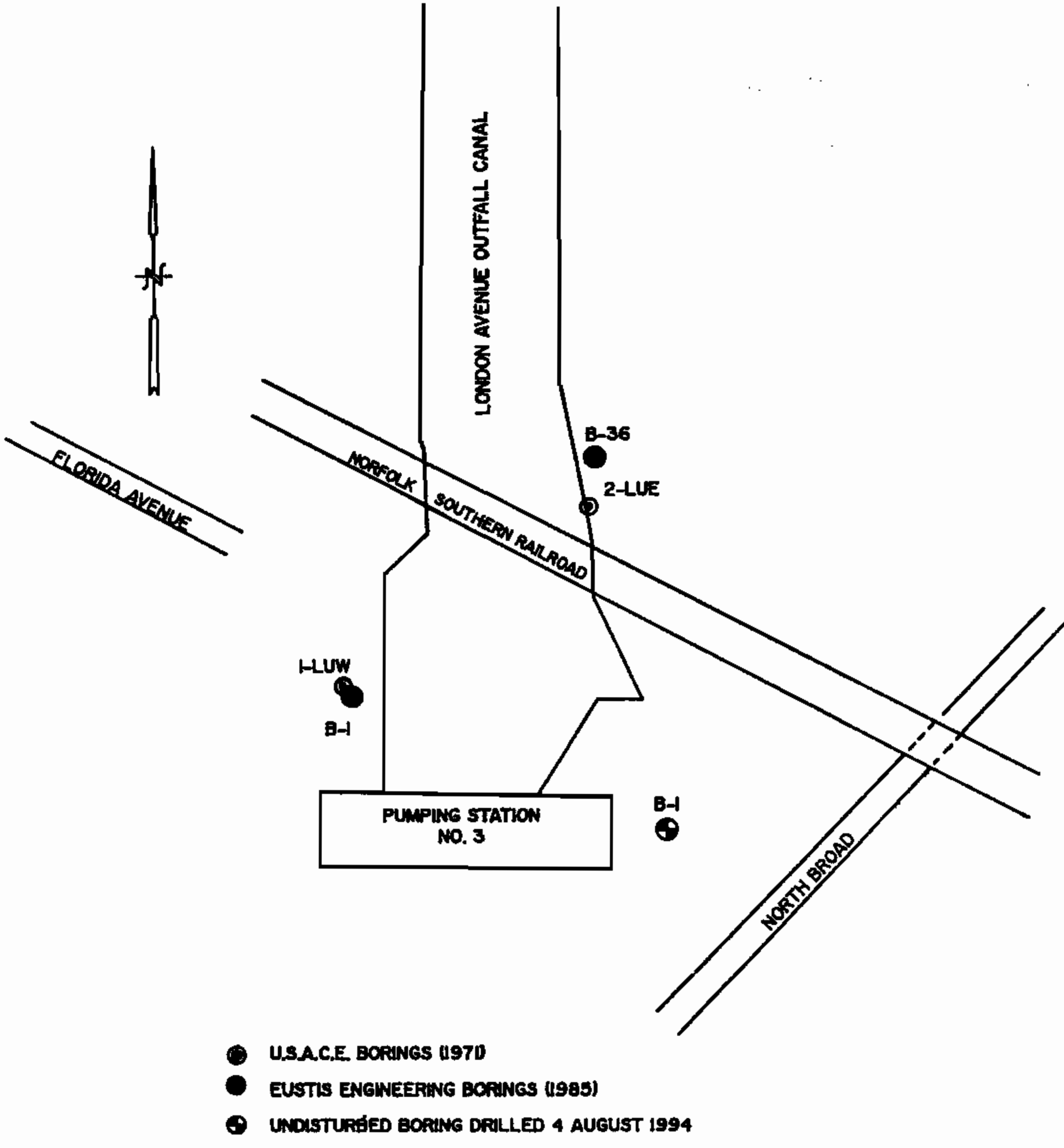
DIST.	ELEV.	DA	RA	DB	RB	FS
200.0	-65.0	284141.	79119.	0.	30004.	1.79

CRIT. ACTIVE LOC 200.0 EL -65.0 DA 284141. RA 79119.

DIS.	EL.	DP	RP	DB	RB	FS
252.0	-65.0	189398.	60501.	0.	30004.	1.79

```
FAILURE TOWARD CANAL DURING LOW WATER EL -5.0
T-WALL MONOLITH T-3   (F.S.=1.3 APPLIED TO SOIL SHEAR STRENGTHS)
20 10 1.0 200 1 1
11 1 2 1
200
 0 62.5  0    0
 0 115  577  577
 0 115  538  538
 0 110  385  385
20 120    0    0
 0 101  365  385
 0 101  404  423
 0 101  442  462
12 120  231  231
 0 110  577  577
 0 119 1269 1269
0,5  160,5  200,8.57  200.1,4  200.2,-5  400,-5
9999.9,0
0,5  160,5  200,8.57  200.1,4  200.2,-5  200.3,-6
200.4,-9.18  400,-9.18  9999.9,0
0,4  200.1,4  200.2,-5  200.3,-6  200.4,-9.18
400,-9.18  9999.9,0
0,-6  200.3,-6  200.4,-9.18  400,-9.18  9999.9,0
0,-16  400,-16  9999.9,0
0,-26  400,-26  9999.9,0
0,-31  400,-31  9999.9,0
0,-36  400,-36  9999.9,0
0,-41  400,-41  9999.9,0
0,-60  400,-60  9999.9,0
0,-65  400,-65  9999.9,0
0,-81  400,-81  9999.9,0
0,-6 200,-6 200.1,-5 200.2,-5 200.3,-5 200.4,-5 400,-5 9999.9,0
1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
4  90200  -16  217  -16  1
217
5  90200  -26  224  -26  1
224
6  90200  -31  228  -31  1
228
7  90200  -36  231  -36  1
231
8  90200  -41  235  -41  1
235
9  90200  -60  248  -60  1
248
10  90200  -65  252  -65  1
252
```

FIGURES 1 - 8



LOCATION OF BORINGS

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

EUSTIS ENGINEERING COMPANY, INC.

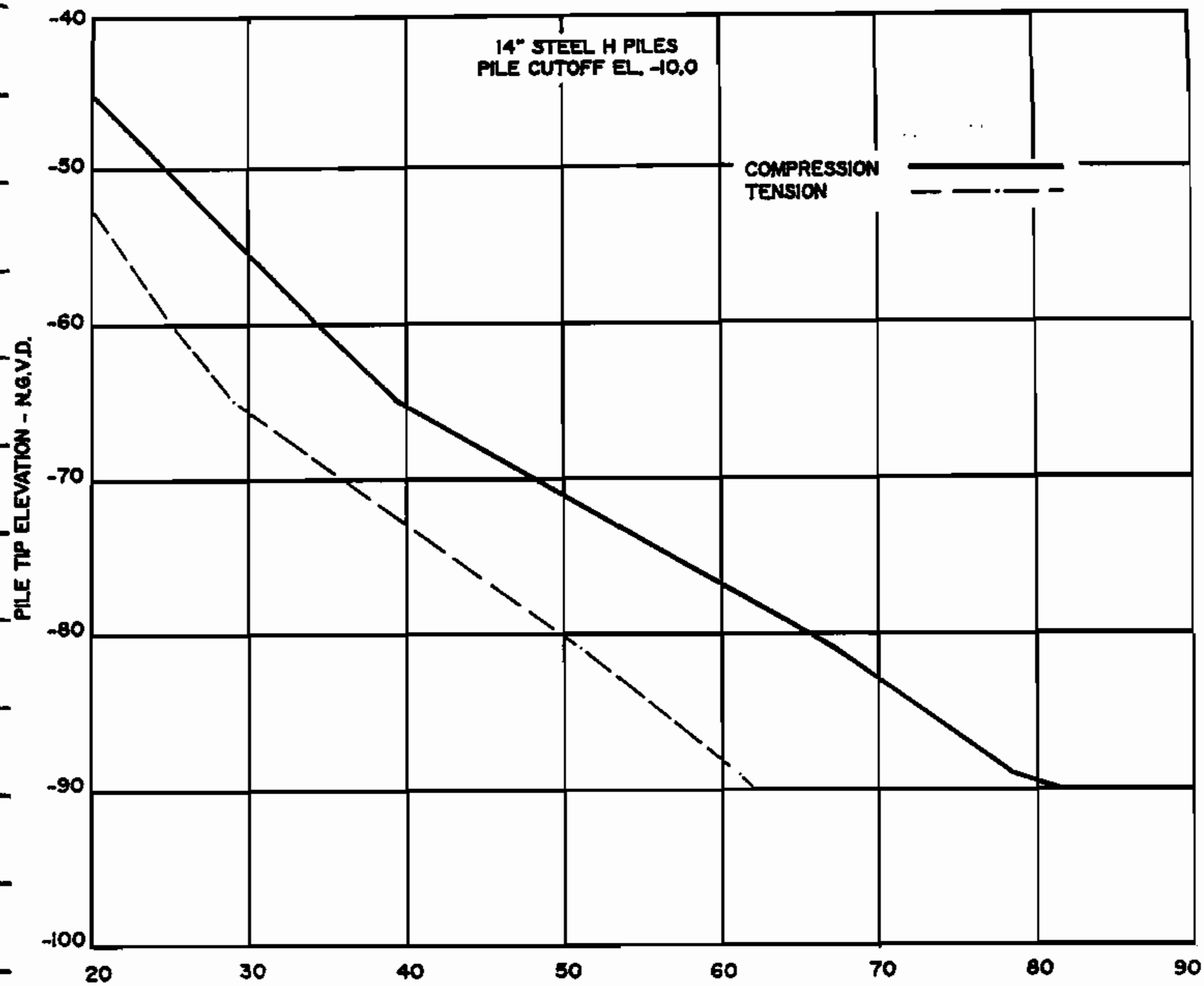
FIGURE 1

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.



ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITY - TONS
FACTOR OF SAFETY = 2

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

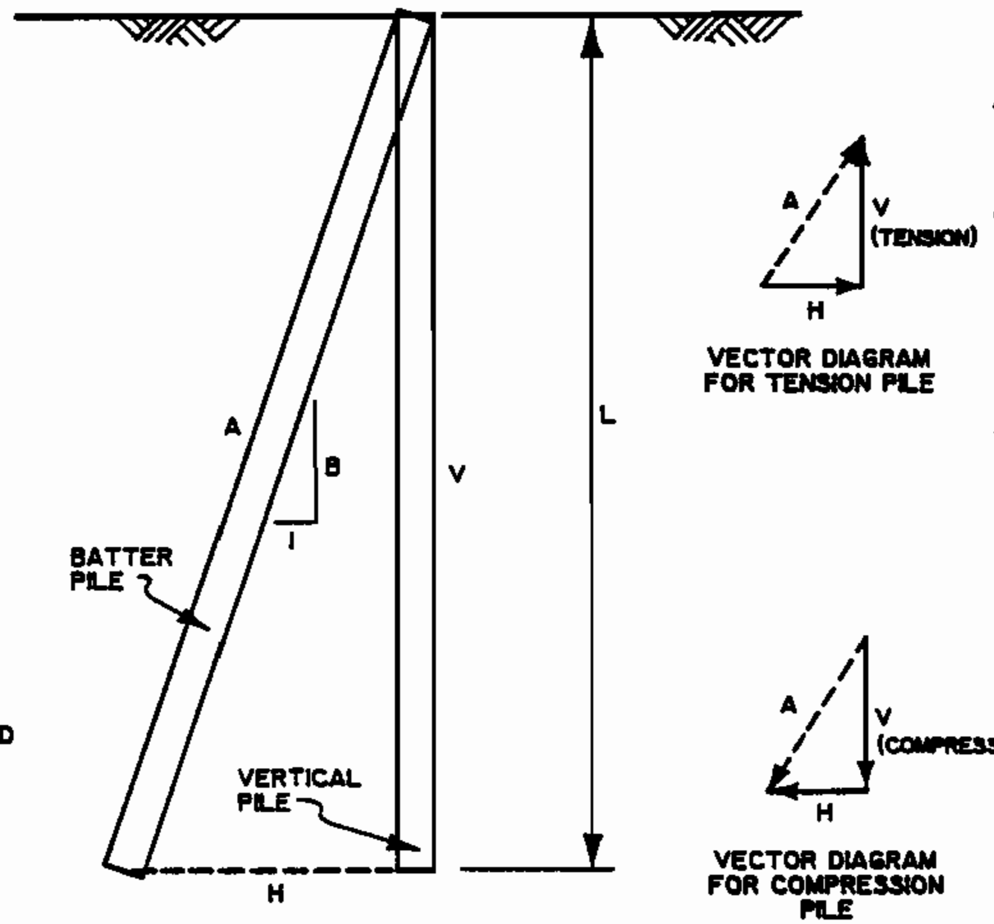
L = VERTICAL COMPONENT OF BATTER PILE EMBEDMENT LENGTH.

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.

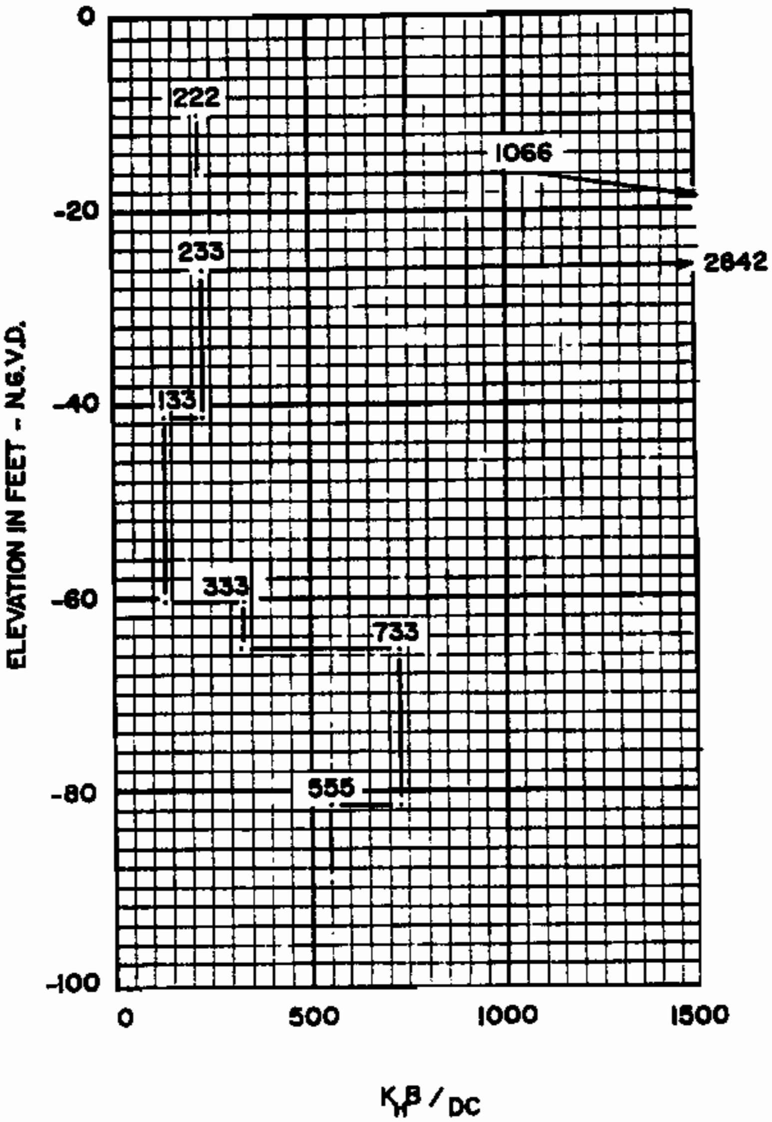
SPACING OF PILE GROUPS

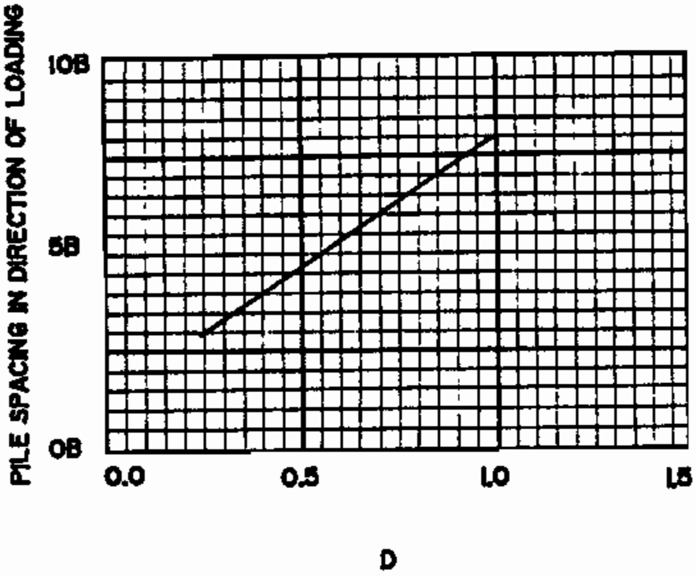
$$SPAC = 0.05\,(L_1) + 0.025\,(L_2) + 0.0125\,(L_3)$$

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 :

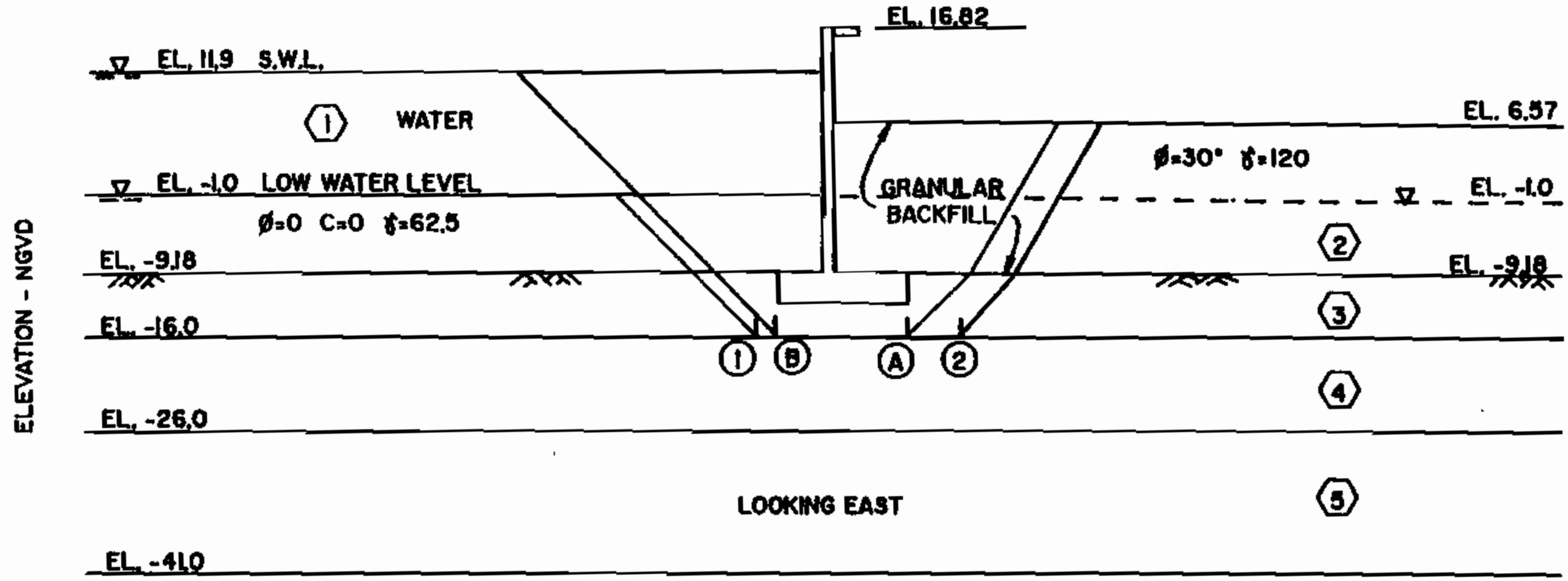
C = 0.5 FOR CYCLIC LOADING
C = 1.0 FOR INITIAL LOADING
B = PILE WIDTH OR DIAMETER - INCHES
D = GROUP EFFECT REDUCTION FACTOR
K_H = MODULUS OF HORIZONTAL SUBGRADE REACTION - LBS/IN^3

SUBGRADE MODULI

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

FIGURE 6

EUSTIS ENGINEERING COMPANY, INC.



SCALE : 1" = 20'

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

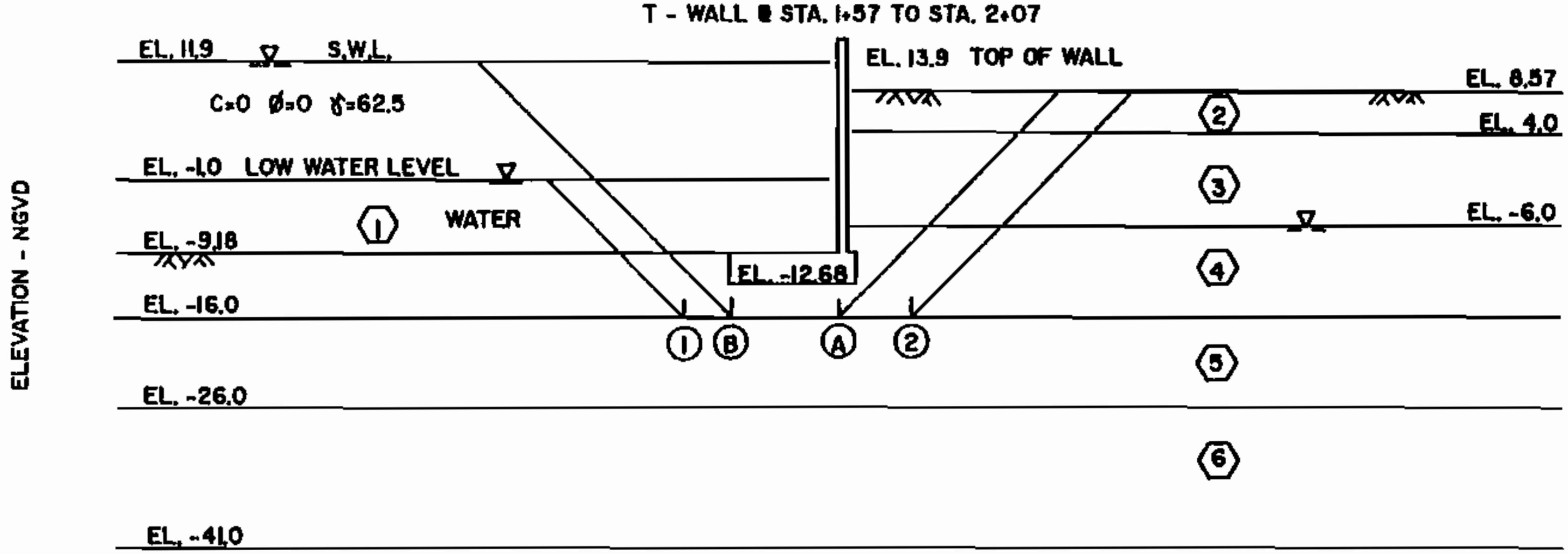
⬡ STRATA NUMBER, SEE FIGURE 2 FOR SOIL DESIGN PARAMETERS BELOW EL. -9.18 NGVD

* 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	SUMMATION OF FORCES LBS/FT 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

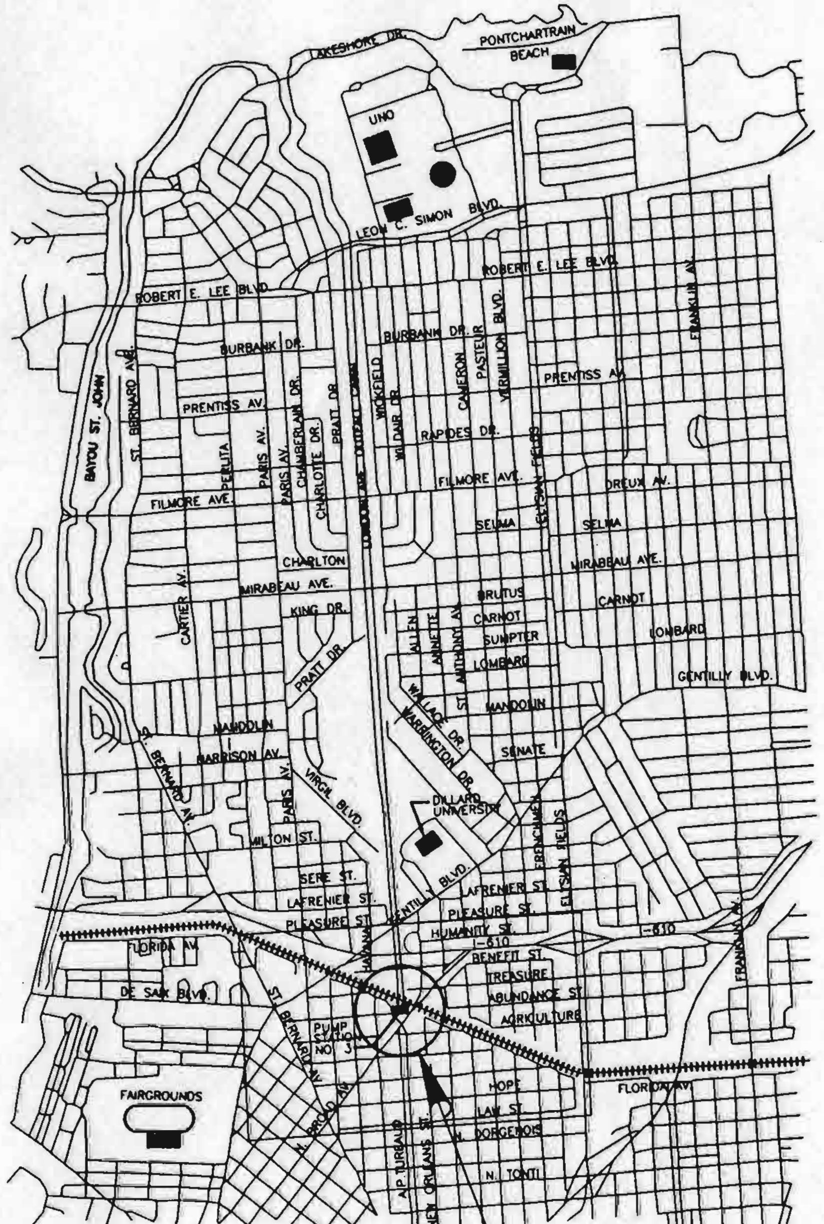
STABILITY ANALYSES

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

FIGURE 6

EUSTIS ENGINEERING COMPANY, INC.

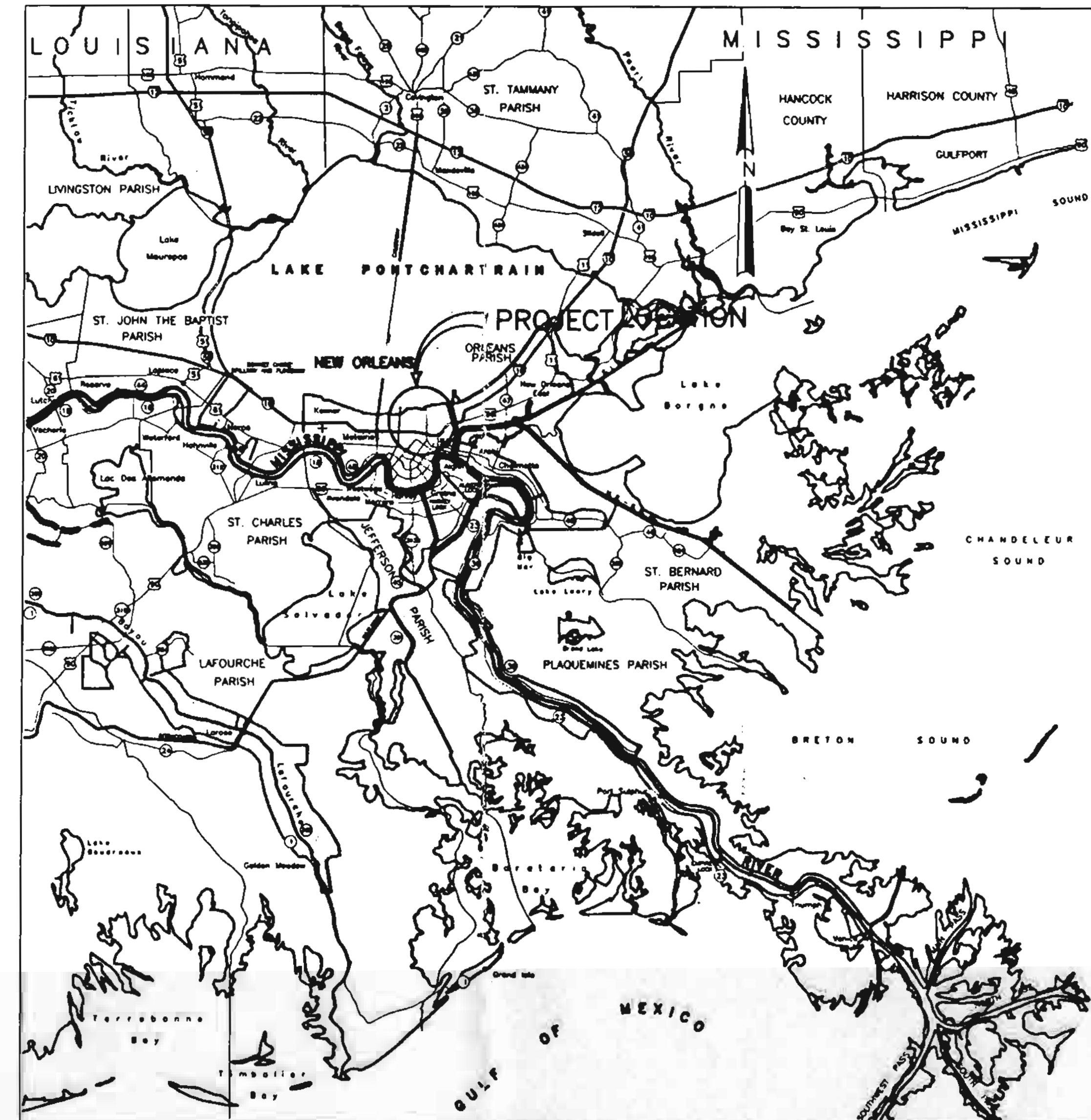
PLATES 1 - 32



LOCATION MAP

SCALE: 1" = 300'

300' 0 300' 600' 900' 1200'



VICINITY MAP

SCALE IN MILES

10 0 10 20 30 40 50

TABULATION OF BENCH MARKS		
DESIGNATION	DESCRIPTION	ELEVATION
NGS P-153	IN NEW ORLEANS, AT THE INTERSECTION OF LAKESHORE DRIVE AND THE LONDON AVENUE CANAL, SET IN THE TOP OF THE EAST END OF THE PEDESTRIAN WALKWAY ALONG SOUTH SIDE OF THE BRIDGE SPANNING THE CANAL, OVER THE EAST ABUTMENT OF THE BRIDGE, 5 FEET SOUTH OF THE SOUTH CURB OF THE DRIVE, 6 INCHES WEST OF THE EAST END OF BRIDGE AND APPROX. 1 FT. ABOVE THE DRIVE.	11.27' N.G.V.D. 1964 EPOCH

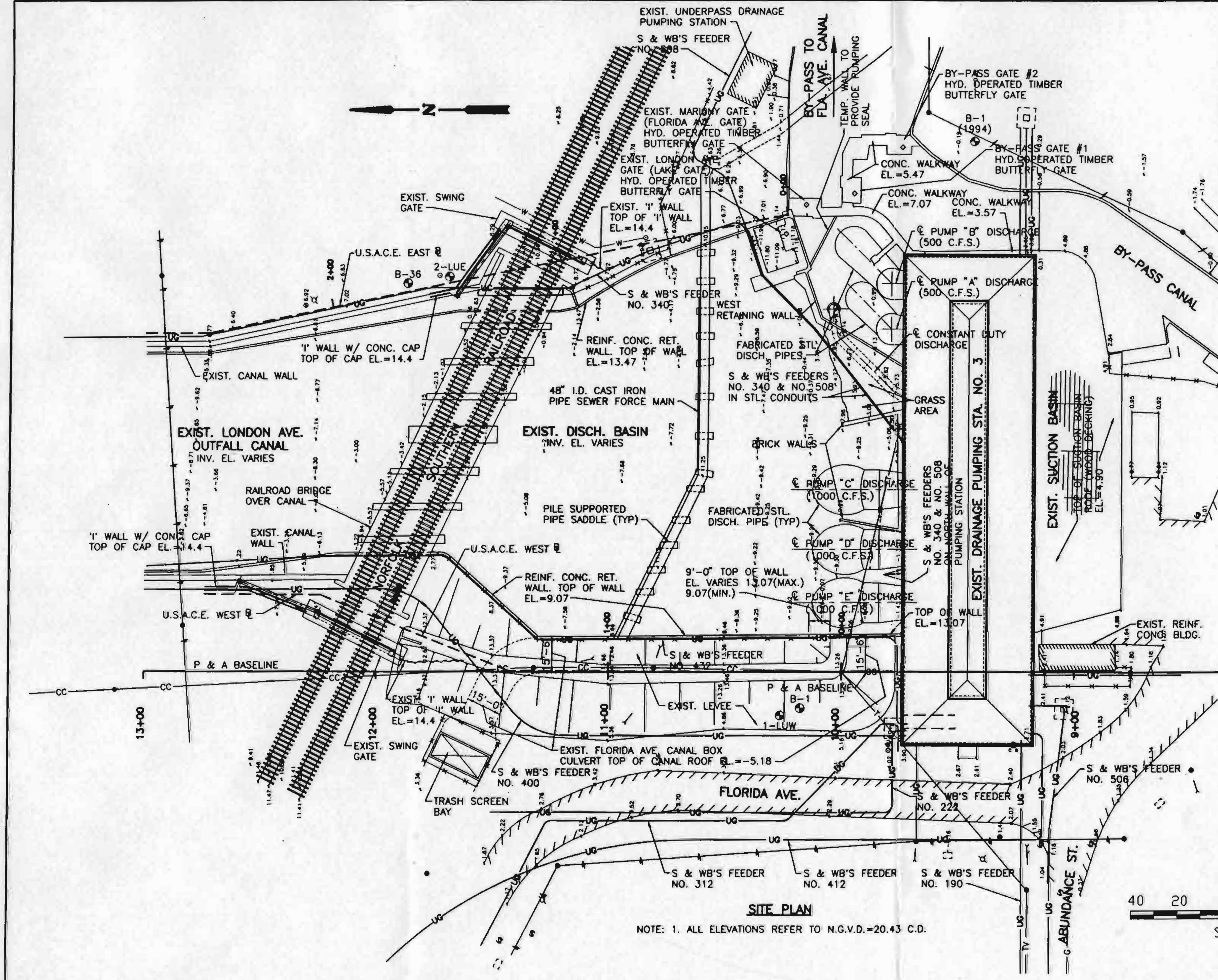
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET — METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A–GENERAL DESIGN
LONDON AVE. OUTFALL CANAL–SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

LOCATION AND VICINITY MAP

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

DESIGNED BY: SIV DRAWN BY: MSC CHECKED BY: SIV	PLOT SCALE 4800	PLOT DATE 09/18/95	FILE NO. H-2-40514
	DATE 12/01/94		



SITE PLAN

NOTE: 1. ALL ELEVATIONS REFER TO N.G.V.D.=20.43 C.D.

40 20 0 40 80FT

SCALE: 1"= 20'

LEGEND

EXISTING (THIN LINE)		EXIST. FIRE HYDRANT (FH)	
PROPOSED (HEAVY LINE)		ABANDONED UTILITY	
PROPOSED CONC. PAVEMENT		EXIST. UTILITY TO BE REMOVED	
EXIST. ROADWAY TO REMAIN		EXIST. OVERHEAD UTILITY LINES	
EXIST. ROADWAY TO BE REMOVED		SOIL BORING LOCATIONS	
REQ'D. SHELL OR SAND/SHELL		GAS MANHOLE	
EXIST. BUILDING		TELEPHONE MANHOLE	
EXIST. FENCE LINE		TELEGRAPH MANHOLE	
TREE		ELECTRIC MANHOLE	
HEDGE		UTILITY VALVE	
DRAIN LINE OR STORM SEWER		WATER METER (WM)	
SANITARY SEWER LINE (S) OR SEWER FORCED MAIN (SFM)		RESIDENTIAL MAIL BOX	
WATER MAIN		TELEPHONE INTERFACE BOX	
GAS LINE		CLEAN OUT (S FOR SEWER, D FOR DRAIN)	
OVERHEAD TELPHONE CABLE		POWER POLE (PP) (SPECIFY IF STEEL)	
OVERHEAD TELEVISION CABLE		TRASH RECEPTACLE (SUB-SURFACE)	
COMMUNICATION CABLE		GUY POLE	
PROPOSED MANHOLE (MH)		GUY WIRE & ANCHOR	
EXIST. CATCH BASIN		PARKING METER	
EXIST. MANHOLE		TELEPHONE POLE	
DROP INLET (D.I.)		COMBINATION POLE	
WATER VALVE MANHOLE		S & WB's ELECTRICAL MANHOLE	
DRAIN MANHOLE		EXIST. GRADE	20.72
SEWER MANHOLE		FINISHED GRADE	20.72
WATER MANHOLE		UNDERGROUND ELECTRICAL FEEDERS IN CONCRETE ENVELOPE	
LIGHT STANDARD		UNDERGROUND ELEC. FEEDERS IN CONC. DUCT	
TRAFFIC SIGNAL STANDARD			

BENCH MARK

REFERENCE BENCH MARK: NGS P-153 ELEVATION: 11.27' (1964 EPOCH)

IN NEW ORLEANS, AT THE INTERSECTION OF LAKESHORE DRIVE AND THE LONDON AVENUE CANAL, SET IN THE TOP OF THE EAST END OF THE PEDESTRIAN WALKWAY ALONG THE SOUTH SIDE OF THE BRIDGE SPANNING THE CANAL, OVER THE EAST ABUTMENT OF THE BRIDGE, 5 FEET SOUTH OF THE SOUTH CURB OF THE DRIVE, 6 INCHES WEST OF THE EAST END OF BRIDGE AND APPROX. 1 FT. ABOVE THE DRIVE.

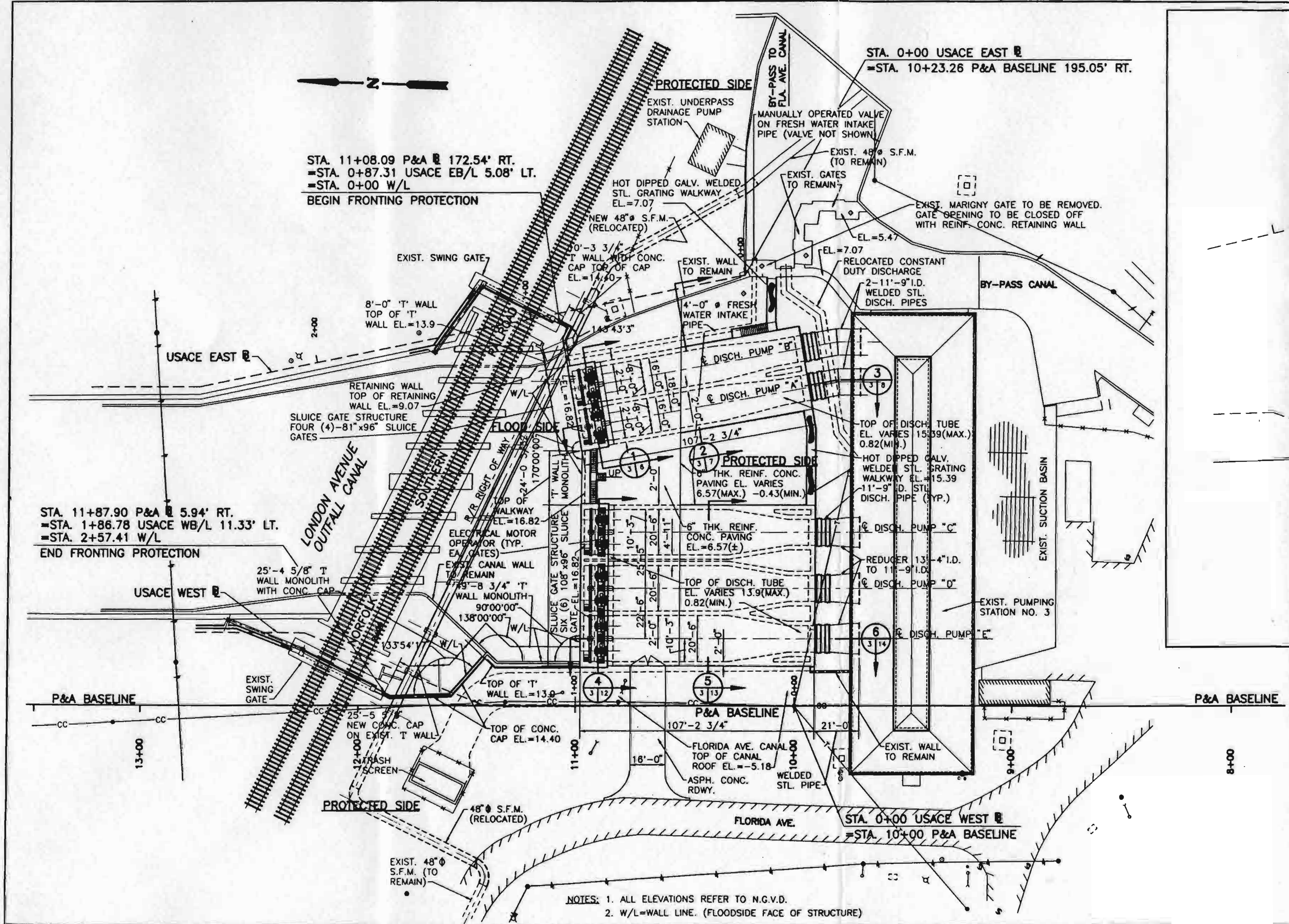
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 28th STREET METAIRIE, LOUISIANA

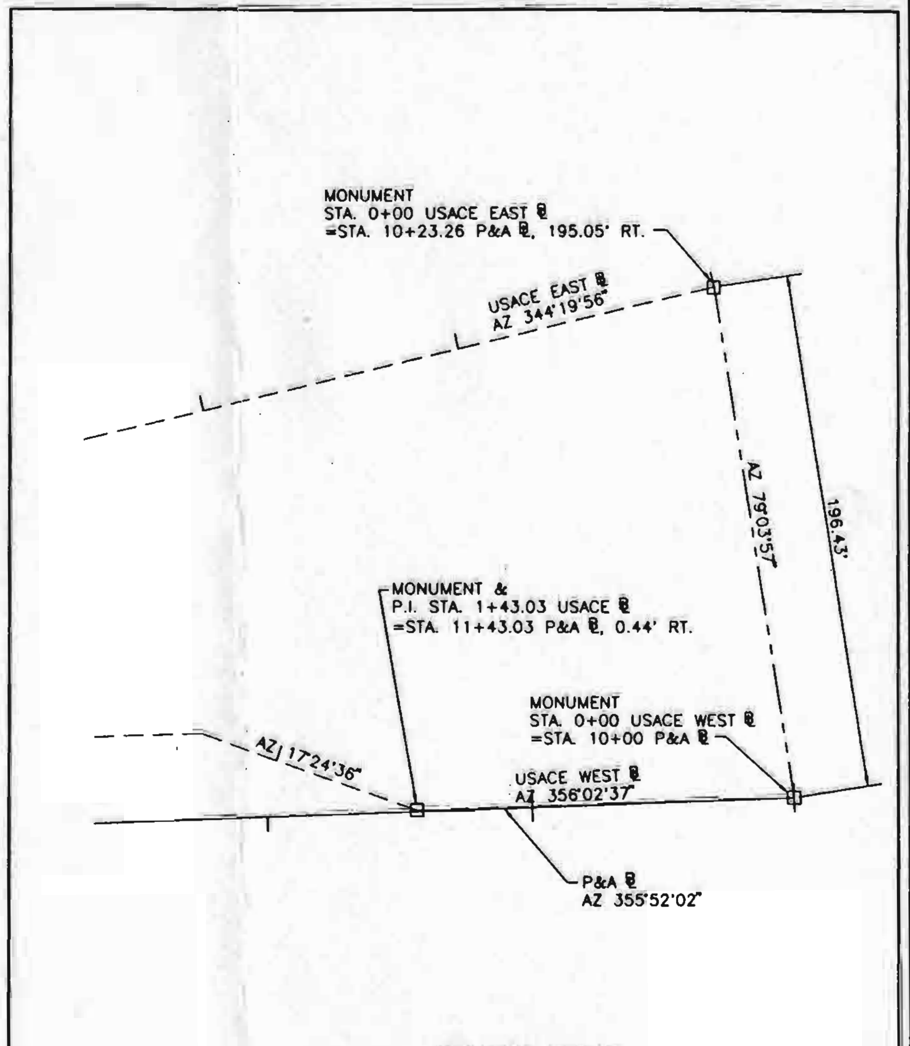
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

EXISTING SITE PLAN

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

DESIGNED BY: S.M. DRAWN BY: M.E.C CHECKED BY: S.M.
PLOT SCALE 240 | PLOT DATE: 09/18/95 | DATE: 11/11/94
FILE NO. H-2-40514





BASELINE LAYOUT

SCALE: 1"=30'

40 20 0 40 80FT

SCALE: 1"= 20'

NOTES: 1. ALL ELEVATIONS REFER TO N.G.V.D.
2. W/L=WALL LINE. (FLOODSIDE FACE OF STRUCTURE)

PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

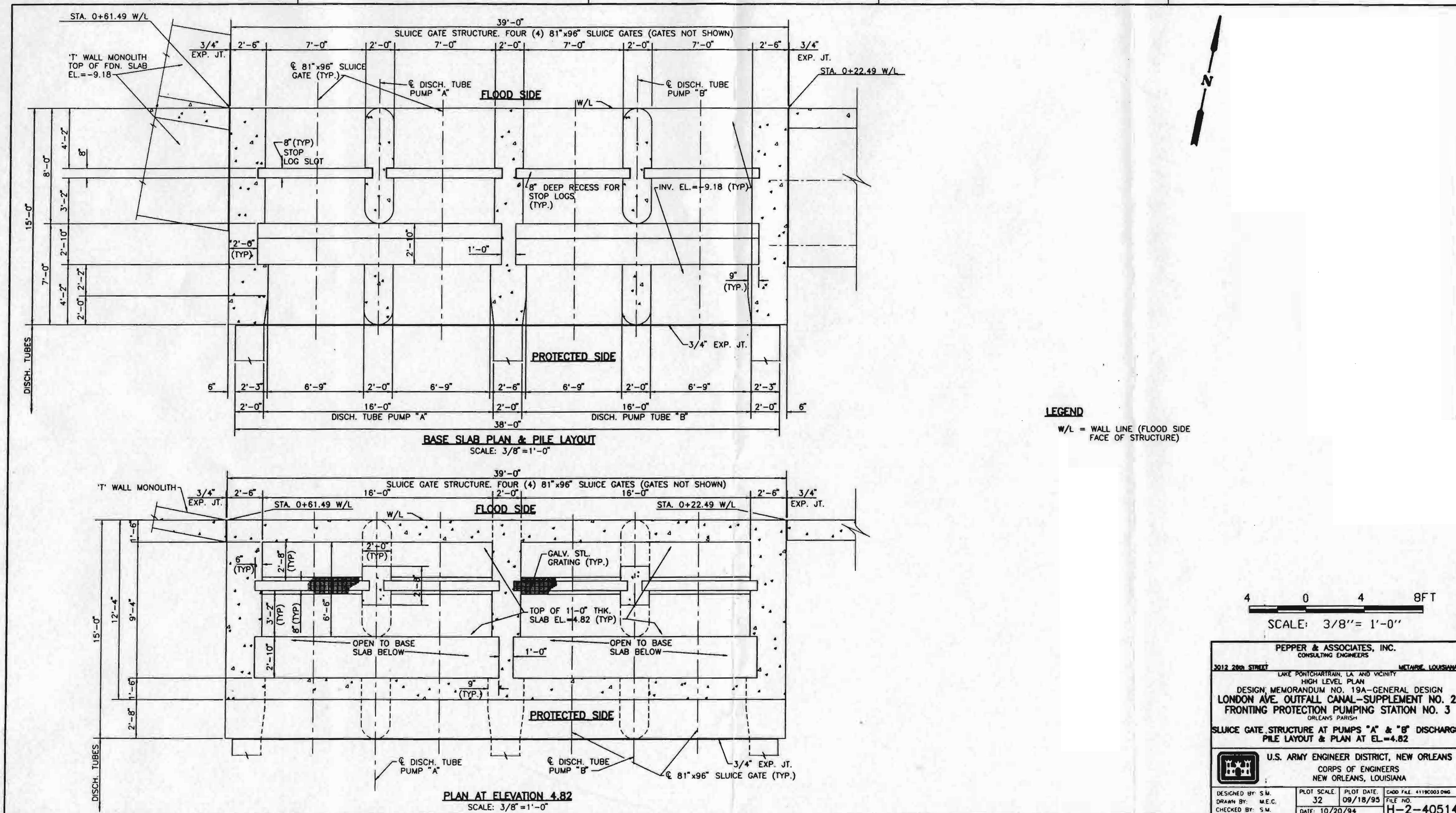
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

GENERAL LAYOUT PLAN

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

DESIGNED BY: S.M. DRAWN BY: M.E.C. CHECKED BY: S.M.	PLOT SCALE: 240	PLOT DATE: 09/15/95	CADD FILE: 4119L006.DWG
	DATE: 11/10/94		FILE NO. H-2-40514

PLATE 3



PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 20th STREET METAIRIE, LOUISIANA

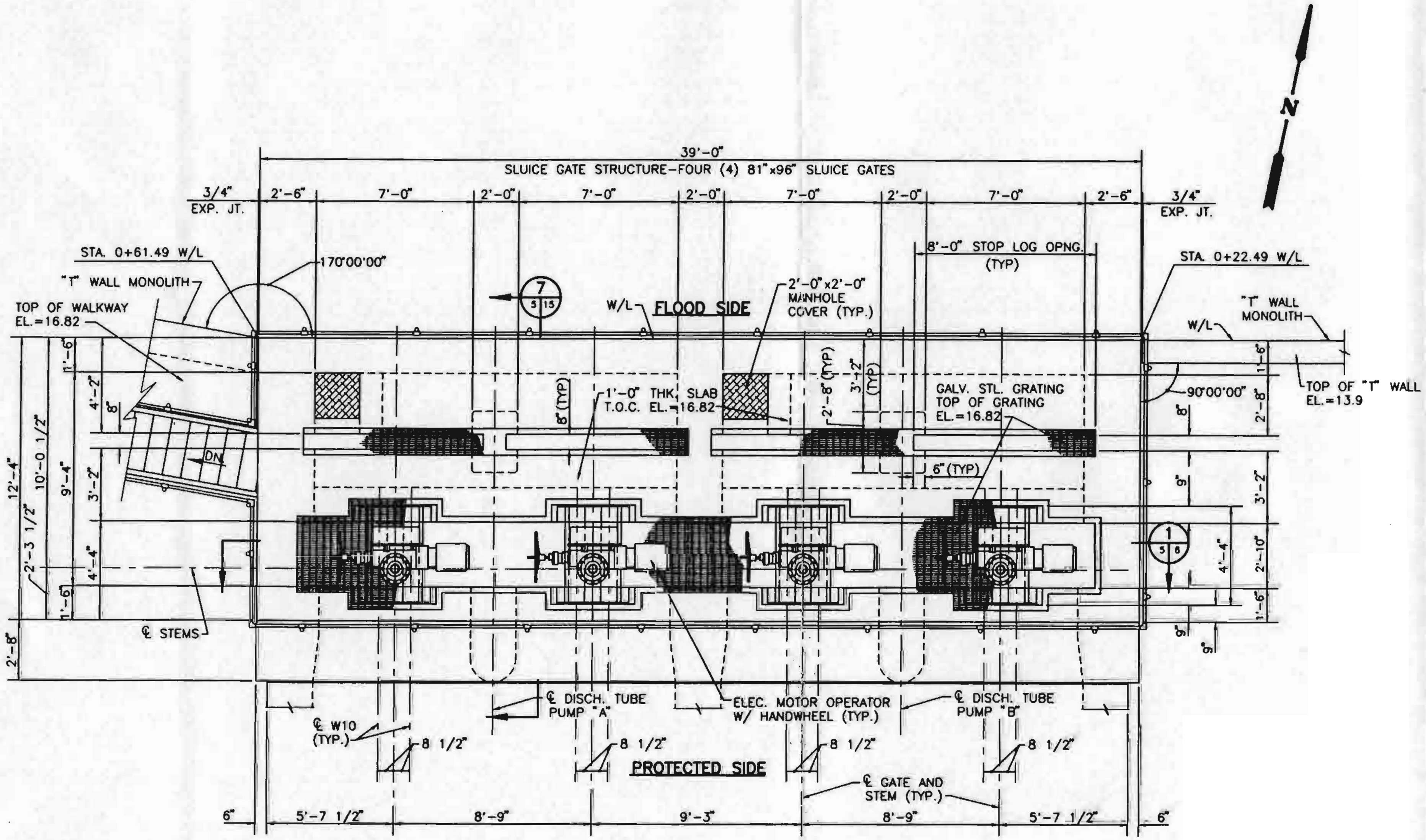
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

SLUICE GATE STRUCTURE AT PUMPS "A" & "B" DISCHARGE
PILE LAYOUT & PLAN AT EL.=4.82

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

DESIGNED BY: S.M.	PLOT SCALE 32	PLOT DATE 09/18/95	CADD FILE 4119C003.DWG
DRAWN BY: M.E.C.	DATE: 10/20/94		FILE NO. H-2-40514
CHECKED BY: S.M.			

PLATE 4



PLAN AT ELEVATION 16.82

SCALE: 3/8"=1'-0"

LEGEND

W/L = WALL LINE (FLOOD SIDE FACE OF STRUCTURE)

SCALE: 3/8"= 1'-0"

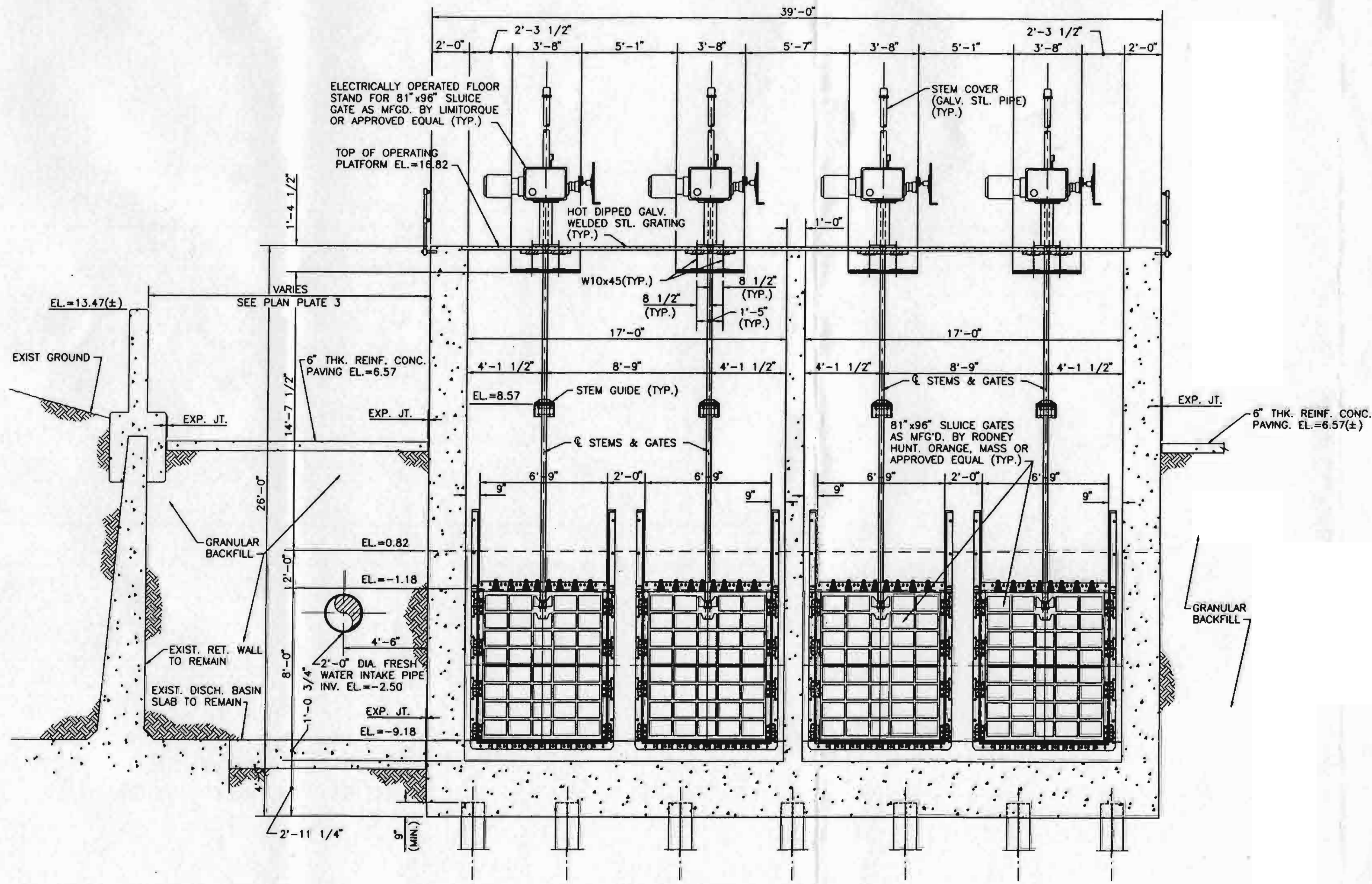
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH
SLUICE GATE STRUCTURE
AT PUMPS "A" & "B" DISCHARGE
PLAN AT EL.=16.82

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

DESIGNED BY: S.M.
DRAWN BY: M.E.C.
CHECKED BY: S.M.
PLOT SCALE 32
PLOT DATE: 09/18/95
DATE: 10/25/94
FILE NO. H-2-40514

PLATE 5



SECTION (1/5 6) (1/3 6)
SCALE: 3/8" = 1'-0"

NOTE: 1. REINFORCING NOT SHOWN
2. EXIST. FDN. PILES NOT SHOWN

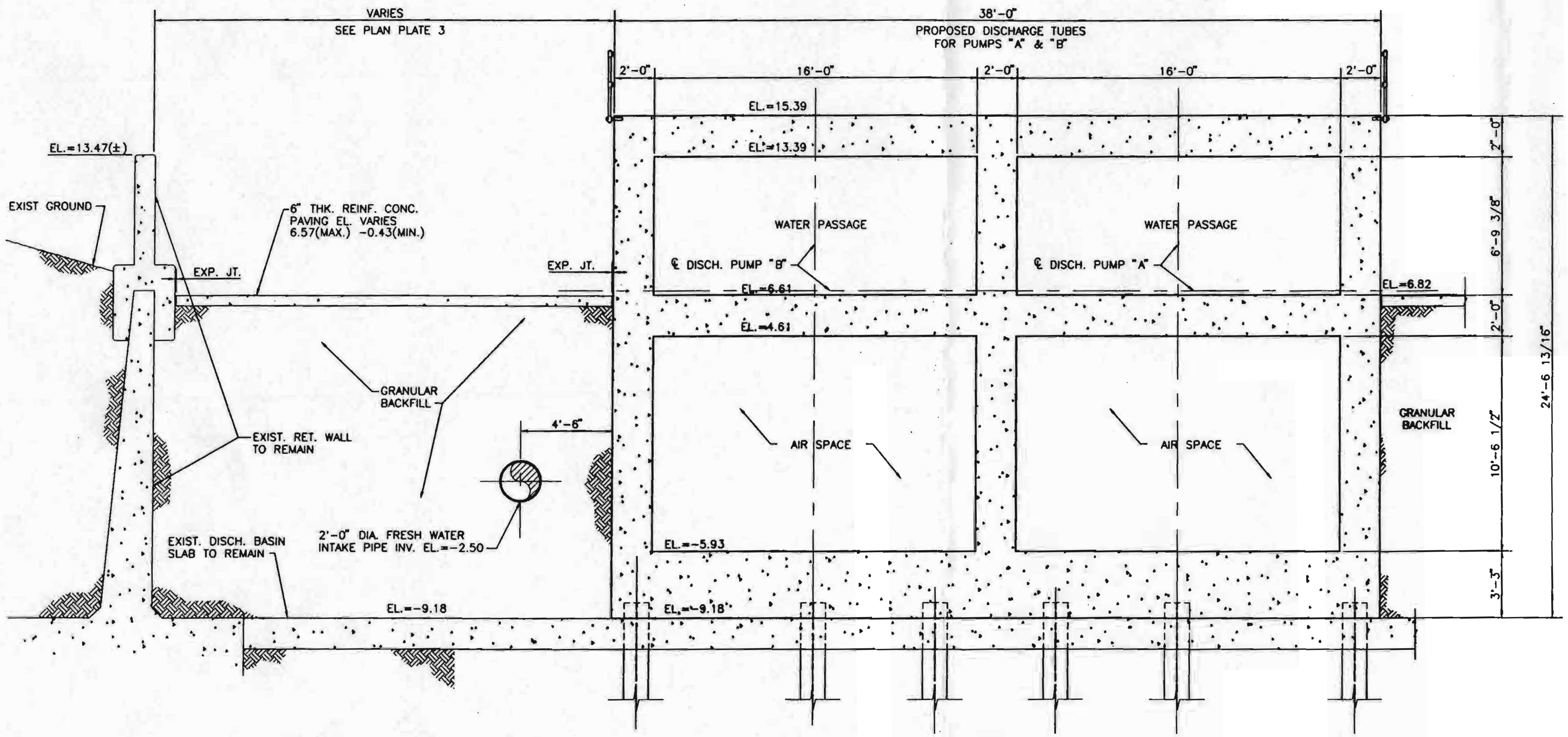
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH
LONGITUDINAL SECTION AT SLUICE GATE STRUCTURE AT PUMPS "A" & "B" DISCHARGE

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

DESIGNED BY: S.M. DRAWN BY: K.B.B. CHECKED BY: S.M.	PLOT SCALE: 32	PLOT DATE: 09/18/95	CADD FILE: 41198001.DWG
	DATE: 11/8/94		FILE NO. H-2-40514

PLATE 6



SECTION (2/3-7) (2/8-7)
SCALE: 3/8" = 1'-0"

NOTE: 1. REINF. NOT SHOWN
2. EXIST. FDN. PILES NOT SHOWN

4 0 4 8FT
SCALE: 3/8" = 1'-0"

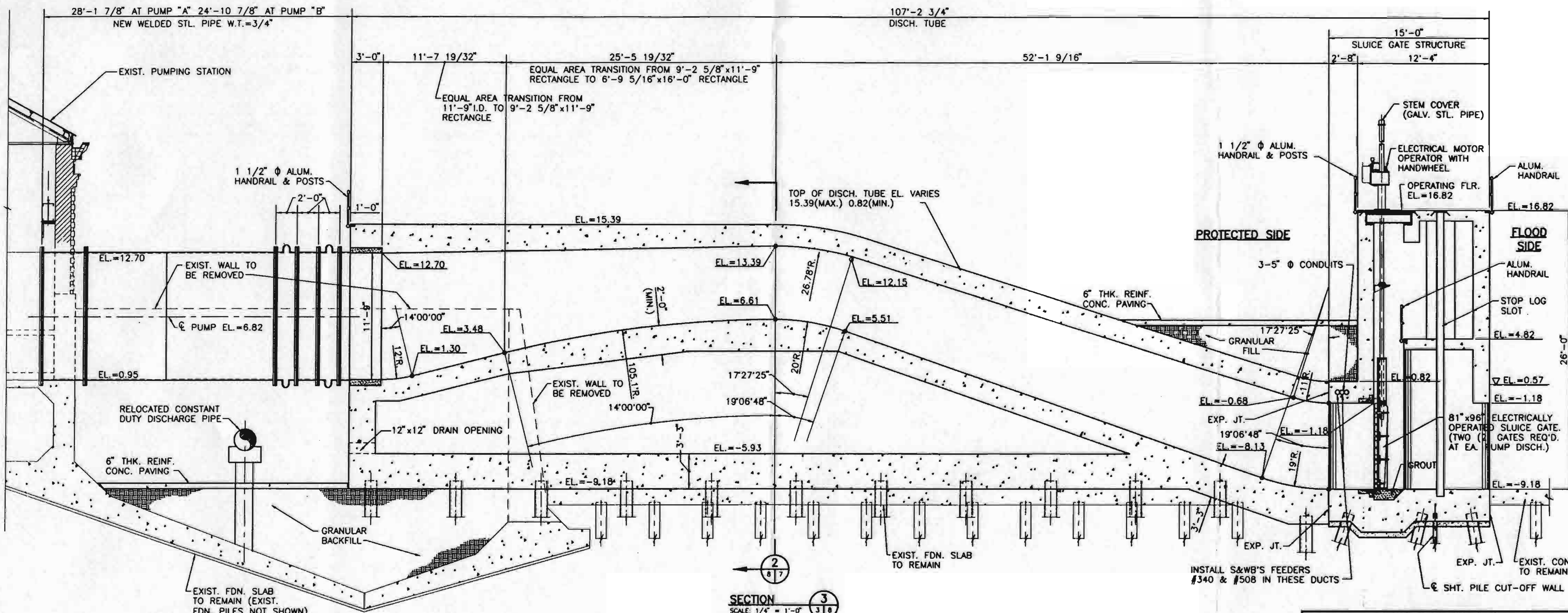
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 28th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN: MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH
CROSS SECTION - DISCHARGE TUBES
FOR PUMPS "A" & "B"

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

DESIGNED BY: S.M.	PLOT SCALE: 32	PLOT DATE: 09/18/95	FILE NO. H-2-40514
DRAWN BY: K.B.B.	DATE: 11/8/94		
CHECKED BY: S.M.			

PLATE 7



SECTION 3/3-8
SCALE: 1/4" = 1'-0"

NOTE: REINF. NOT SHOWN

8 0 8 16FT
SCALE: 1/4"=1'-0"

PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

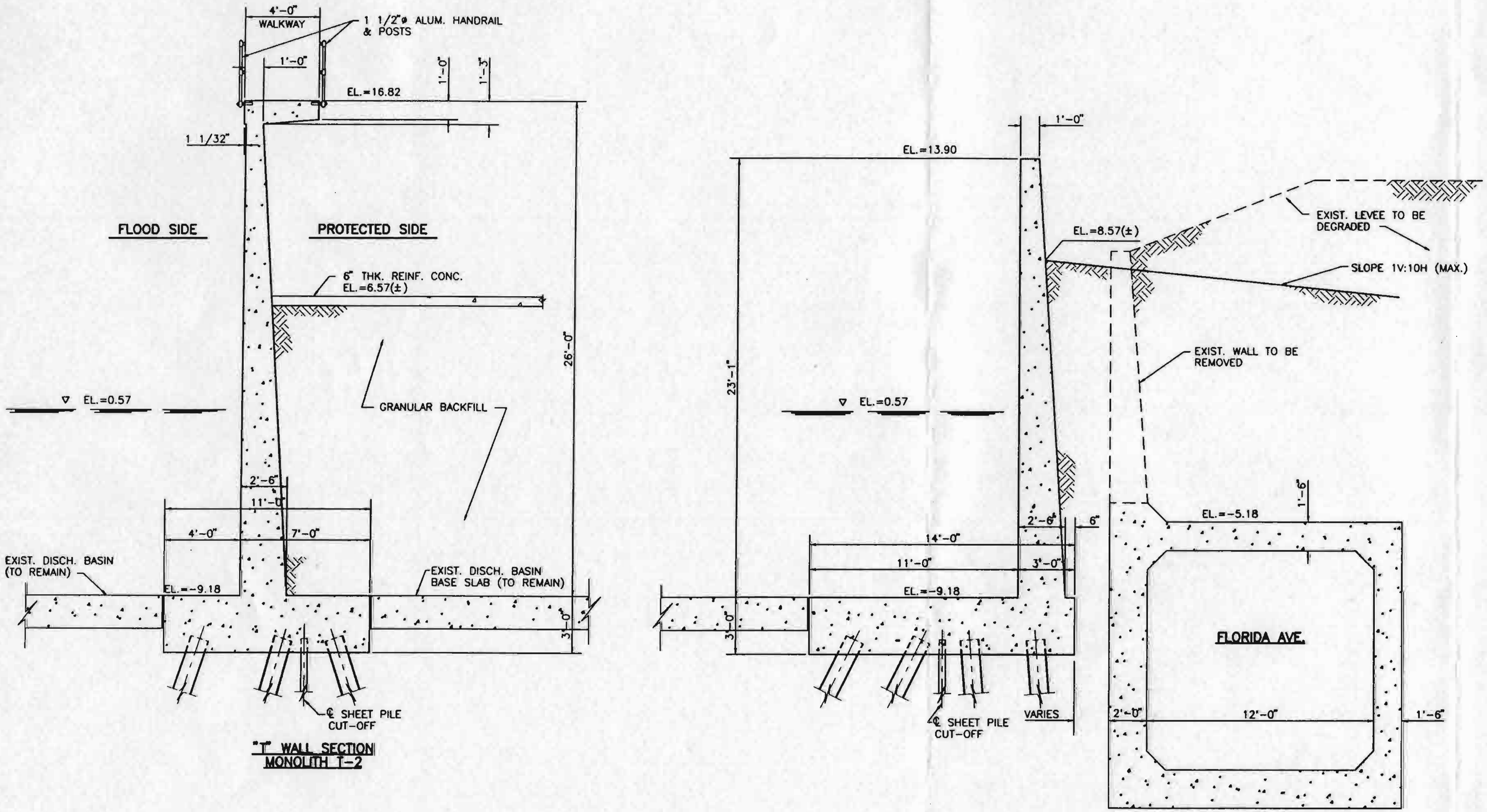
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

LONGITUDINAL SECTION – DISCHARGE TUBE FOR PUMPS "A" & "B"

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

DESIGNED BY: DRAWN BY: CHECKED BY	PLOT SCALE: 48	PLOT DATE: 09/13/95	CADD FILE: 4119B08A.DWG
	DATE: 11/29/94		FILE NO. H-2-40514

PLATE 8



"T" WALL SECTION
MONOLITH T-2

"T" WALL SECTION
MONOLITH T-3

4 0 4 8FT

SCALE: 3/8" = 1'-0"

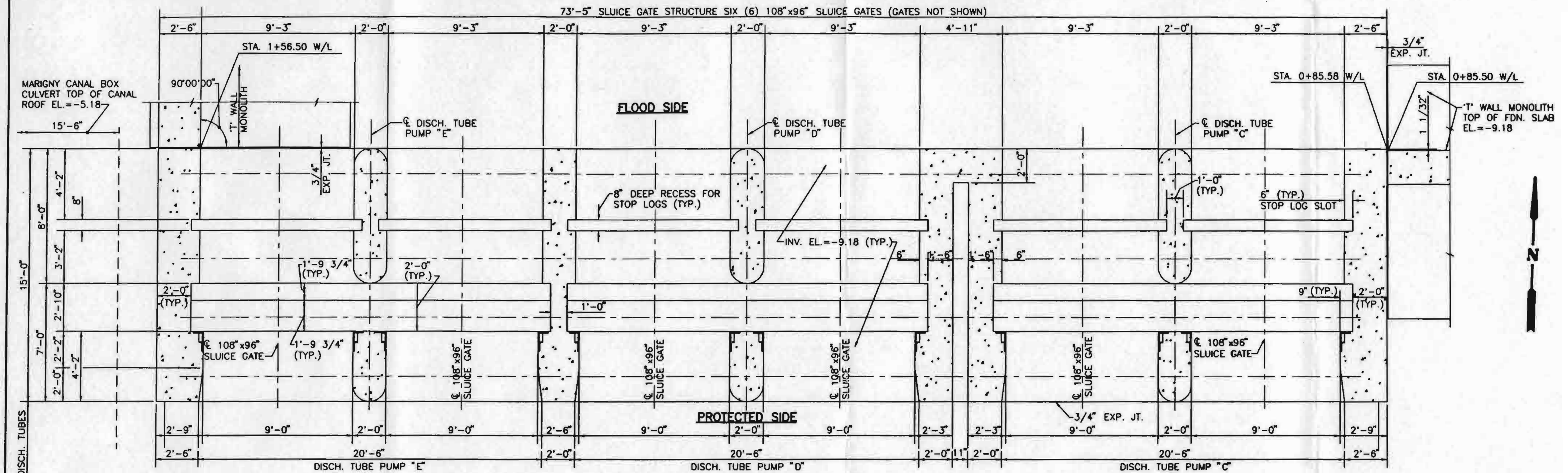
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 20th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

"T" WALL SECTIONS

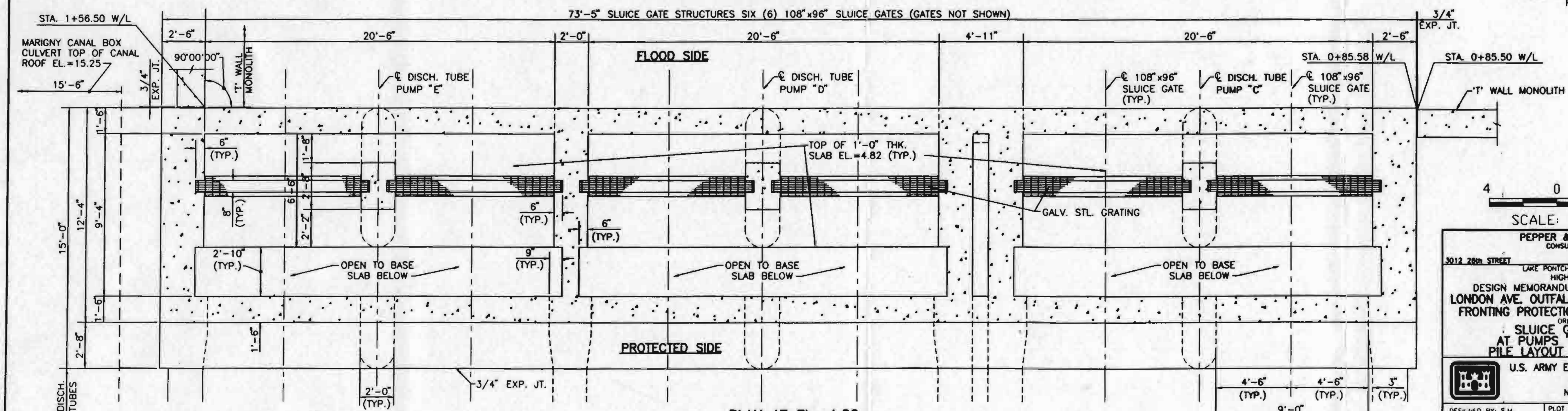
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: S.M.	PLOT SCALE: 32	PLOT DATE: 09/14/95	CADD FILE: 4119805A.DWG
DRAWN BY: K.B.B.	DATE: 11/8/94		FILE NO. H-2-40514
CHECKED BY: S.M.			



BASE SLAB PLAN & PILE LAYOUT

SCALE: 3/8"=1'-0"



PLAN AT EL.=4.82

SCALE: 3/8"=1'-0"

LEGEND

W/L = WALL LINE (FLOOD SIDE FACE OF STRUCTURE)

4 0 4 8FT

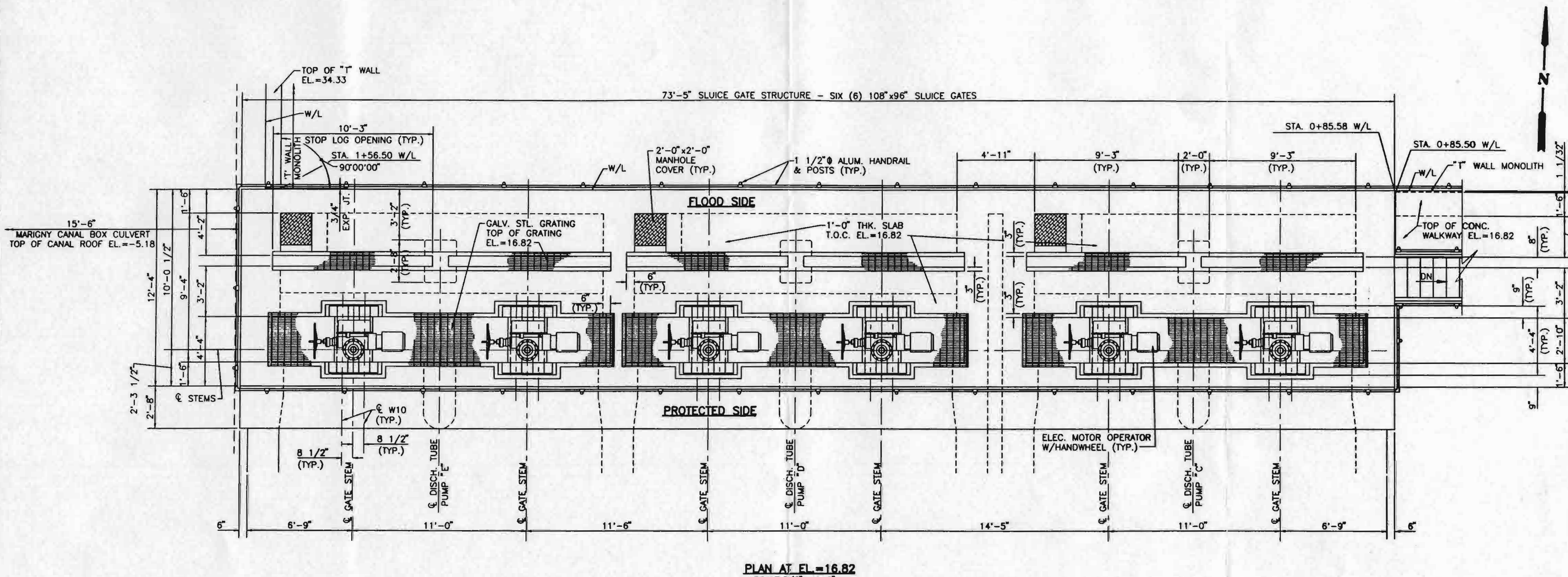
SCALE: 3/8"= 1'-0"

PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 28th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH
SLUICE GATE STRUCTURE
AT PUMPS "C", "D" & "E" DISC.
PILE LAYOUT & PLAN AT EL.=4.82

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

DESIGNED BY: S.M. DRAWN BY: B.L. CHECKED BY: S.M.	PLOT SCALE: 32	PLOT DATE 09/18/95	CADD FILE
	DATE: 10/24/94		FILE NO. H-2-40514



PLAN AT EL.=16.82
SCALE: 3/8"=1'-0"

4 0 4 8FT
SCALE: 3/8"=1'-0"

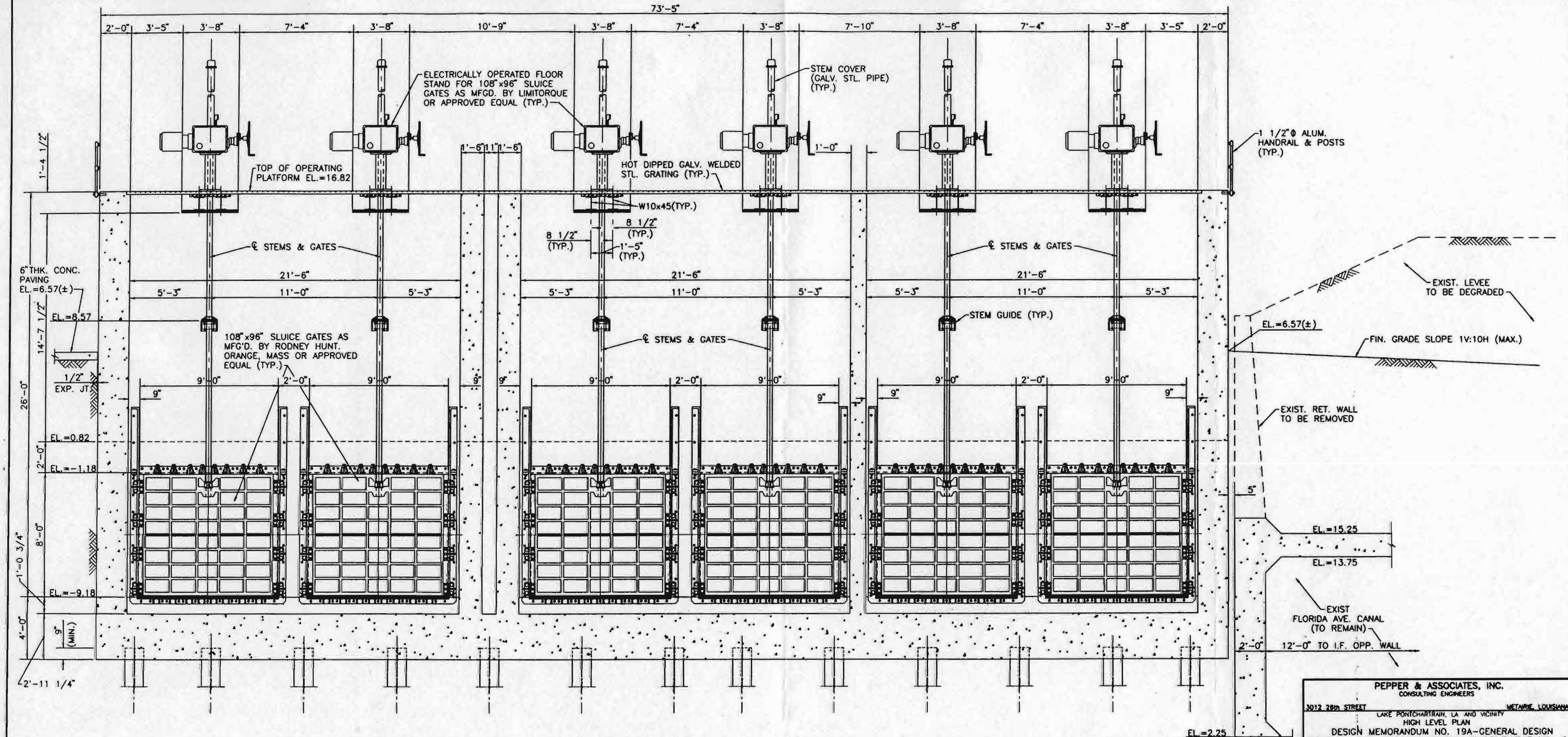
LEGEND
W/L = WALL LINE. FLOOD SIDE FACE OF WALL.

PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH
SLUICE GATE STRUCTURE
AT PUMPS "C", "D" & "E" DISCHARGE
PLAN AT EL. 16.82

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

DESIGNED BY: S.M.
DRAWN BY: B.L.
CHECKED BY: S.M.
PLOT SCALE: 32
PLOT DATE: 09/18/95
DATE: 10/26/94
FILE NO. H-2-40514



SECTION 4/3 12
SCALE: 3/8" = 1'-0"

NOTE: REINFORCING NOT SHOWN

4 0 4 8FT

SCALE: 3/8" = 1'-0"

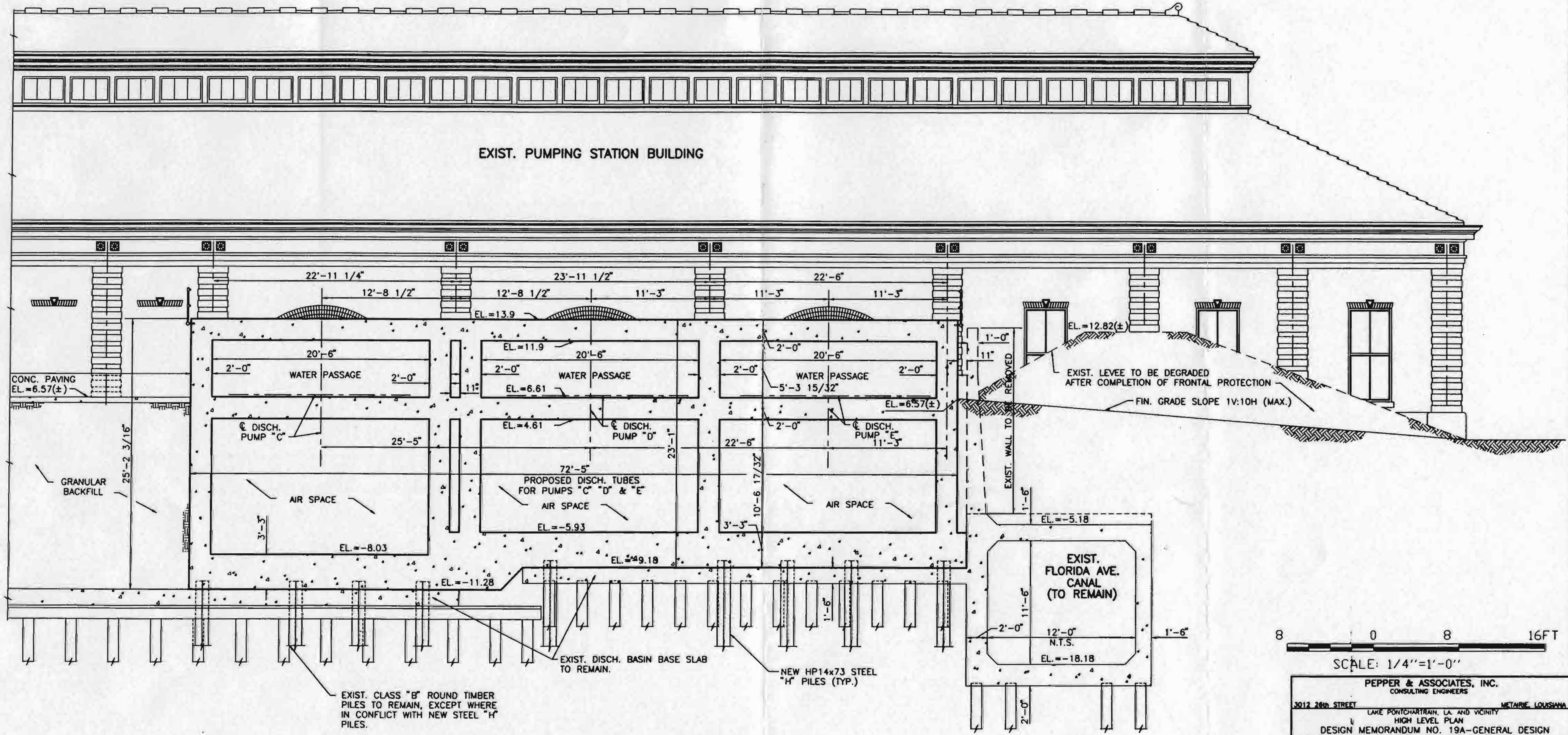
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET — METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

LONGITUDINAL SECTION AT SLUICE GATE STRUCTURE AT PUMPS "C", "D" & "E"

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

DESIGNED BY S.M. DRAWN BY: B.L. CHECKED BY S.M.	PLOT SCALE: 32	PLOT DATE: 09/14/95	CADD FILE. 4119L004.DWG
	DATE: 10/28/94		FILE NO. H-2-40514



SECTION 5/3 13 5/14 13
SCALE: 1/4" = 1'-0"

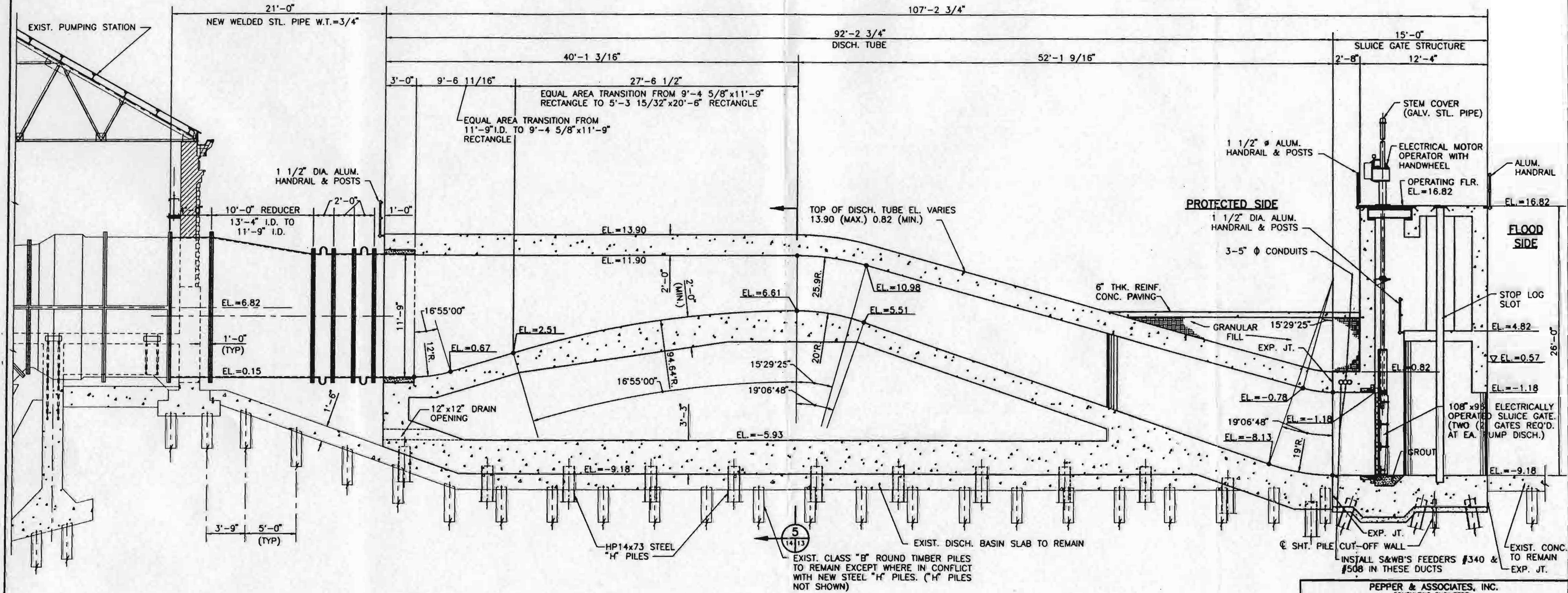
NOTE: REINF. NOT SHOWN

PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH
CROSS SECTION – DISCHARGE TUBES
FOR PUMPS "C" "D" & "E"

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

DESIGNED BY: S.M.	PLOT SCALE: 48	PLOT DATE: 09/18/95	CADD FILE: 41198002.DWG
DRAWN BY: M.E.C.			FILE NO.
CHECKED BY: S.M.	DATE: 11/10/94		H-2-40514



SECTION (6 / 3 | 14)
SCALE: 1/4" = 1'-0"

NOTE:
1.) REINF. NOT SHOWN
2.) SECTION AT PUMP "C" SIMILAR

8 0 8 16FT
SCALE: 1/4"=1'-0"

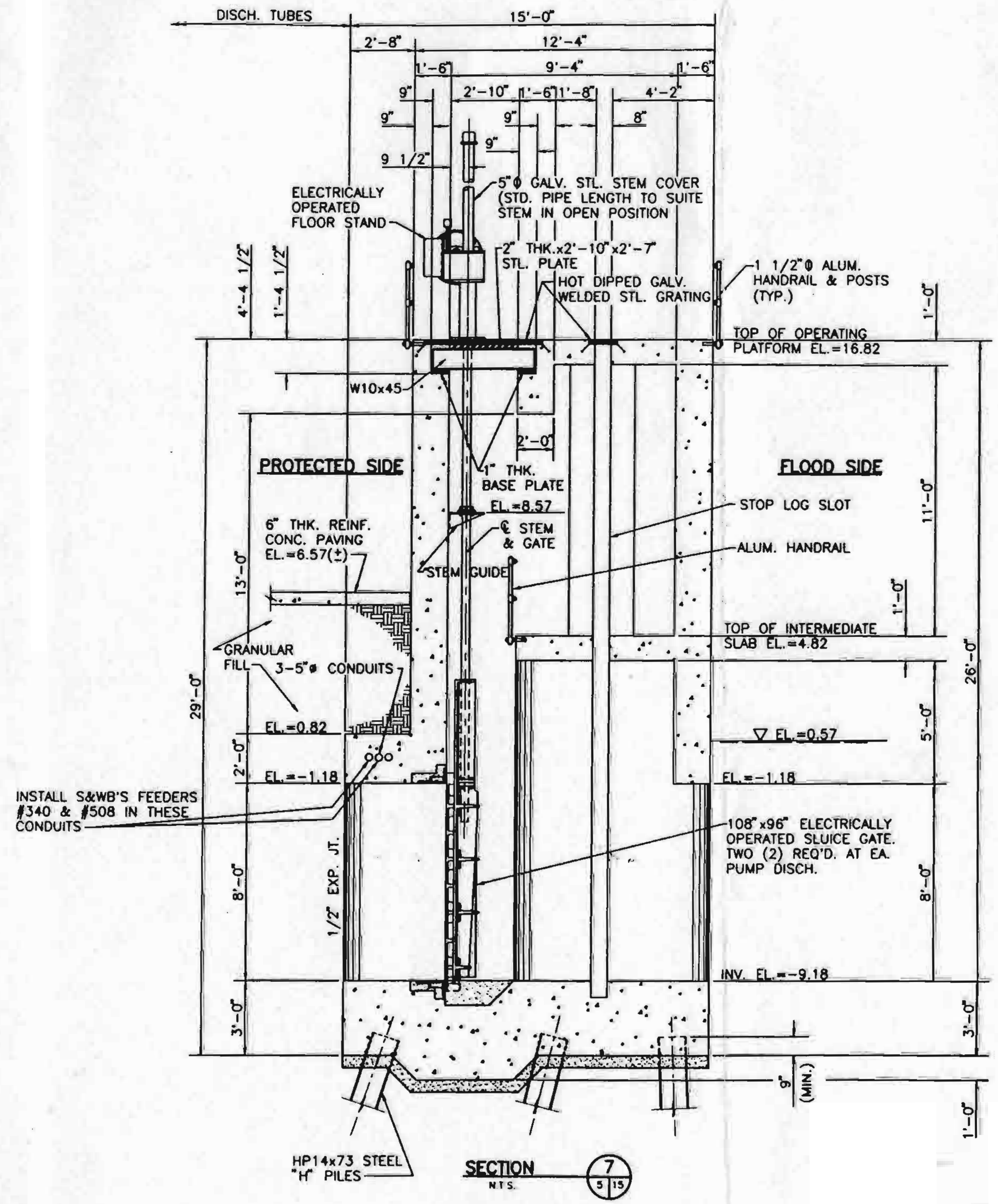
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A–GENERAL DESIGN
LONDON AVE. OUTFALL CANAL–SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

LONGITUDINAL SECTION – DISCHARGE TUBE FOR PUMPS "D" & "E"

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

DESIGNED BY S.M. / DRAWN BY M.E.C. / CHECKED BY S.M.	PLOT SCALE 48	PLOT DATE 09/18/95
	DATE: 10/12/94	FILE NO. H-2-40514



NOTE: REINFORCING NOT SHOWN

4 0 4 8FT

SCALE: 3/8"= 1'-0"

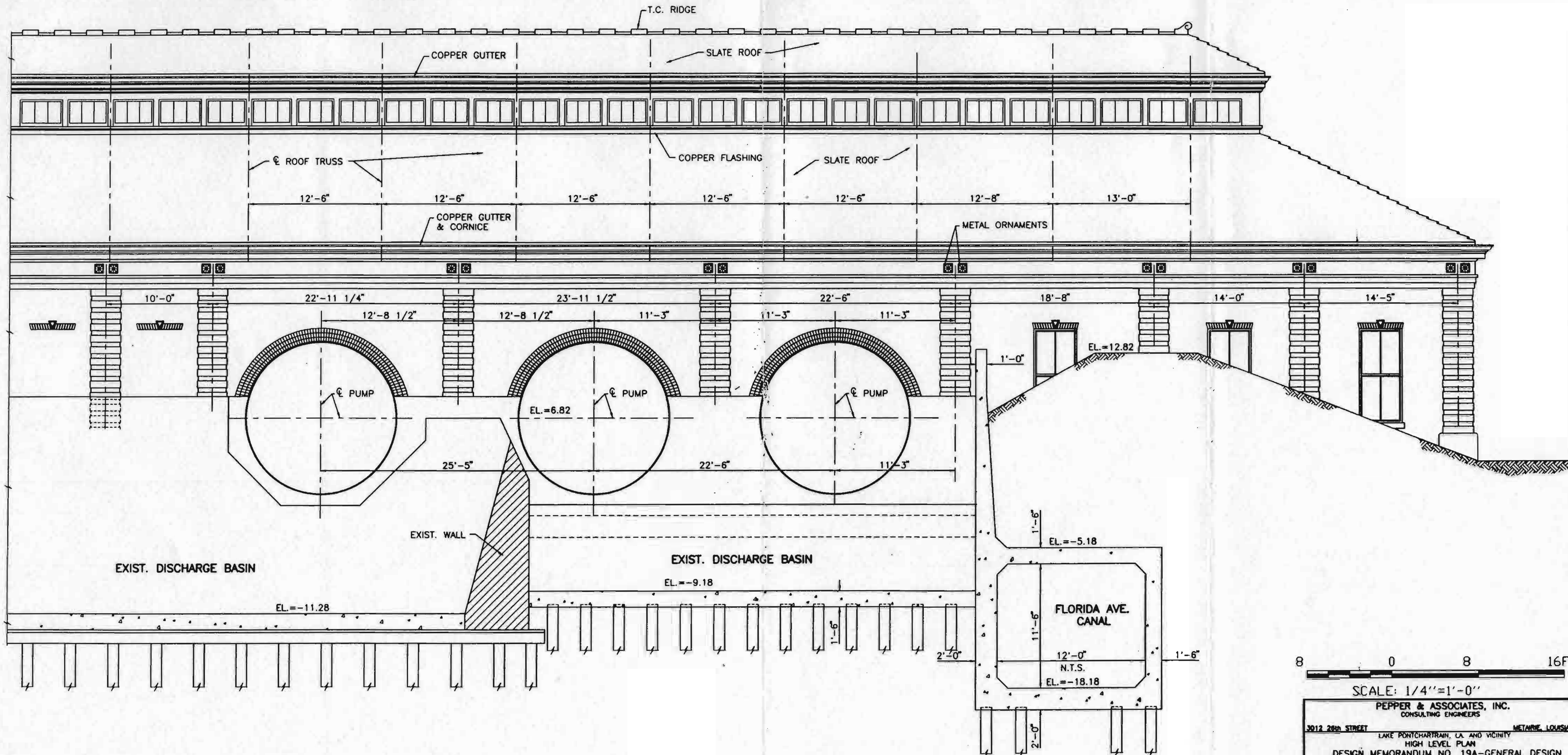
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

CROSS SECTION AT SLUICE GATE STRUCTURE

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

DESIGNED BY: S.M. DRAWN BY: B.L. CHECKED BY: S.M.	PLOT SCALE: 32	PLOT DATE: 09/18/95	CADD FILE: 4118L005.DWG
	DATE: 11/02/94		FILE NO. H-2-40514



PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

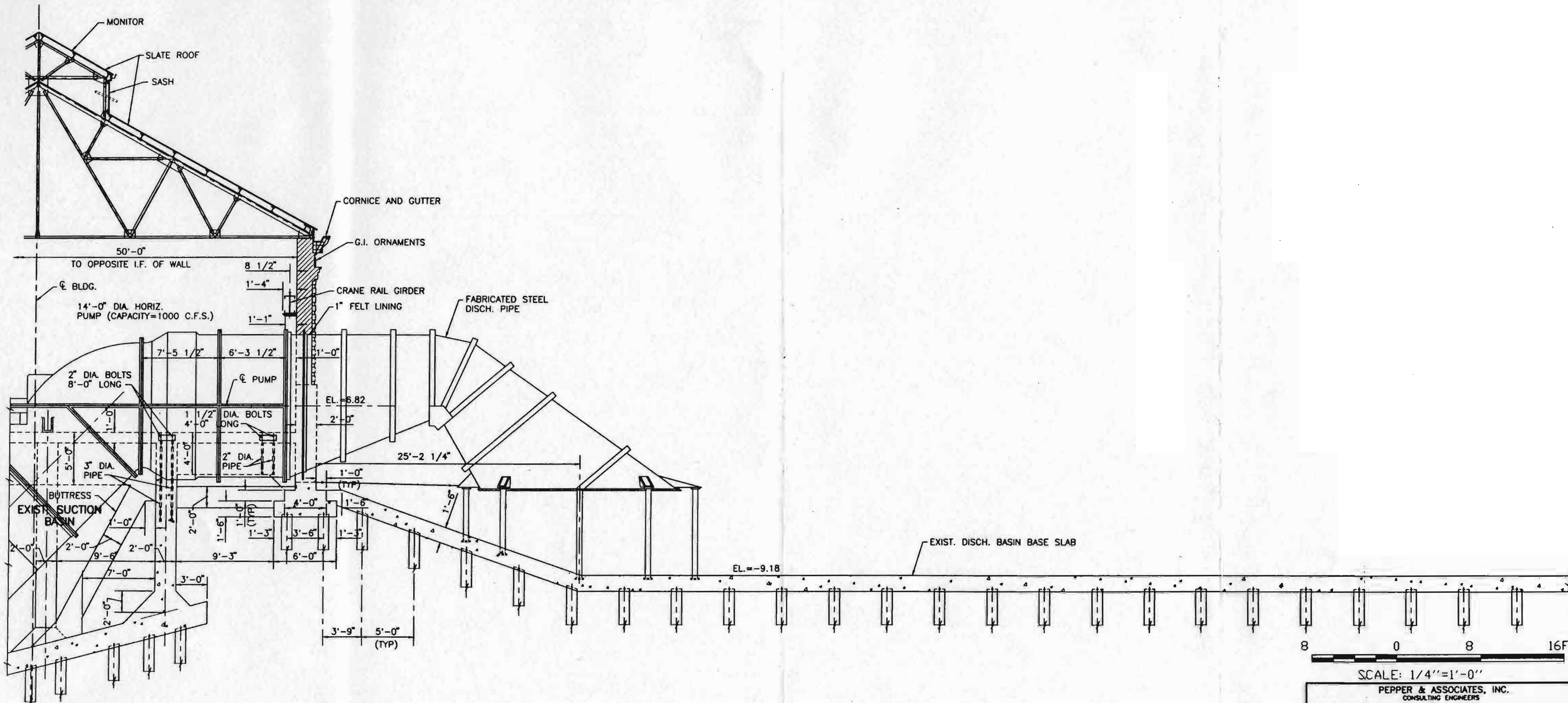
LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A–GENERAL DESIGN
LONDON AVE. OUTFALL CANAL–SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

CROSS SECTION – EXISTING DISCHARGE BASIN AT PUMPS "C" "D" & "E"

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

DESIGNED BY S.M.	PLOT SCALE 48	PLOT DATE 09/18/95	CADD FILE 4119C002.DWG
DRAWN BY M.E.C.			FILE NO. H-2-40514
CHECKED BY S.M.	DATE: 10/17/94		

PLATE 16



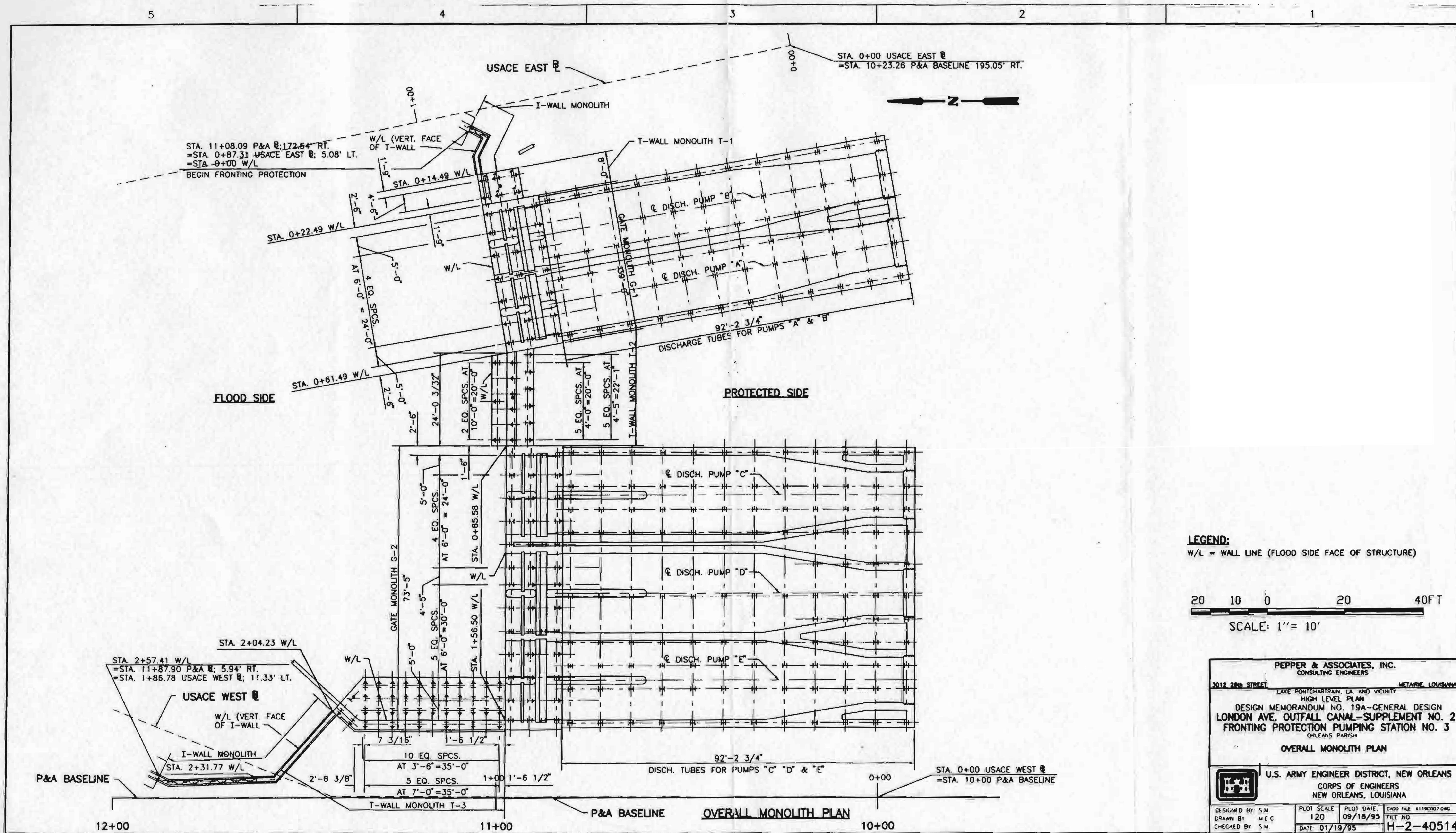
NOTE: REINF. NOT SHOWN.

PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A–GENERAL DESIGN
LONDON AVE. OUTFALL CANAL–SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH
LONGITUDINAL SECTION – EXISTING DISCHARGE BASIN AT PUMPS "C" "D" & "E" DISCHARGE

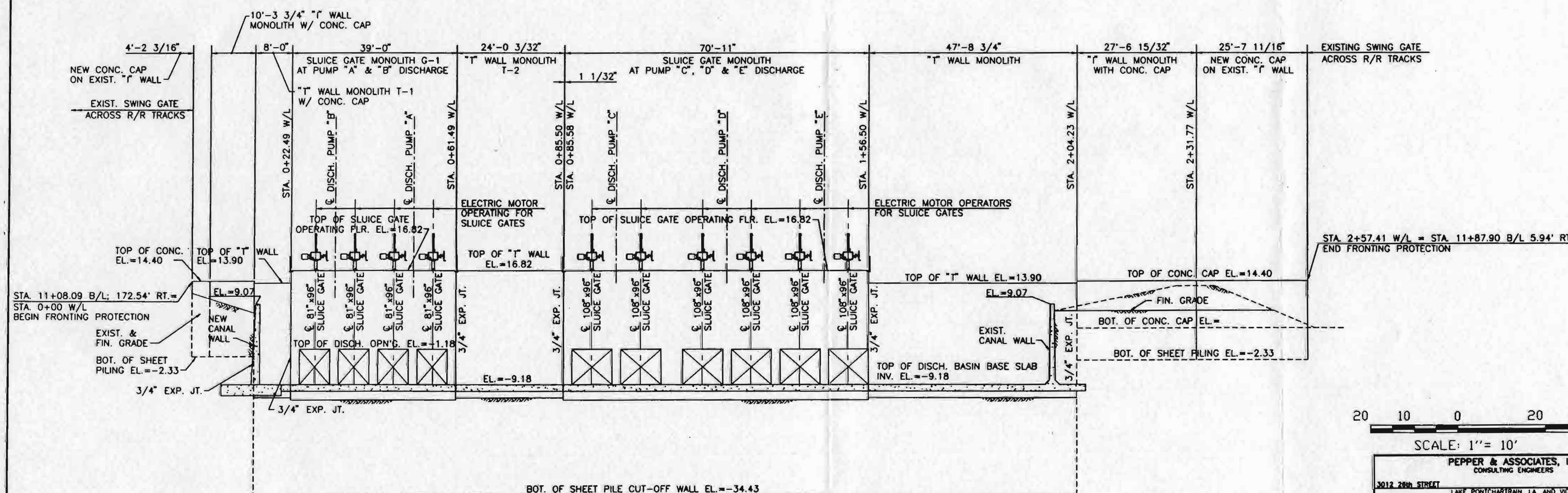
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: S.M. DRAWN BY: M.E.C. CHECKED BY: S.M.	PLOT SCALE: 48	PLOT DATE: 09/14/95	CADD FILE: 4118C001.DWG
	DATE: 10/12/94		FILE NO. H-2-40514



USACE EAST ℄
STA. 0+00 USACE EAST ℄
=STA. 10+23.26 P&A BASELINE 195.05' RT.
I-WALL MONOLITH
STA. 11+08.09 P&A ℄; 172.54' RT.
=STA. 0+87.31 USACE EAST ℄; 5.08' LT.
=STA. 0+00 W/L
BEGIN FRONTING PROTECTION
W/L (VERT. FACE OF T-WALL
T-WALL MONOLITH T-1
STA. 0+14.49 W/L
STA. 0+22.49 W/L
℄ DISCH. PUMP "B"
℄ DISCH. PUMP "A"
GATE MONOLITH G-1
4 EQ. SPCS. AT 6'-0" = 24'-0"
92'-2 3/4"
DISCHARGE TUBES FOR PUMPS "A" & "B"
STA. 0+61.49 W/L
FLOOD SIDE
PROTECTED SIDE
T-WALL MONOLITH T-2
2 EQ. SPCS. AT 10'-0"=20'-0"
5 EQ. SPCS. AT 4'-0"=20'-0"
5 EQ. SPCS. AT 4'-5"=22'-1"
24'-0 3/32"
℄ DISCH. PUMP "C"
℄ DISCH. PUMP "D"
℄ DISCH. PUMP "E"
STA. 0+85.58 W/L
4 EQ. SPCS. AT 6'-0" = 24'-0"
GATE MONOLITH G-2
73'-5"
5 EQ. SPCS. AT 6'-0"=30'-0"
STA. 1+56.50 W/L
STA. 2+04.23 W/L
STA. 2+57.41 W/L
=STA. 11+87.90 P&A ℄; 5.94' RT.
=STA. 1+86.78 USACE WEST ℄; 11.33' LT.
USACE WEST ℄
W/L (VERT. FACE OF I-WALL
I-WALL MONOLITH
STA. 2+31.77 W/L
P&A BASELINE
7 3/16"
1'-6 1/2"
10 EQ. SPCS. AT 3'-6"=35'-0"
5 EQ. SPCS. AT 7'-0"=35'-0"
2'-8 3/8"
T-WALL MONOLITH T-3
92'-2 3/4"
DISCH. TUBES FOR PUMPS "C" "D" & "E"
STA. 0+00 USACE WEST ℄
=STA. 10+00 P&A BASELINE
P&A BASELINE
OVERALL MONOLITH PLAN
12+00
11+00
10+00
LEGEND:
W/L = WALL LINE (FLOOD SIDE FACE OF STRUCTURE)
20 10 0 20 40FT
SCALE: 1"= 10'
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH
OVERALL MONOLITH PLAN
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
DESIGNED BY: S.M.
DRAWN BY: M.E.C.
CHECKED BY: S.M.
PLOT SCALE 120
PLOT DATE: 09/18/95
DATE: 01/19/95
H-2-40514

PLATE 18



PROFILE
N.T.S.

20 10 0 20 40FT
SCALE: 1"= 10'

PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 28th STREET METAIRIE, LOUISIANA

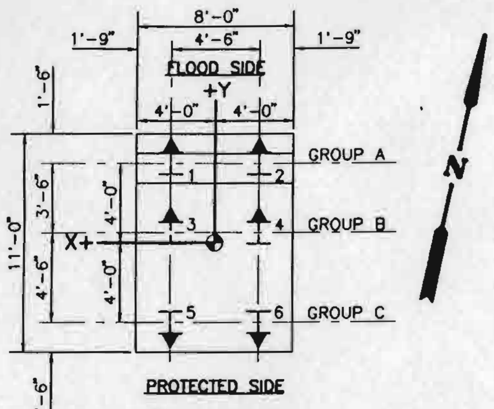
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

FLOOD SIDE PROFILE

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

DESIGNED BY: S.M.	PLOT SCALE: 120	PLOT DATE: 09/14/95	CADD FILE: 41198007.DWG
DRAWN BY: K.B.B.	DATE: 11/23/94		FILE NO. H-2-40514
CHECKED BY: S.M.			

PLATE 19

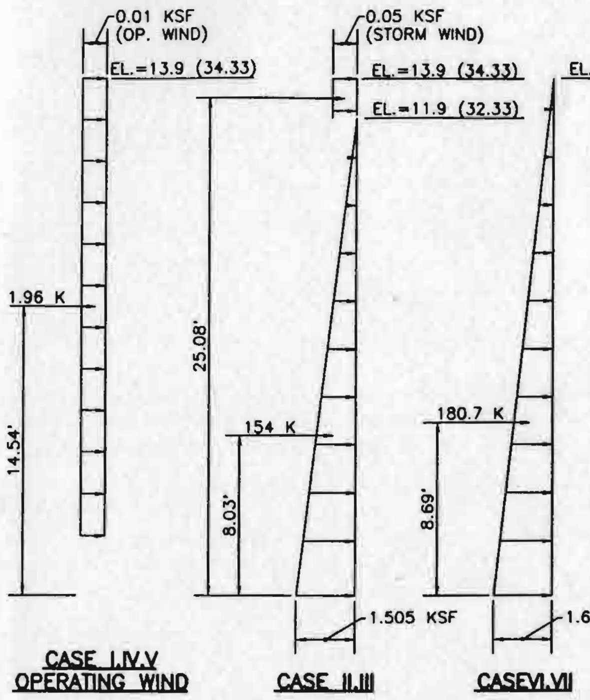


PILE LAYOUT

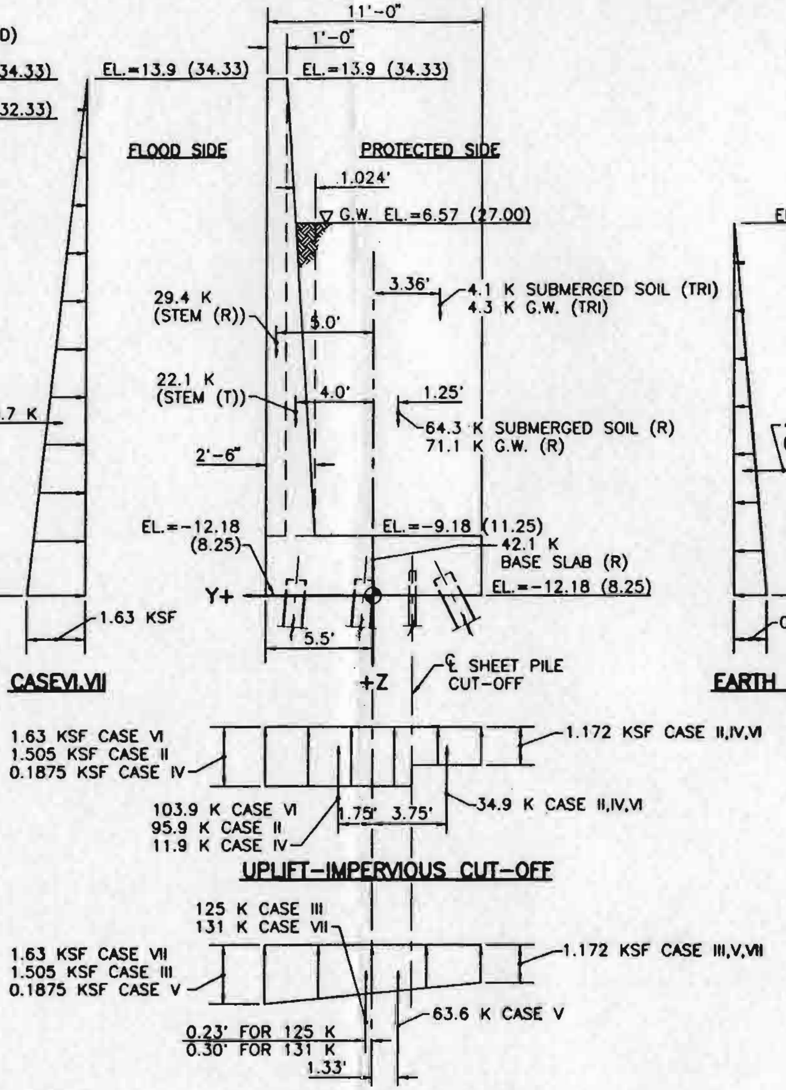
PILE TIP EL.=-80.43 (-60.0)

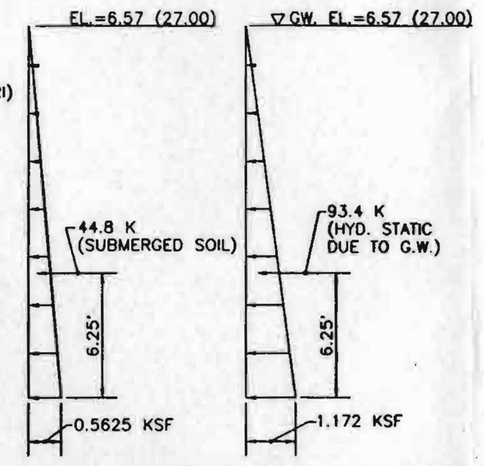
PILE LEGEND

- -12V ON 1H BATTER-GROUP A-HP14x73 (ASTM A36)
- -12V ON 1H BATTER-GROUP B-HP14x73 (ASTM A36)
- -2V ON 1H BATTER-GROUP C-HP14x73 (ASTM A36)



HYD. STATIC PRESSURES





EARTH & HYD. STATIC PRESSURE

CASE II THRU VII

ALL ELEVATIONS REFER TO N.G.V.D.
NUMBERS IN PARENTHESES INDICATE
ELEVATIONS RELATIVE TO CAIRO DATUM.

LEGEND: PRESSURE DIAGRAMS

T OR TRI = TRIANGULAR
R = RECTANGULAR
GW = GROUND WATER
WL = WATER LEVEL

8 0 8 16FT

SCALE: 1/4"=1'-0"

LOAD CASES:

- CASE I - CONSTRUCTION CASE. OP. WIND, NO WATER OR EARTH LOADS
- CASE II - WL IN CANAL AT EL.=11.9 (20.43), STORM WIND, BACKFILL IN PLACE GW AT EL.=6.57 (27.00), IMPERVIOUS CUT-OFF
- CASE III - WL IN CANAL AT EL.=11.9 (20.43), STORM WIND, BACKFILL IN PLACE GW AT EL.=6.57 (27.00), PERVIOUS CUT-OFF
- CASE IV - WL IN CANAL EMPTY, BACKFILL IN PLACE, OP. WIND GW AT EL.=6.57 (27.00), IMPERVIOUS CUT-OFF
- CASE V - WL IN CANAL EMPTY, BACKFILL IN PLACE, OP. WIND GW AT EL.=6.57 (27.00), PERVIOUS CUT-OFF
- CASE VI - WL IN CANAL AT EL.=13.9 (34.33), STORM WIND, BACKFILL IN PLACE GW AT EL.=6.57 (27.00), IMPERVIOUS CUT-OFF
- CASE VII - WL IN CANAL AT EL.=13.9 (34.33), STORM WIND, BACKFILL IN PLACE GW AT EL.=6.57 (27.00), PERVIOUS CUT-OFF

SUMMARY OF PILE ANALYSIS

LOAD CASE	PILE NO.	MAX. COMP. (kips)	% OF ALLOWABLE		PILE NO.	MAX. TENSION (kips)	% OF ALLOWABLE		MAX. PILE CAP MOVEMENT		
			ALF	CBF			ALF	CBF	X (IN)	Y (IN)	Z (IN)
I	1,2	32.1	0.25	0.09					0.5040E-08	-0.4232E-01	0.6723E-02
II	5,6	20.4	0.16	0.06	1,2	17.1	0.17	0.05	-0.5541E-08	0.4735E-01	0.1199E-01
III	5,6	20.8	0.16	0.06	1,2	15.7	0.16	0.05	-0.5246E-08	0.4672E-01	0.1276E-01
IV	3,4	103.4	0.80	0.32	5,6	76.8	0.77	0.25	0.3518E-07	0.4014E+00	0.5493E-01
V	3,4	102.8	0.79	0.31	5,6	77.5	0.78	0.25	0.3536E-07	0.4091E+00	0.5430E-01
VI	5,6	33.7	0.26	0.09	1,2	28.6	0.29	0.08	-0.1193E-07	0.2450E-01	0.8466E-02
VII	5,6	34.1	0.26	0.09	1,2	27.2	0.27	0.07	-0.1163E-07	0.2470E-01	0.9236E-02

MAX ALLOWABLE COMP ON HP14x73 = 130 kips } TIP EL AT -80.43 (-60.0)
MAX ALLOWABLE TENSION ON HP14x73 = 100 kips }

APPLIED LOADS

LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	Mz (kips-ft)
▲ I	0.	-1.96	93.57	206.89	0.	0.
II	0.	-16.66	35.15	-267.15	0.	0.
III	0.	-16.66	40.81	-258.54	0.	0.
* IV	0.	100.40	89.40	719.64	0.	0.
* V	0.	100.40	76.80	684.24	0.	0.
* VI	0.	-32.48	20.40	-461.60	0.	0.
* VII	0.	-38.48	26.23	-452.70	0.	0.

* UNUSUAL LOADING CONDITIONS. APPLIED LOAD REDUCED TO 75% OF ACTUAL LOAD FOR 33% INCREASE IN ALLOWABLE STRESSES FOR UNUSUAL CONDITION.

▲ CONSTRUCTION CONDITION

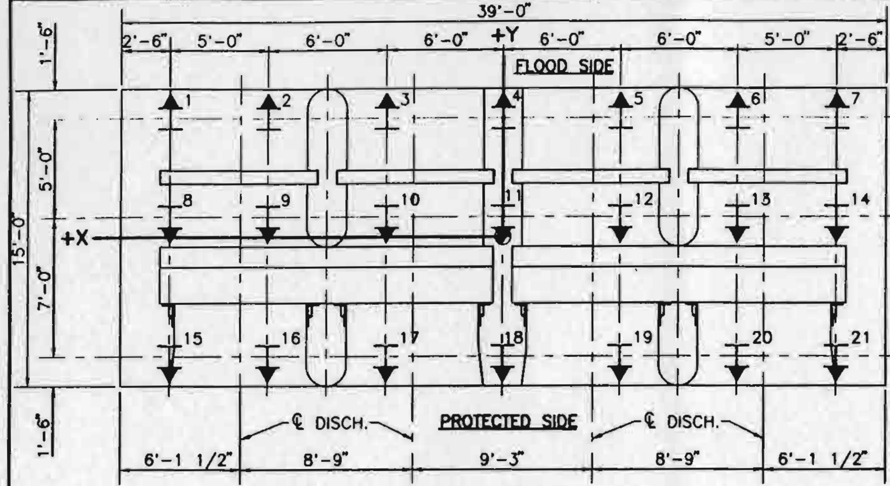
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 28th STREET — METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN: MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

MONOLITH T-1
FOUNDATION DESIGN

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

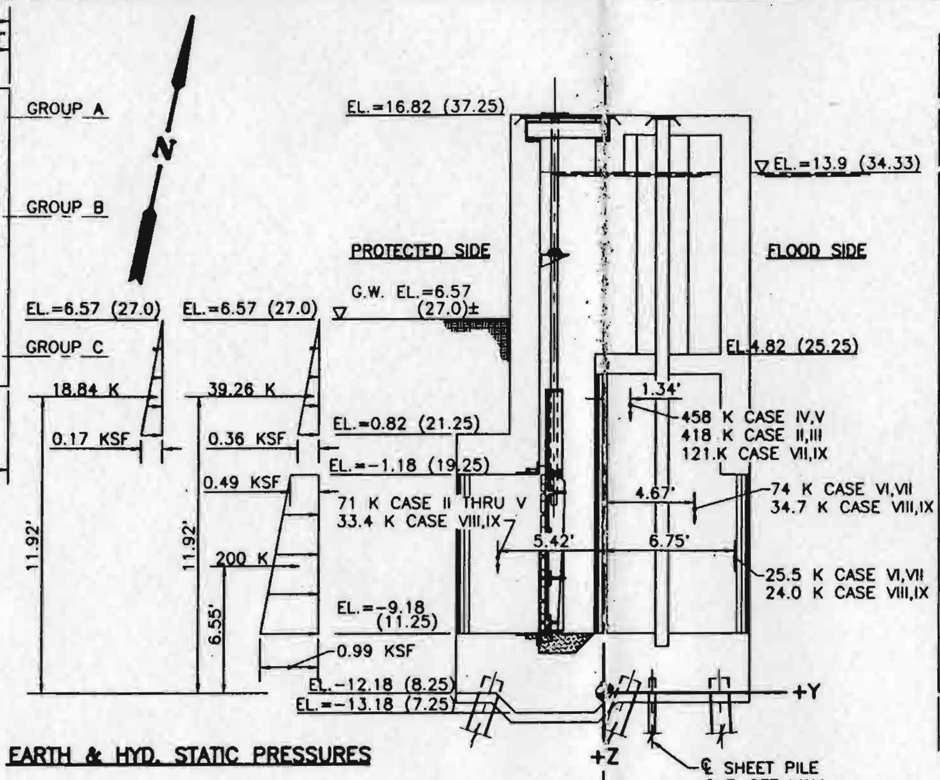
DESIGNED BY S.M. | DRAWN BY B.L. | CHECKED BY S.M.
PLOT SCALE 48 | PLOT DATE: 09/18/95 | DATE: 01/20/95
CADD FILE: 4119L014.DWG | FILE NO. H-2-40514



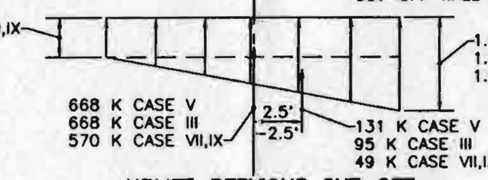
PILE LAYOUT
(PILE TIP ELEV.=-80.43 (-60.0))

PILE LEGEND

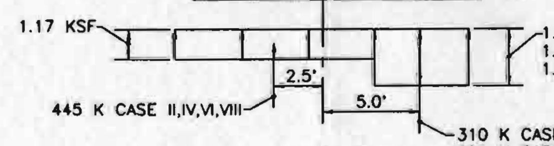
- -24V ON 1H BATTER-GROUP A-HP14x73 (ASTM A36)
- -2.75V ON 1H BATTER-GROUP B-HP14x73 (ASTM A36)
- -3V ON 1H BATTER-GROUP C-HP14x73 (ASTM A36)



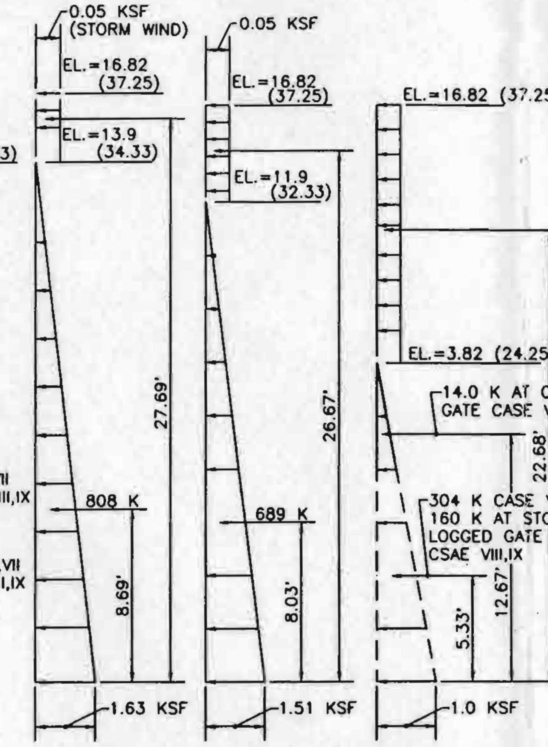
EARTH & HYD. STATIC PRESSURES



UPLIFT-PERVIOUS CUT-OFF



UPLIFT-IMPERVIOUS CUT-OFF

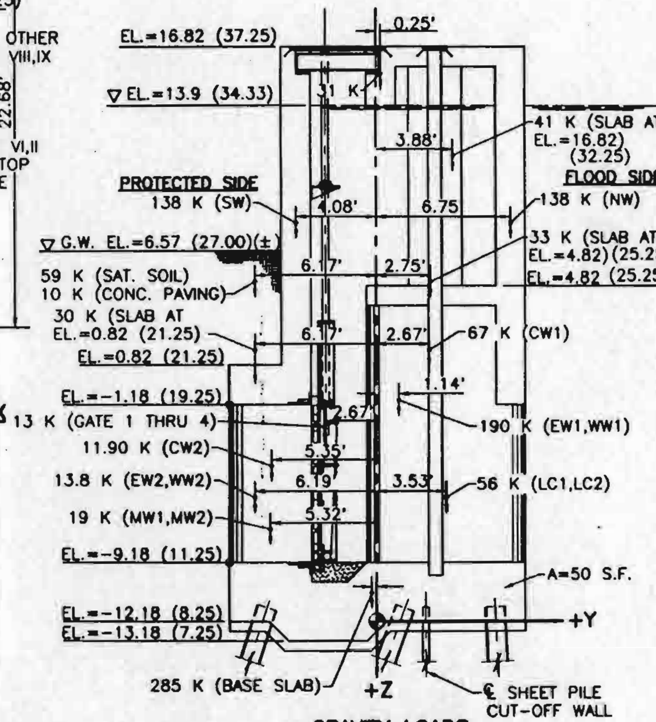


EARTH & HYD. STATIC PRESSURES

LEGEND: PRESSURE DIAGRAMS

T OR TRI	=	TRIANGULAR
R	=	RECTANGULAR
GW	=	GROUND WATER
WL	=	WATER LEVEL

ALL ELEVATIONS REFER TO N.G.V.D. NUMBERS IN PARENTHESES INDICATE ELEVATIONS RELATIVE TO CAIRO DATUM.



GRAVITY LOADS

ALPHA-NUMERIC LABELS ABOVE IDENTIFY STRUCTURAL COMPONENTS OF THE GATE STRUCTURE. REFER TO PRELIMINARY DESIGN CALCULATIONS, PAGES G1-3 & G1-4.

LOAD CASES:

CASE I - CONSTRUCTION CASE OP. WIIND. NO WATER OR EARTH LOADS.

CASE II - WL IN CANAL AT EL.=11.9 (32.33), GATES CLOSED WL IN DISCH. TUBE AT EL.=6.61 (27.04) STORM WIND, BACKFILL IN PLACE, IMPERVIOUS CUT-OFF

CASE III - WL IN CANAL AT EL.=11.9 (32.33), GATES CLOSED WL IN DISCH. TUBE AT EL.=6.61 (27.04) STORM WIND, BACKFILL IN PLACE, PERVIOUS CUT-OFF

CASE IV - WL IN CANAL AT EL.=13.9 (34.33), GATES CLOSED WL IN DISCH. TUBE AT EL.=6.61 (27.04), STORM WIND, BACKFILL IN PLACE, IMPERVIOUS CUT-OFF

CASE V - WL IN CANAL AT EL.=13.9 (34.33), GATES CLOSED WL IN DISCH. TUBE AT EL.=6.61 (27.04) STORM WIND, BACKFILL IN PLACE, PERVIOUS CUT-OFF

CASE VI - STOP LOG IN PLACE AT BOTH PUMPS WL IN CANAL AT EL.=3.82 (24.25), WL IN DISCH. TUBE AT EL.=-9.18 (11.25) OP. WIND, BACKFILL IN PLACE, IMPERVIOUS CUT-OFF

CASE VII - STOP LOG IN PLACE AT BOTH PUMPS WL IN CANAL AT EL.=3.82 (24.25), WL IN DISCH. TUBE AT EL.=-9.18 (11.25) OP. WIND, BACKFILL IN PLACE, PERVIOUS CUT-OFF

CASE VIII - STOP LOG IN PLACE AT PUMP "B" ONLY. WL IN CANAL AT EL.=3.82 (24.25), WL IN DISCH. TUBE AT EL.=-9.18 (11.25) WL IN DISCH. TUBE AT PUMP "A" AT EL.=3.82 (24.25), OP. WIND, BACKFILL IN PLACE, IMPERVIOUS CUT-OFF

CASE IX - STOP LOG IN PLACE AT PUMP "A" ONLY WL IN CANAL AT EL.=3.82 (24.25), WL IN DISCH. TUBE AT EL.=-9.18 (11.25) WL IN DISCH. TUBE AT PUMP "B" AT EL.=3.82 (24.25), OP. WIND, BACKFILL IN PLACE, PERVIOUS CUT-OFF

NOTE: G.W. LEVEL ASSUMED AT EL.=6.57 (27.00) FOR LOAD CASES II THRU VII.

APPLIED LOADS

LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	Mz (kips-ft)
▲ I	0.	-9.88	1105.12	233.13	0.	0.
II	0.	-439.38	964.53	-3820.39	0.	0.
III	0.	-439.38	932.89	-3741.27	0.	0.
IV *	0.	-416.03	735.62	-3963.78	0.	0.
V *	0.	-416.03	702.98	-3882.18	0.	0.
VI	0.	-250.83	681.00	-262.34	0.	0.
VII	0.	-250.83	661.71	-481.29	0.	0.
VIII	0.	-120.90	795.15	116.99	-1079.85	1299.38
IX	0.	-120.90	775.86	-101.95	1079.85	-1299.38

* APPLIED LOAD REDUCED TO 75 % OF ACTUAL LOAD FOR 33 % INCREASE IN ALLOWABLE STRESSES FOR UNUSUAL CONDITION.
▲ CONSTRUCTION CONDITION

SUMMARY OF PILE ANALYSIS

LOAD CASE	PILE NO.	MAX. COMP. (KIPS)	% OF ALLOWABLE ALF	% OF ALLOWABLE CBF	PILE NO.	MAX. TENSION (KIPS)	% OF ALLOWABLE ALF	% OF ALLOWABLE CBF	MAX. PILE CAP MOVEMENT X (IN)	MAX. PILE CAP MOVEMENT Y (IN)	MAX. PILE CAP MOVEMENT Z (IN)
I	1 TO 7	110.6	0.85	0.34		54.2	0.54	0.20	-0.3298E-09	0.4610E+00	0.1411E+00
II	8 TO 14	95.1	0.73	0.27	1 TO 7	27.6	0.28	0.10	-0.3345E-07	-0.2225E+00	-0.1293E-01
III	8 TO 14	98.1	0.75	0.28	1 TO 7	30.5	0.31	0.11	-0.3337E-07	-0.2416E+00	-0.1853E-01
IV	8 TO 14	94.8	0.73	0.28	1 TO 7	45.3	0.45	0.15	-0.3202E-07	-0.2686E+00	-0.3111E-01
V	8 TO 14	97.7	0.75	0.29	1 TO 7	48.4	0.48	0.16	-0.3194E-07	-0.2877E+00	-0.3655E-01
VI	8 TO 14	75.5	0.58	0.22		0.	0.	0.	-0.1699E-07	-0.1852E+00	-0.1707E-01
VII	8 TO 14	73.4	0.56	0.21		0.	0.	0.	-0.1732E-07	-0.1805E+00	-0.1654E-01
VIII	1	119.3	0.92	0.37	7	17.8	0.18	0.09	0.2869E-02	0.1050E+00	0.5115E-01
IX	7	116.5	0.90	0.37	1	20.6	0.21	0.09	-0.2869E-02	0.1097E+00	0.5168E-01

MAX ALLOWABLE COMP ON HP14x73 = 130 kips
MAX ALLOWABLE TENSION ON HP14x73 = 100 kips
} TIP EL AT -80.43 (-60.0)

SCALE: 1/4"=1'-0"

PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 28th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
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MONOLITH G-1
FOUNDATION DESIGN

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

DESIGNED BY S.M. DRAWN BY B.L. CHECKED BY S.M.
PLOT SCALE 48 | PLOT DATE 12/15/94 | CADD FILE 4118L010.DWG
DATE 12/08/94 | FILE NO. H-2-40514

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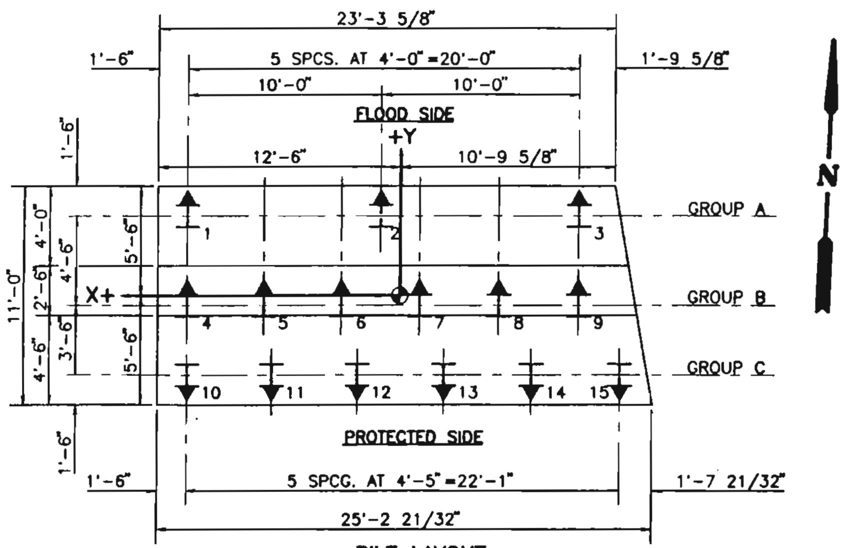


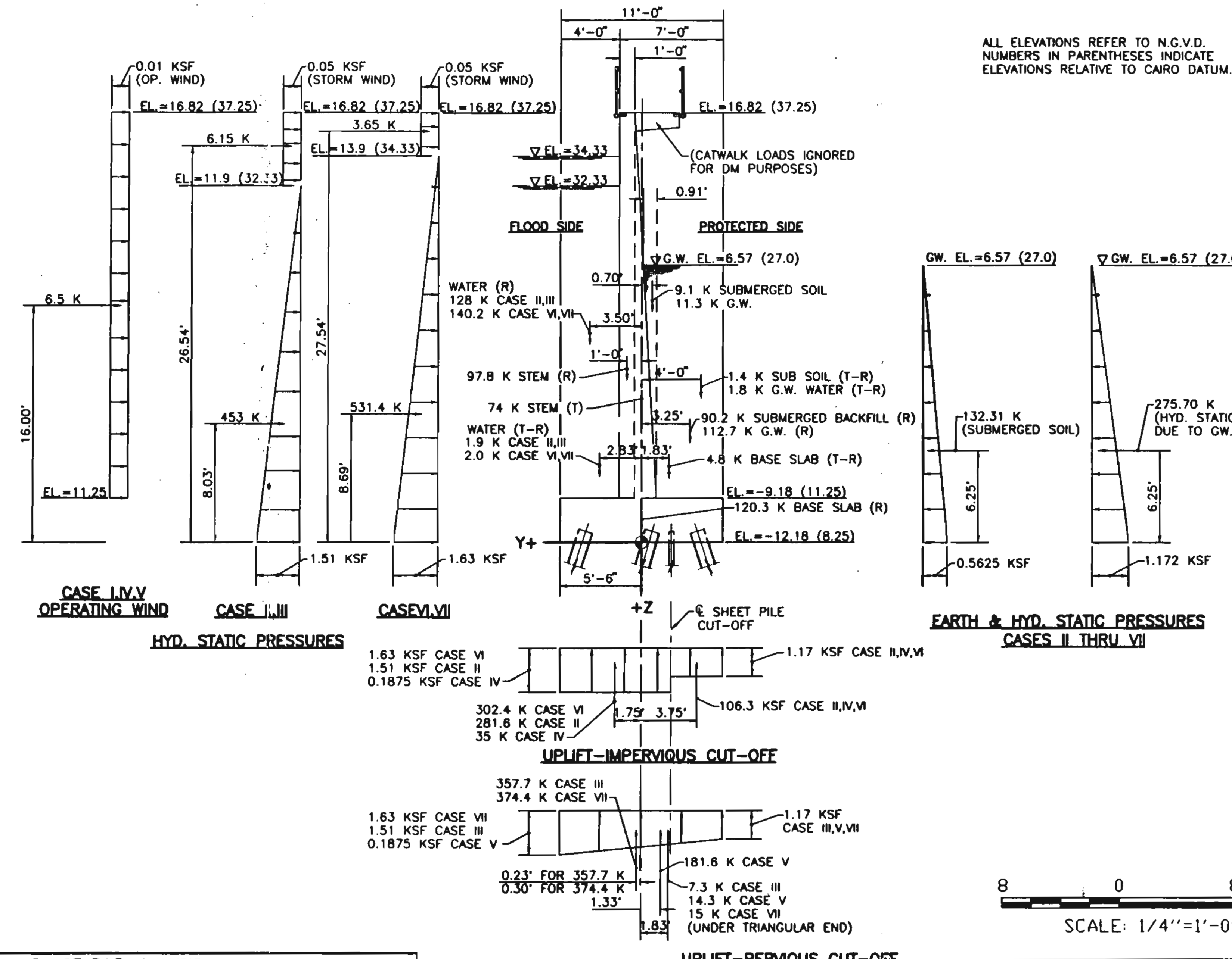
PILE LAYOUT
PILE TIP EL.=-80.43 (-60.0)

PILE LEGEND

- -3V ON 1H BATTER-GROUP A-HP14x73 (ASTM A36)
- -3.5V ON 1H BATTER-GROUP B-HP14x73 (ASTM A36)
- -3V ON 1H BATTER-GROUP C-HP14x73 (ASTM A36)

LEGEND: PRESSURE DIAGRAMS

T OR TRI = TRIANGULAR
R = RECTANGULAR
GW = GROUND WATER
WL = WATER LEVEL



ALL ELEVATIONS REFER TO N.G.V.D. NUMBERS IN PARENTHESES INDICATE ELEVATIONS RELATIVE TO CAIRO DATUM.

8 0 8 16FT
SCALE: 1/4"=1'-0"

LOAD CASES:

CASE I – CONSTRUCTION CASE. OP. WIND, NO WATER OR EARTH LOADS
CASE II – WL IN CANAL AT EL.=11.9 (32.33), STORM WIND, BACKFILL IN PLACE GW AT EL.=6.57 (27.0), IMPERIOUS CUT-OFF
CASE III – WL IN CANAL AT EL.=11.9 (32.33), STORM WIND, BACKFILL IN PLACE GW AT EL.=6.57 (27.0), PERVIOUS CUT-OFF
CASE IV – WL IN CANAL EMPTY, BACKFILL IN PLACE, OP. WIND GW AT EL.=6.57 (27.0), IMPERVIOUS CUT-OFF
CASE V – WL IN CANAL EMPTY, BACKFILL IN PLACE, OP. WIND GW AT EL.=6.57 (27.0), PERVIOUS CUT-OFF
CASE VI – WL IN CANAL AT EL.=13.9 (34.33), STORM WIND, BACKFILL IN PLACE GW AT EL.=6.57 (27.0), IMPERVIOUS CUT-OFF
CASE VII – WL IN CANAL AT EL.=13.9 (34.33), STORM WIND, BACKFILL IN PLACE GW AT EL.=6.57 (27.0), PERVIOUS CUT-OFF

SUMMARY OF PILE ANALYSIS

LOAD CASE	PILE NO.	MAX. COMP. (KIPS)	% OF ALLOWABLE		PILE NO.	MAX. TENSION (KIPS)	% OF ALLOWABLE		MAX. PILE CAP MOVEMENT		
			ALF	CBF			ALF	CBF	X (IN)	Y (IN)	Z (IN)
I	2	56.7	0.44	0.15					0.3964E-03	-0.4161E-01	0.2101E-01
II	10	33.7	0.26	0.09	2	42.9	0.43	0.12	0.3892E-03	0.2146E-01	-0.5285E-03
III	10	34.5	0.28	0.09	2	36.4	0.36	0.10	0.4045E-03	0.1607E-01	0.1150E-02
IV	9	108.0	0.83	0.31	10	92.7	0.93	0.27	-0.2660E-03	0.2439E+00	0.1980E-02
V	9	109.1	0.84	0.31	10	94.9	0.95	0.28	-0.3080E-03	0.2615E+00	-0.3247E-02
VI	10	45.7	0.35	0.12	2	61.7	0.62	0.16	0.4362E-03	0.1319E-01	-0.4618E-02
VII	10	46.5	0.36	0.12	2	55.7	0.56	0.15	0.4506E-03	0.8270E-02	-0.3071E-02

MAX ALLOWABLE COMP ON HP14x73 = 130 kips
MAX ALLOWABLE TENSION ON HP14x73 = 100 kips } TIP EL AT -80.43 (-60.0)

APPLIED LOADS

LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	Mz (kips-ft)
▲ I	0.	-6.5	296.93	-14.966	0.	0.
II	0.	-52.82	139.43	-1119.36	0.	0.
III	0.	-52.82	155.49	-1080.58	0.	0.
* IV	0.	394.37	256.13	2128.38	0.	0.
* V	0.	394.37	208.67	1994.73	0.	0.
* VI	0.	-98.60	98.00	-1575.00	0.	0.
* VII	0.	-98.60	113.00	-1540.00	0.	0.

* UNUSUAL LOADING CONDITION
▲ CONSTRUCTION CONDITION

PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 28th STREET METAIRIE, LOUISIANA

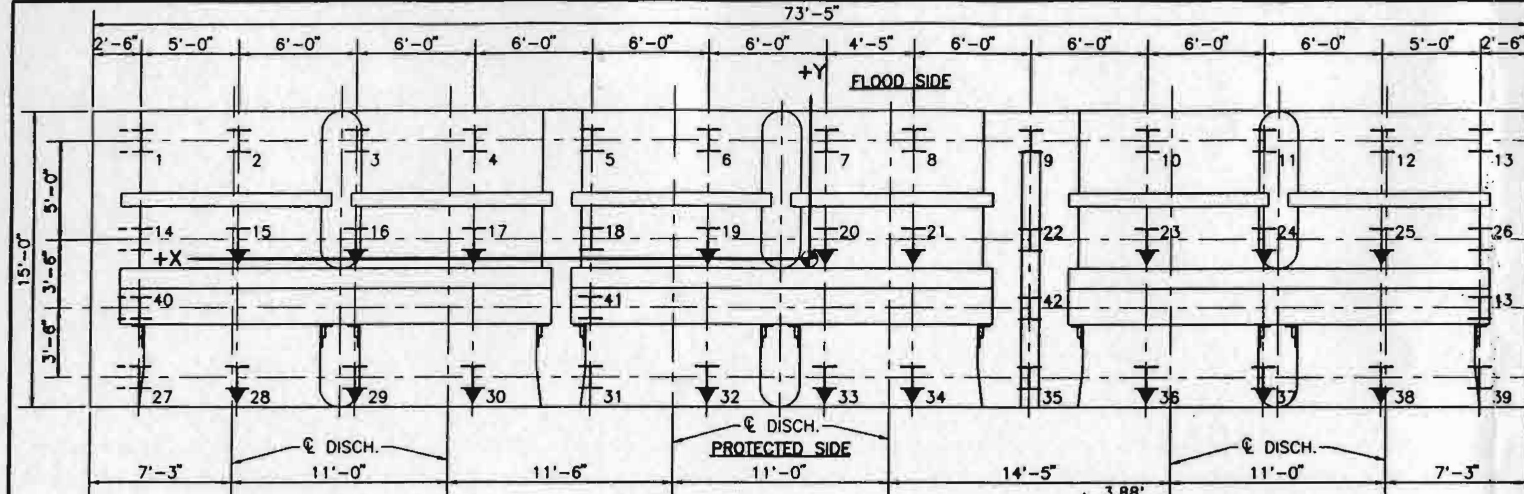
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

MONOLITH T-2 FOUNDATION DESIGN

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

DESIGNED BY: S.M. DRAWN BY: B.L. CHECKED BY: S.M.
PLOT SCALE 48 | PLOT DATE 09/18/95 | CADD FILE 4119L011.DWG
DATE: 12/19/94 | FILE NO. H-2-40514

5 | 4 | 3 | 2 | 1



PILE LAYOUT

PILE TIP EL.=-80.43 (-60.0)

PILE LEGEND

- VERTICAL-GROUP A-HP14x73 (ASTM A36)
- -3V ON 1H BATTER-(EXCEPT PILES 14, 18, 22, 26 VERTICAL GROUP B-HP14x73 (ASTM A36)
- VERTICAL-GROUP C-HP14x73 (ASTM A36)
- -3V ON 1H BATTER-(EXCEPT PILES 27, 31, 35, 39 VERTICAL GROUP D-HP14x73 (ASTM A36)

LOAD CASES:

CASE I - CONSTRUCTION CASE OP. WIND. NO WATER OR EARTH LOADS.

CASE II - WL IN CANAL AT EL.=11.9 (32.33), GATES CLOSED WL IN DISCH. TUBE AT EL.=6.61 (27.04) STORM WIND, BACKFILL IN PLACE, IMPERVIOUS CUT-OFF

CASE III - WL IN CANAL AT EL.=11.9 (32.33), GATES CLOSED WL IN DISCH. TUBE AT EL.=6.61 (27.04) STORM WIND, BACKFILL IN PLACE, PERVIOUS CUT-OFF

CASE IV - WL IN CANAL AT EL.=13.9 (34.33), GATES CLOSED WL IN DISCH. TUBE AT EL.=6.61 (27.04), STORM WIND, BACKFILL IN PLACE, PERVIOUS CUT-OFF

CASE V - WL IN CANAL AT EL.=13.9 (34.33), GATES CLOSED WL IN DISCH. TUBE AT EL.=6.61 (27.04) STORM WIND, BACKFILL IN PLACE, PERVIOUS CUT-OFF

CASE VI - STOP LOG IN PLACE AT ALL THREE PUMPS WL IN CANAL AT EL.=3.82 (24.25), WL IN DISCH. TUBES AT EL.=-9.18 (11.25) OP. WIND, BACKFILL IN PLACE, IMPERVIOUS CUT-OFF

CASE VII - STOP LOG IN PLACE AT ALL THREE PUMPS WL IN CANAL AT EL.=3.82 (24.25), WL IN DISCH. TUBES AT EL.=-9.18 (11.25) OP. WIND, BACKFILL IN PLACE, PERVIOUS CUT-OFF

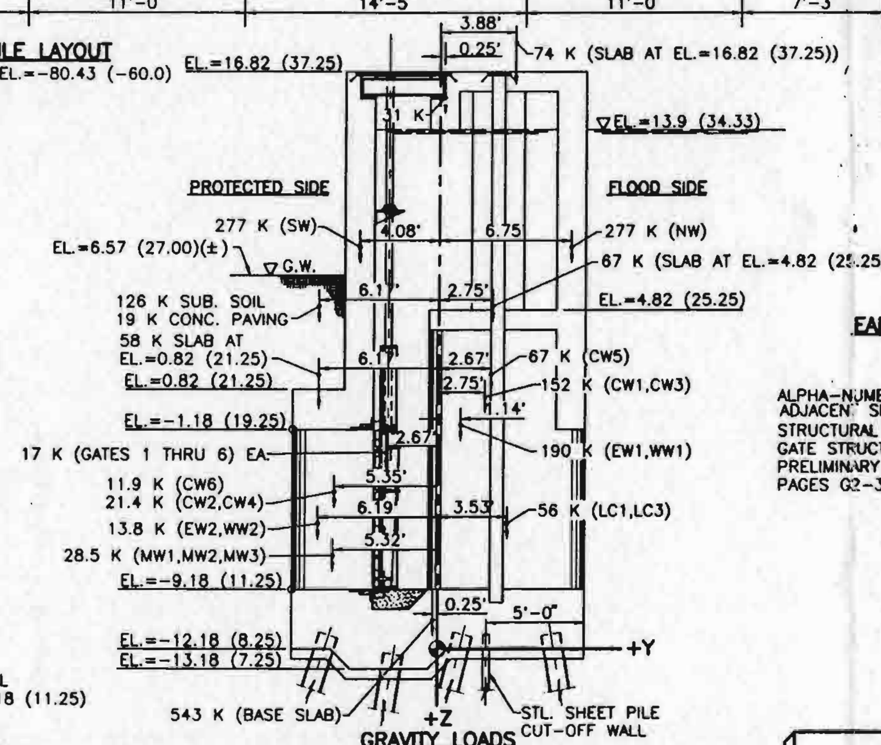
CASE VIII - STOP LOG IN PLACE AT PUMP "C" ONLY WL IN CANAL AT EL.=3.82 (24.25), WL IN DISCH. TUBE AT EL.=-9.18 (11.25), WL IN DISCH. TUBE AT PUMPS "D" & "E" AT EL.=3.82 (24.25) OP. WIND, BACKFILL IN PLACE, IMPERVIOUS CUT-OFF

CASE IX - STOP LOG IN PLACE AT PUMP "C" ONLY WL IN CANAL AT EL.=3.82 (24.25), WL IN DISCH. TUBE AT EL.=-9.18 (11.25), WL IN DISCH. TUBE AT PUMPS "D" & "E" AT EL.=3.82 (24.25) OP. WIND, BACKFILL IN PLACE, PERVIOUS CUT-OFF

CASE X - STOP LOG IN PLACE AT PUMP "D" ONLY WL IN CANAL AT EL.=3.82 (24.25), WL IN DISCH. TUBE AT EL.=-9.18 (11.25), WL IN DISCH. TUBE AT PUMPS "C" & "E" AT EL.=3.82 (24.25) OP. WIND, BACKFILL IN PLACE, IMPERVIOUS CUT-OFF

CASE XI - STOP LOG IN PLACE AT PUMP "D" ONLY WL IN CANAL AT EL.=3.82 (24.25), WL IN DISCH. TUBE AT EL.=-9.18 (11.25), WL IN DISCH. TUBE AT PUMPS "C" & "E" AT EL.=3.82 (24.25) OP. WIND, BACKFILL IN PLACE, PERVIOUS CUT-OFF

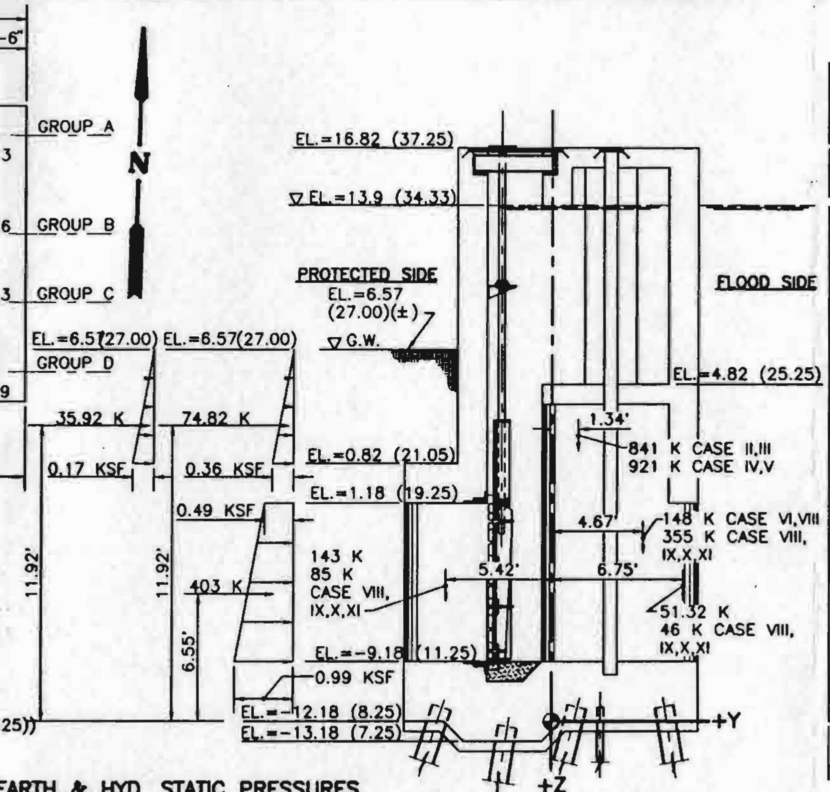
NOTE: G.W. LEVEL ASSUMED AT EL.=6.57 (27.00) FOR LOAD CASES II THRU XI.



GRAVITY LOADS

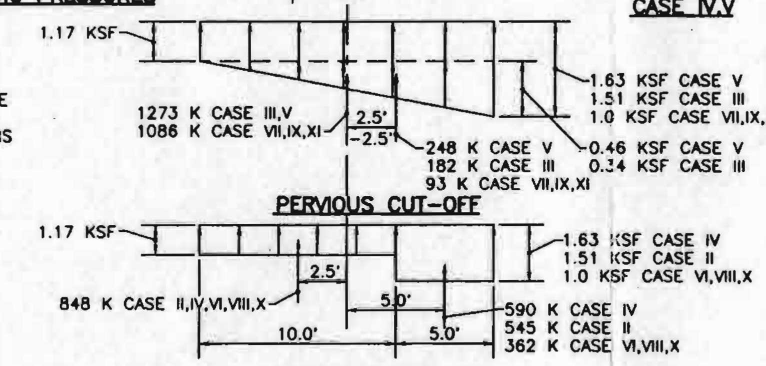
APPLIED LOADS						
LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	Mz (kips-ft)
▲ I	.0	-18.83	2027.74	795.80	-797.20	0.
II	-51.43	-816.00	1814.51	-6752.62	-279.32	0.
III	-51.43	-816.00	1754.20	-6601.84	-279.32	0.
* IV	-38	-777	1387	-7153	-209	0.
* V	-38	-777	1325	-6998	-209	0.
VI	-51.43	-478.03	1162.02	-432.94	-279.32	0.
VII	-51.43	-478.03	1193.15	-510.73	-279.32	0.
VIII	-51.43	-101.67	1449.30	721.56	-4274.39	3490.69
IX	-51.43	-101.67	1480.42	643.77	-4274.39	3490.69
X	-51.43	-102.46	1449.30	711.67	-366.61	-858.54
XI	-51.43	-102.46	1480.42	633.87	-366.61	-858.542

* APPLIED LOAD REDUCED TO 75 % OF ACTUAL LOAD FOR 33 % INCREASE IN ALLOWABLE STRESSES FOR UNUSUAL CONDITION.
▲ CONSTRUCTION CONDITION



EARTH & HYD. STATIC PRESSURES

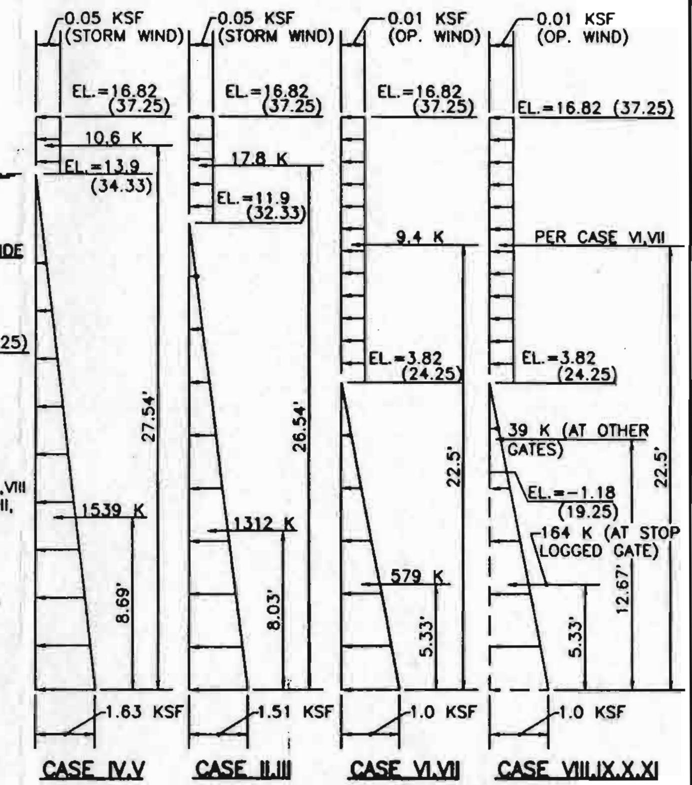
ALPHA-NUMERIC LABELS IN ADJACENT SKETCH IDENTIFY STRUCTURAL COMPONENTS OF THE GATE STRUCTURE. REFER TO PRELIMINARY DESIGN CALCULATIONS PAGES G2-3 & G2-4.



IMPERVIOUS CUT-OFF

SUMMARY OF PILE ANALYSIS											
LOAD CASE	PILE NO.	MAX. COMP. (KIPS)	% OF ALLOWABLE		PILE NO.	MAX. TENSION (KIPS)	% OF ALLOWABLE		MAX. PILE CAP MOVEMENT		
			ALF	CBF			ALF	CBF	X (IN)	Y (IN)	Z (IN)
I	27	100	0.77	0.28	13				0.7142E-05	0.1628E+00	0.5638E-01
II	28	133	1.03	0.48	13	19	0.19	0.18	-0.1663E+00	-0.3202E+00	-0.1065E-01
III	30	131	1.01	0.48	13	20	0.20	0.19	-0.1663E+00	-0.3272E+00	-0.1264E-01
IV	32	126	0.97	0.44	13	32	0.32	0.19	-0.1239E+00	-0.3282E+00	-0.1899E-01
V	28	125	0.96	0.44	13	33	0.33	0.20	-0.1239E+00	-0.3354E+00	-0.2104E-01
VI	15	89	0.69	0.35	39	37	0.37	0.22	-0.1663E+00	-0.2153E+00	-0.8719E-02
VII	15	89	0.69	0.35	39	35	0.35	0.21	-0.1663E+00	-0.2117E+00	-0.7693E-02
VIII	27	80	0.62	0.31	15	21	0.21	0.18	-0.1659E+00	0.5478E-01	0.3204E-01
IX	27	83	0.62	0.32	15	21	0.21	0.18	-0.1659E+00	0.5841E-01	0.3306E-01
X	39	54	0.41	0.25		0.	0.	0.	-0.1664E+00	0.5455E-01	0.3204E-01
XI	39	56	0.43	0.26		0.	0.	0.	-0.1664E+00	0.5818E-01	0.3306E-01

MAX ALLOWABLE COMP ON HP14x73 = 130 kips } TIP EL AT -80.43 (-60.0)
MAX ALLOWABLE TENSION ON HP14x73 = 100 kips }



EARTH & HYD. STATIC PRESSURES

ALL ELEVATIONS REFER TO N.G.V.D. NUMBERS IN PARENTHESES INDICATE ELEVATIONS RELATIVE TO CAIRO DATUM.

LEGEND: PRESSURE DIAGRAMS

- T OR TRI = TRIANGULAR
- R = RECTANGULAR
- GW = GROUND WATER
- WL = WATER LEVEL

8 0 8 16FT

SCALE: 1/4"=1'-0"

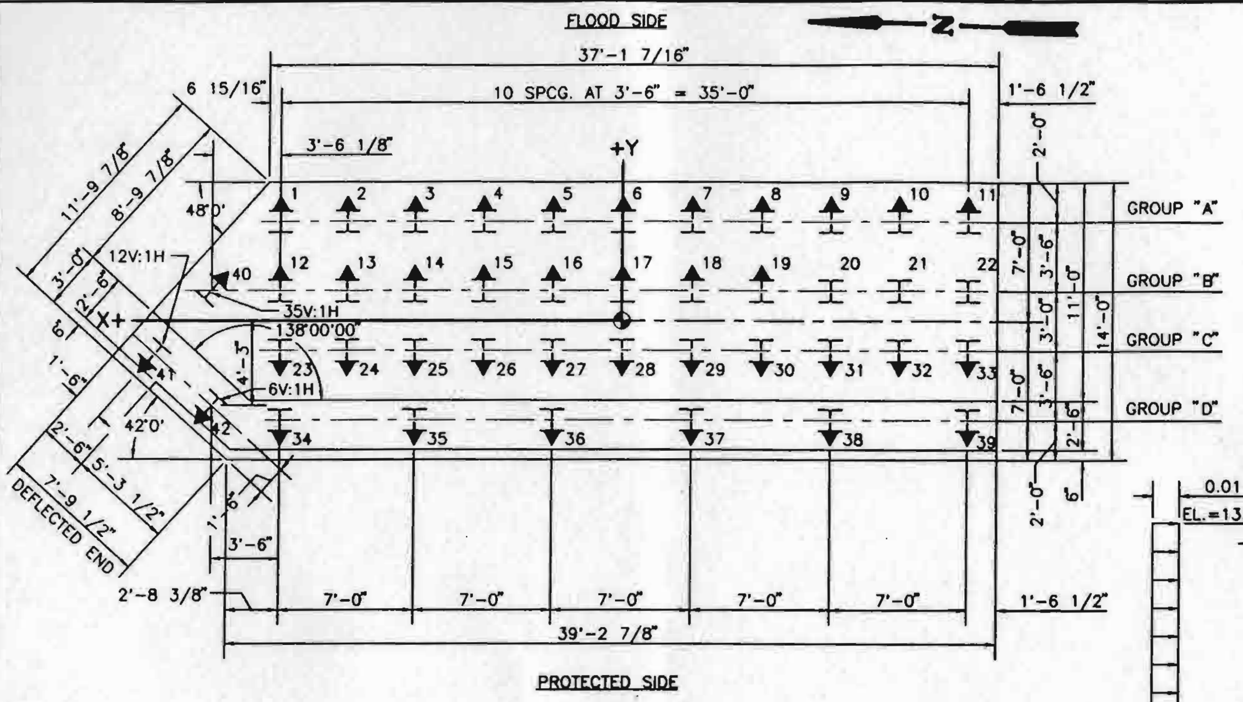
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

MONOLITH G-2 FOUNDATION DESIGN

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

DESIGNED BY S.M. | DRAWN BY B.L. | CHECKED BY S.M.
PLOT SCALE 48 | PLOT DATE 09/18/95 | CADD FILE 4118L013.DWG
DATE: 01/03/95 | FILE NO. H-2-40514



PILE LAYOUT

PILE TIP EL.=-80.43 (60.0)

PILE LEGEND

- 2V ON 1H BATTER-GROUP A-HP14x73 (ASTM A36)
- 2V ON 1H BATTER-GROUP A-HP14x73 (ASTM A36) (PILES 20, 21 & 22 VERT.)
- 12V ON 1H (ODD NUMBERED PILES) 8V ON 1H (EVEN NUMBERED PILES) BATTER GROUP C-HP14x73 (ASTM A36)
- 12V ON 1H BATTER-GROUP D-HP14x73 (ASTM A36)

APPLIED LOADS

LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	Mz (kips-ft)
▲ I		-10.4	487.6	-1229.8	-1209.3	
II	-20.8	105.9	354.2	-1113.0	-1085.5	3782.5
III	-20.8	105.9	340.4	-1082.8	-774.3	3782.5
* IV	-76.0	652.0	107.0	3183.0	-122.0	1501.0
* V	-76.0	652.0	104.0	2748.0	-524.0	1501.0
* VI	-5.0	-18.0	280.0	-2051.0	-965.0	3068.0
* VII	-5.0	-18.0	269.0	-1941.0	714.0	3068.0

* UNUSUAL LOADING CONDITIONS APPLIED LOAD REDUCED TO 75% OF ACTUAL LOAD FOR 33% INCREASE IN ALLOWABLE STRESS FOR UNUSUAL CONDITION.

▲ CONSTRUCTION CONDITION

LOAD CASES:

CASE I – CONSTRUCTION CASE. OP. WIND. NO WATER OR EARTH LOADS

CASE II – WL IN CANAL AT EL.=11.9 (32.33). STORM WIND, BACKFILL IN PLACE GW AT EL.=6.57 (27.00), IMPERVIOUS CUT-OFF

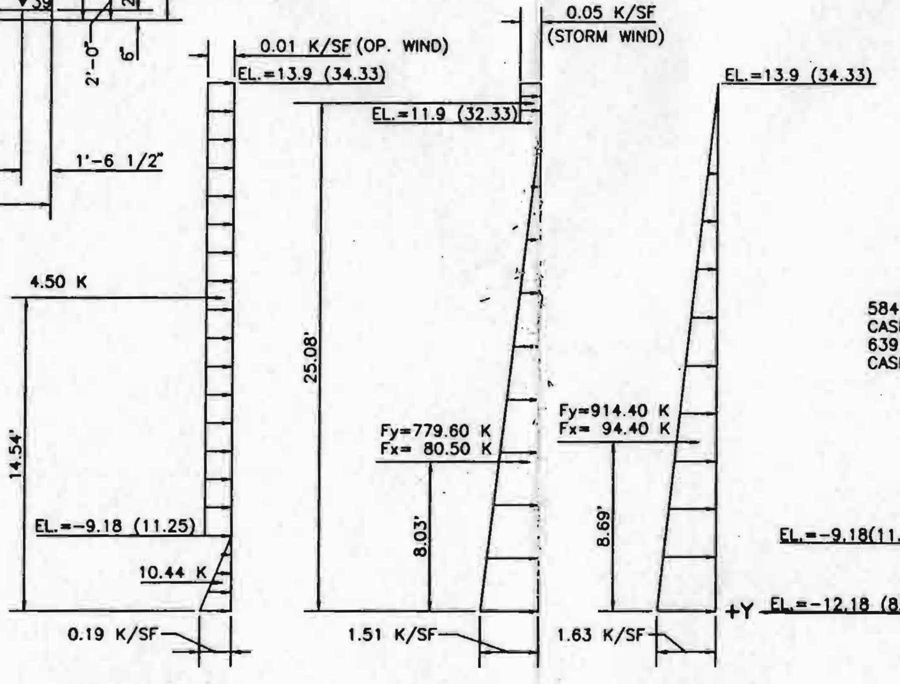
CASE III – WL IN CANAL AT EL.=11.9 (32.33). STORM WIND, BACKFILL IN PLACE GW AT EL.=6.57 (27.00), PERVIOUS CUT-OFF

CASE IV – WL IN CANAL EMPTY, BACKFILL IN PLACE, OP. WIND GW AT EL=6.57 (27.00), IMPERVIOUS CUT-OFF

CASE V – WL IN CANAL EMPTY, BACKFILL IN PLACE, OP. WIND GW AT EL.=6.57 (27.00), PERVIOUS CUT-OFF

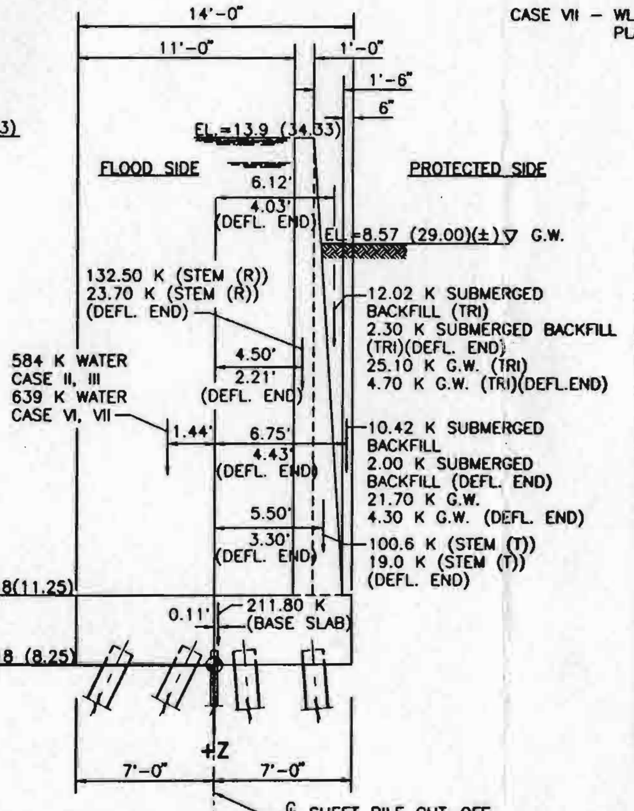
CASE VI – WL IN CANAL AT EL.=13.9 (34.33), STORM WIND, BACKFILL IN PLACE GW AT EL.=6.57 (27.00), IMPERVIOUS CUT-OFF

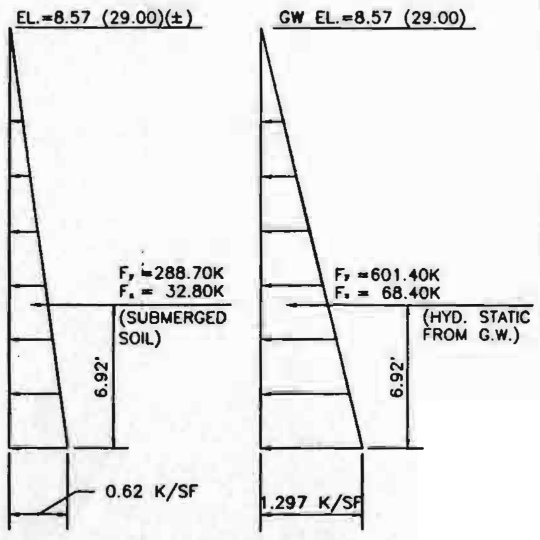
CASE VII – WL IN CANAL AT EL.=13.9 (34.33), STORM WIND, BACKFILL IN PLACE GW AT EL.=6.57 (27.00), PERVIOUS CUT-OFF



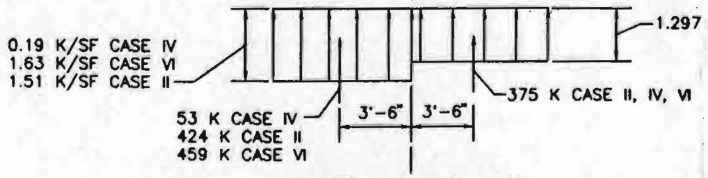
CASE I, IV, V
OP.WIND & GW

CASE II, III
CASE VI, VII
HYD. STATIC PRESSURES

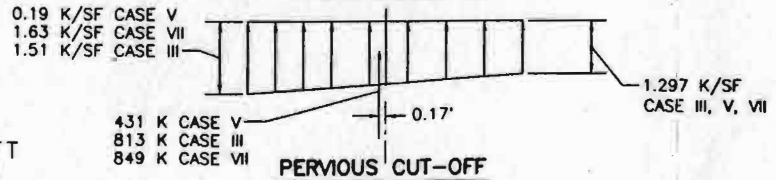




EARTH & HYD. STATIC PRESSURES
CASES II THRU VII



IMPERVIOUS CUT-OFF



PERVIOUS CUT-OFF
UPLIFT LOADS

SUMMARY OF PILE ANALYSIS

LOAD CASE	PILE NO.	MAX. COMP. (KIPS)	% OF ALLOWABLE ALF	% OF ALLOWABLE CBF	PILE NO.	MAX. TENSION (KIPS)	% OF ALLOWABLE ALF	% OF ALLOWABLE CBF	MAX. PILE CAP MOVEMENT X (IN)	MAX. PILE CAP MOVEMENT Y (IN)	MAX. PILE CAP MOVEMENT Z (IN)
I	34	30.2	0.23	0.09					-0.1272E -01	0.5528E -02	0.9455E -02
II	39	66.6	0.51	0.19	11	57.4	0.57	0.20	-0.5974E -01	0.7063E -01	-0.3204E -02
III	12	73.8	0.57	0.25	11	57.1	0.57	0.20	-0.5884E -01	0.7080E -01	-0.3473E -02
IV	19	103	0.79	0.43	40	72.8	0.73	0.24	-0.2313E +00	0.2800E +00	-0.3615E -01
V	19	108.6	0.84	0.45	40	79.9	0.80	0.26	-0.2341E +00	0.3006E +00	-0.3886E -01
VI	39	66.3	0.51	0.19	11	61.6	0.62	0.19	-0.1390E -01	0.4128E -01	-0.3245E -03
VII	39	64.8	0.50	0.18	11	61.0	0.61	0.19	-0.1278E -01	0.3731E -01	-0.1928E -04

MAX. ALLOWABLE COMP. ON HP14x73 = 130 K
MAX. ALLOWABLE TENSION ON HP14x73 = 100 K
} TIP EL. ● -80.43 (-60.0)

ALL ELEVATIONS REFER TO N.G.V.D. NUMBERS IN PARENTHESES INDICATE ELEVATIONS RELATIVE TO CAIRO DATUM.

LEGEND: PRESSURE DIAGRAMS

T OR TRI = TRIANGULAR
R = RECTANGULAR
GW = GROUND WATER
WL = WATER LEVEL

8 0 8 16FT

SCALE: 1/4"=1'-0"

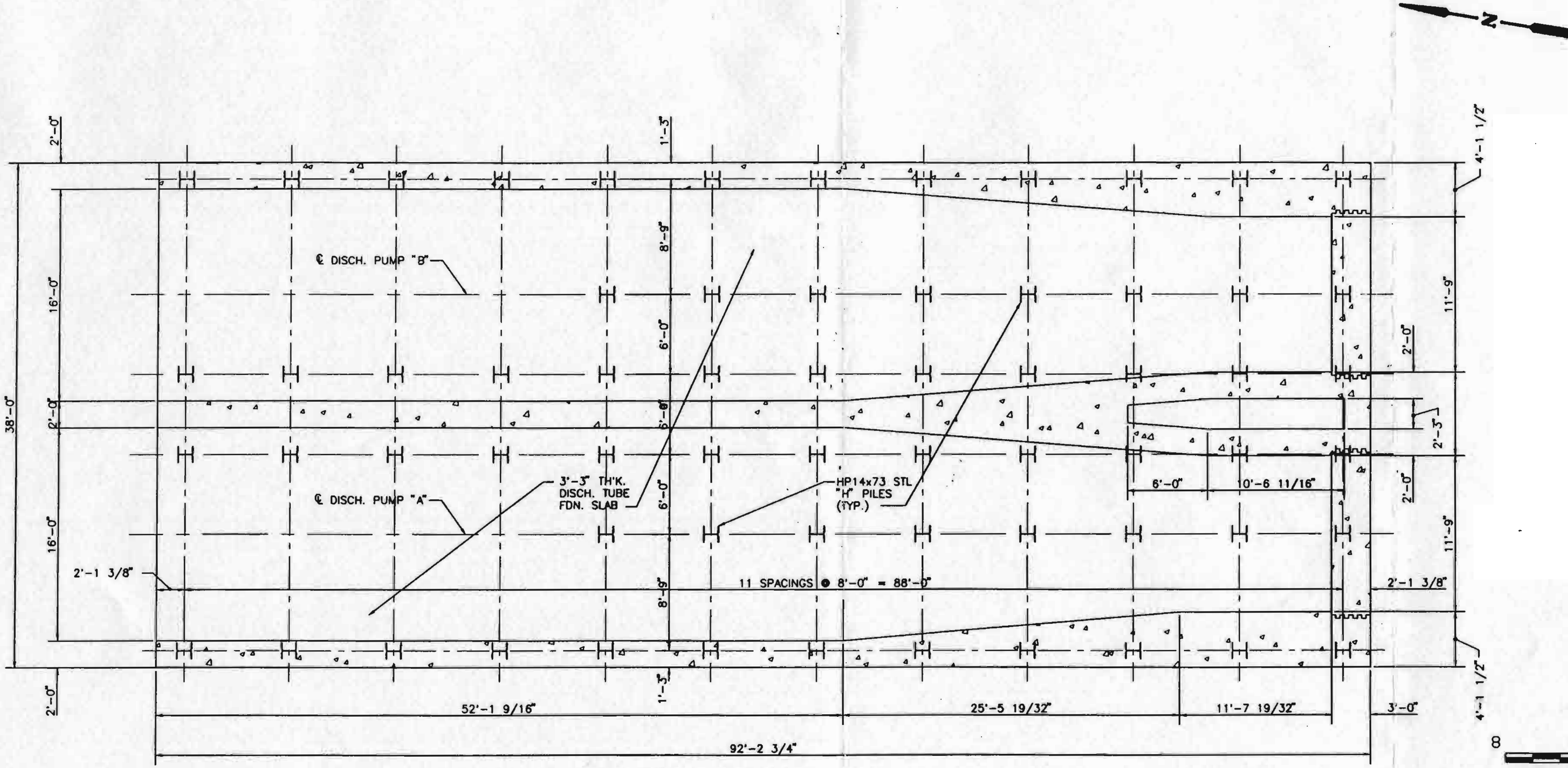
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET — METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

MONOLITH T-3 FOUNDATION DESIGN

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

DESIGNED BY: S.M.
DRAWN BY: B.L.
CHECKED BY: S.M.
PLOT SCALE: 48
PLOT DATE: 09/18/95
DATE: 12/27/94
CADD FILE 4119L012
FILE NO. H-2-40514



PILE LAYOUT
SCALE: 1/4" = 1'-0"

NOTE: ALL PILES TO BE HP14x73 STEEL "H" PILES WITH TIP ELEVATIONS AT -80.43

8 0 8 16FT
SCALE: 1/4"=1'-0"

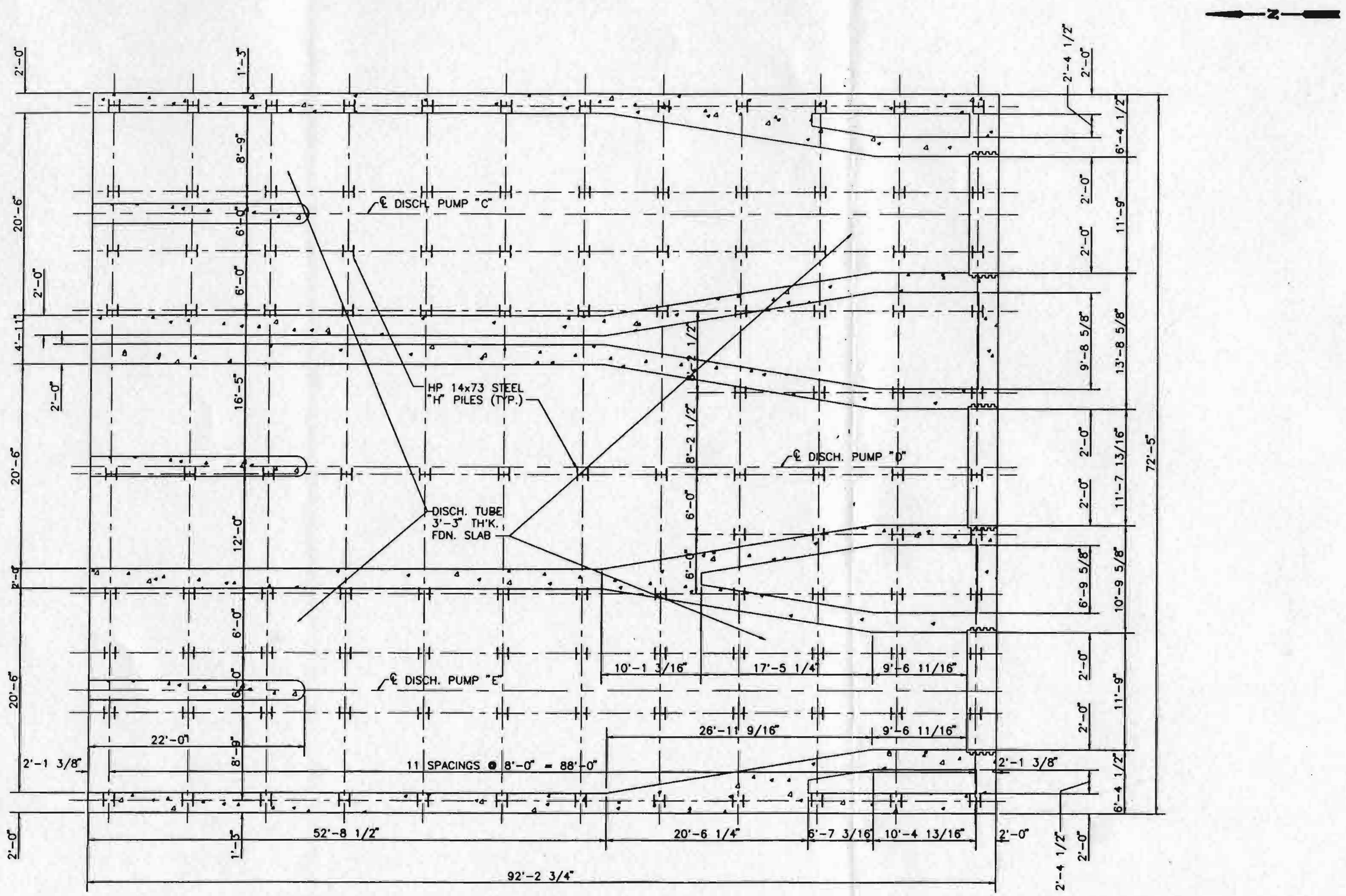
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 28th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A–GENERAL DESIGN
LONDON AVE. OUTFALL CANAL–SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

FOUNDATION DESIGN – DISCHARGE TUBE
FOR PUMPS "A" & "B"

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

DESIGNED BY: S.M.	PLOT SCALE 48	PLOT DATE: 09/14/95	CADD FILE 41198008.DWG
DRAWN BY: K.B.B.	DATE: 12/20/94		FILE NO. H-2-40514
CHECKED BY: S.M.			



PILE LAYOUT

SCALE 3/16"=1'-0"

NOTE: ALL PILES TO BE HP14x73 STL "H" PILES WITH TIPS AT EL. -80.43

8 0 8 16FT

SCALE: 3/16"= 1'-0"

PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

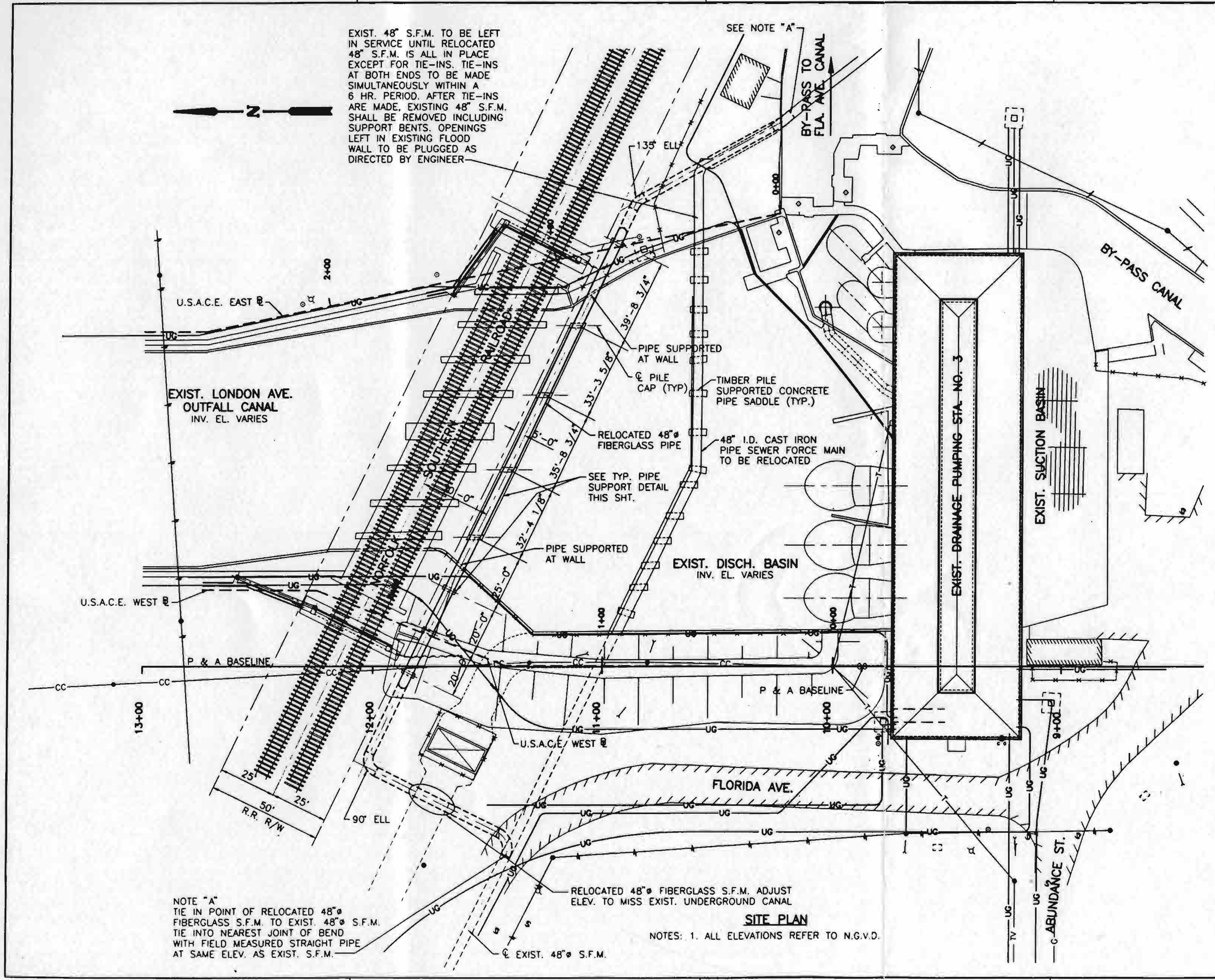
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

FOUNDATION DESIGN - DISCHARGE TUBE
FOR PUMPS "C" "D" & "E"

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

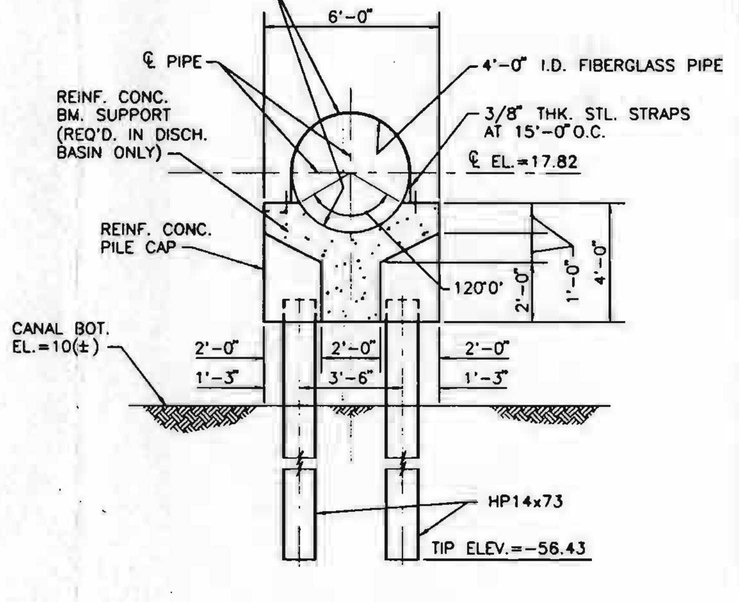
DESIGNED BY: S.M. DRAWN BY: K.B.B. CHECKED BY: S.M.	PLOT SCALE 64	PLOT DATE: 09/15/95	CADD FILE: 41190010.DWG
	DATE: 12/20/95		FILE NO. H-2-40514

PLATE 26



SITE PLAN

NOTES: 1. ALL ELEVATIONS REFER TO N.G.V.D.



TYPICAL PIPE SUPPORT DETAIL

SCALE: 3/8"=1'-0"

40 20 0 40 80FT

SCALE: 1"= 20'

PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

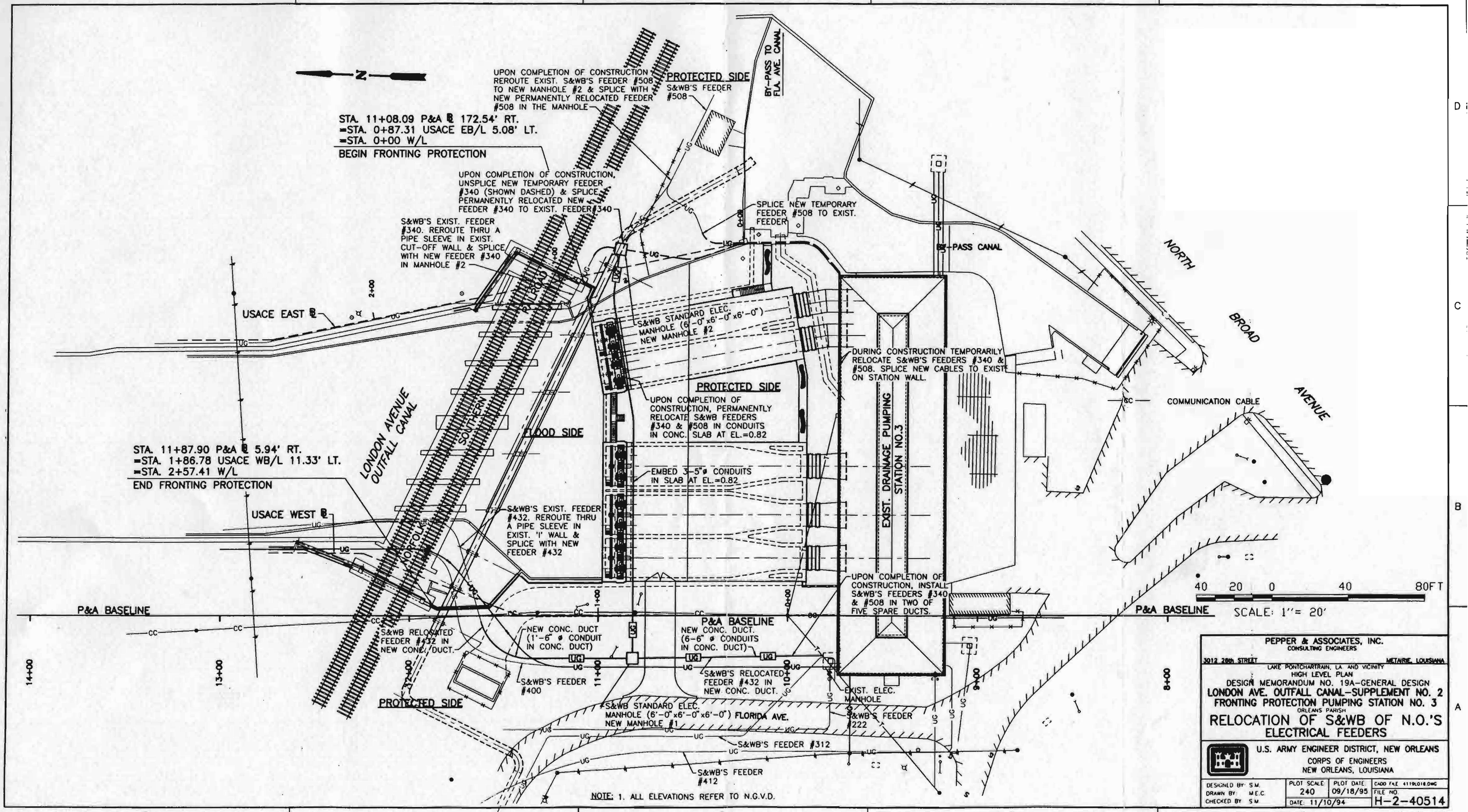
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

RELOCATION OF EXISTING 48"ø S.F.M.

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

DESIGNED BY S.M. / DRAWN BY M.E.D. / CHECKED BY S.M.	PLOT SCALE 240	PLOT DATE 09/14/95	FILE NO. H-2-40514
	DATE 11/11/94		

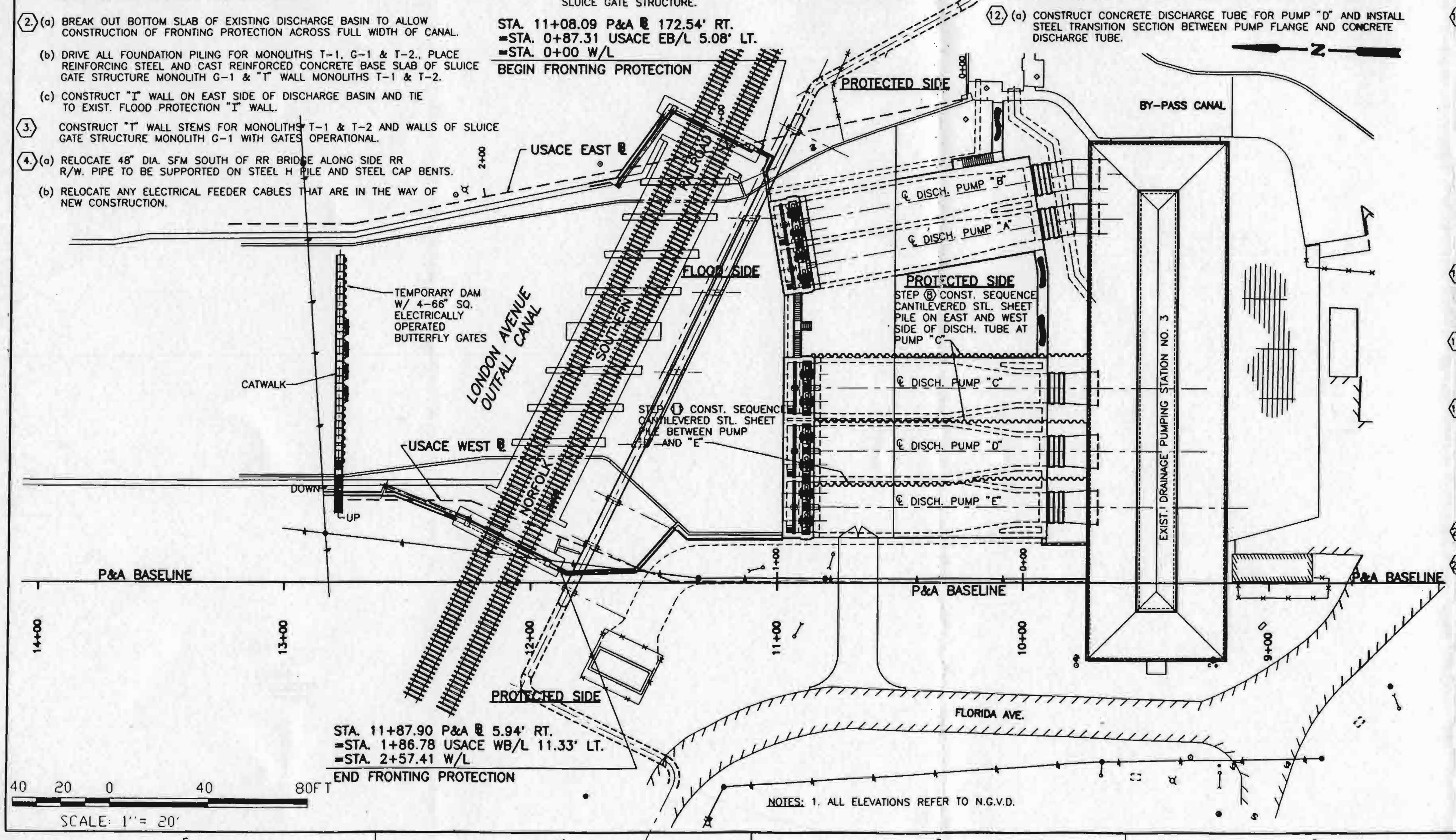
PLATE 27



N
UPON COMPLETION OF CONSTRUCTION REROUTE EXIST. S&WB'S FEEDER #508 TO NEW MANHOLE #2 & SPLICE WITH NEW PERMANENTLY RELOCATED FEEDER #508 IN THE MANHOLE
PROTECTED SIDE
S&WB'S FEEDER #508
BY-PASS TO FLA. AVE. CANAL
STA. 11+08.09 P&A ℄ 172.54' RT.
=STA. 0+87.31 USACE EB/L 5.08' LT.
=STA. 0+00 W/L
BEGIN FRONTING PROTECTION
UPON COMPLETION OF CONSTRUCTION, UNSPLICE NEW TEMPORARY FEEDER #340 (SHOWN DASHED) & SPLICE PERMANENTLY RELOCATED NEW FEEDER #340 TO EXIST. FEEDER #340
SPLICE NEW TEMPORARY FEEDER #508 TO EXIST. FEEDER
S&WB'S EXIST. FEEDER #340. REROUTE THRU A PIPE SLEEVE IN EXIST. CUT-OFF WALL & SPLICE WITH NEW FEEDER #340 IN MANHOLE #2
BY-PASS CANAL
NORTH BROAD AVENUE
USACE EAST ℄
S&WB STANDARD ELEC. MANHOLE (6'-0"x6'-0"x6'-0") NEW MANHOLE #2
DURING CONSTRUCTION TEMPORARILY RELOCATE S&WB'S FEEDERS #340 & #508. SPLICE NEW CABLES TO EXIST. ON STATION WALL.
PROTECTED SIDE
UPON COMPLETION OF CONSTRUCTION, PERMANENTLY RELOCATE S&WB FEEDERS #340 & #508 IN CONDUITS IN CONC. SLAB AT EL.=0.82
COMMUNICATION CABLE
LONDON AVENUE OUTFALL CANAL
SOUTHERN
NORFOLK
FLOOD SIDE
EXIST. DRAINAGE PUMPING STATION NO.3
STA. 11+87.90 P&A ℄ 5.94' RT.
=STA. 1+86.78 USACE WB/L 11.33' LT.
=STA. 2+57.41 W/L
END FRONTING PROTECTION
EMBED 3-5"ø CONDUITS IN SLAB AT EL.=0.82
USACE WEST ℄
S&WB'S EXIST. FEEDER #432. REROUTE THRU A PIPE SLEEVE IN EXIST. 'I' WALL & SPLICE WITH NEW FEEDER #432
UPON COMPLETION OF CONSTRUCTION, INSTALL S&WB'S FEEDERS #340 & #508 IN TWO OF FIVE SPARE DUCTS.
40 20 0 40 80FT
SCALE: 1"= 20'
P&A BASELINE
P&A BASELINE
P&A BASELINE
S&WB RELOCATED FEEDER #432 IN NEW CONC. DUCT.
NEW CONC. DUCT (1'-6" ø CONDUIT IN CONC. DUCT)
NEW CONC. DUCT. (6-6" ø CONDUITS IN CONC. DUCT)
S&WB'S RELOCATED FEEDER #432 IN NEW CONC. DUCT.
EXIST. ELEC. MANHOLE
S&WB'S FEEDER #400
S&WB'S FEEDER #222
PROTECTED SIDE
S&WB STANDARD ELEC. MANHOLE (6'-0"x6'-0"x6'-0") NEW MANHOLE #1
FLORIDA AVE.
S&WB'S FEEDER #312
S&WB'S FEEDER #412
14+00
13+00
12+00
11+00
10+00
9+00
8+00
2+00
1+00
0+00
NOTE: 1. ALL ELEVATIONS REFER TO N.G.V.D.
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 20th STREET
METAIRIE, LOUISIANA
LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH
RELOCATION OF S&WB OF N.O.'S ELECTRICAL FEEDERS
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
DESIGNED BY: S.M.
DRAWN BY: M.E.C.
CHECKED BY: S.M.
PLOT SCALE 240
PLOT DATE: 09/18/95
DATE: 11/10/94
FILE NO. H-2-40514

SUGGESTED GENERAL CONSTRUCTION SEQUENCE

1. (a) CONSTRUCT A CANTILEVERED STEEL SHEETING DAM AT C.O.E. STA. 2+60 E/BL (TREASURE ST.) ACROSS THE CANAL WITH SILL ELEV. AT 1.57 N.G.V.D. (22.0) WITH FOUR (4) 66" SQ. BUTTERFLY GATES; THAN DEWATER AREA BETWEEN TEMPORARY DAM AND DPS#3. WATER MAY BE EMPTIED INTO THE FLORIDA AVE. CANAL.
   (b) SIMULTANEOUSLY CONSTRUCT AN 8" WIDE CONCRETE WALL (WEIR) IN DISCHARGE WELL OF PUMPS "A" AND "B", AT AN ELEVATION ~ 6" 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.
   (c) 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".
2. (a) BREAK OUT BOTTOM SLAB OF EXISTING DISCHARGE BASIN TO ALLOW CONSTRUCTION OF FRONTING PROTECTION ACROSS FULL WIDTH OF CANAL.
   (b) DRIVE ALL FOUNDATION PILING FOR MONOLITHS T-1, G-1 & T-2., PLACE REINFORCING STEEL AND CAST REINFORCED CONCRETE BASE SLAB OF SLUICE GATE STRUCTURE MONOLITH G-1 & "T" WALL MONOLITHS T-1 & T-2.
   (c) CONSTRUCT "I" WALL ON EAST SIDE OF DISCHARGE BASIN AND TIE TO EXIST. FLOOD PROTECTION "I" WALL.
3. CONSTRUCT "T" WALL STEMS FOR MONOLITHS T-1 & T-2 AND WALLS OF SLUICE GATE STRUCTURE MONOLITH G-1 WITH GATES OPERATIONAL.
4. (a) RELOCATE 48" DIA. SFM SOUTH OF RR BRIDGE ALONG SIDE RR R/W. PIPE TO BE SUPPORTED ON STEEL H PILE AND STEEL CAP BENTS.
   (b) RELOCATE ANY ELECTRICAL FEEDER CABLES THAT ARE IN THE WAY OF NEW CONSTRUCTION.
5. DRIVE REST OF FOUNDATION PILING FOR FRONTING PROTECTION AND CAST REINF. CONC. BASE SLAB OF SLUICE GATE STRUCTURE MONOLITH G-2.
6. CONSTRUCT WALLS OF SLUICE GATE STRUCTURE FOR MONOLITHS G-2 WITH GATES OPERATIONAL.
7. (a) TAKE PUMP "C" OUT OF SERVICE.
   (b) REMOVE DISCHARGE PIPING FROM FLANGE INSIDE BUILDING WALL INCLUDING DISCHARGE HOOD.
   (c) CLOSE SLUICE GATES FOR PUMP "C" IN SLUICE GATE STRUCTURE.
8. 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.
9. (a) CONSTRUCT CONCRETE DISCHARGE TUBE FOR PUMP "C" AND INSTALL STEEL TRANSITION SECTION BETWEEN PUMP FLANGE AND CONCRETE DISCHARGE TUBE.
   (b) RESTORE PUMP "C" TO SERVICE AND OPEN SLUICE GATES FOR PUMP "C".
   (c) REMOVE STEEL SHEETING ON EAST AND WEST SIDE OF NEW CONCRETE DISCHARGE TUBE OF PUMP "C".
10. (a) TAKE PUMP "D" OUT OF SERVICE.
    (b) REMOVE STEEL DISCHARGE PIPING FROM FLANGE INSIDE BUILDING WALL INCLUDING DISCHARGE HOOD.
    (c) CLOSE SLUICE GATES FOR PUMP "D" IN SLUICE GATE STRUCTURE.
11. 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.
12. (a) CONSTRUCT CONCRETE DISCHARGE TUBE FOR PUMP "D" AND INSTALL STEEL TRANSITION SECTION BETWEEN PUMP FLANGE AND CONCRETE DISCHARGE TUBE.
    (b) RESTORE PUMP "D" TO SERVICE AND OPEN SLUICE GATES FOR PUMP "D".
13. REMOVE STEEL SHEETING BETWEEN PUMPS "D" AND "E".
14. (a) TAKE PUMP "E" OUT OF SERVICE.
    (b) REMOVE STEEL DISCHARGE PIPING FROM FLANGE INSIDE BUILDING INCLUDING DISCHARGE HOOD.
    (c) CLOSE SLUICE GATES FOR PUMP "E" IN SLUICE GATE STRUCTURE.
15. (a) CONSTRUCT CONCRETE DISCHARGE TUBE FOR PUMP "E" AND INSTALL STEEL TRANSITION SECTION BETWEEN PUMP FLANGE AND CONCRETE DISCHARGE TUBE.
    (b) RESTORE PUMP "E" TO SERVICE AND OPEN SLUICE GATES FOR PUMP "E".
16. (a) CLOSE SLUICE GATES FOR PUMPS "A" AND "B" IN SLUICE GATE STRUCTURE.
    (b) TAKE PUMPS "A" AND "B" OUT OF SERVICE.
    (c) REMOVE STEEL DISCHARGE PIPING INCLUDING DISCHARGE HOODS FROM FLANGES INSIDE BUILDING WALL.
    (d) RELOCATE CONSTANT DUTY PUMP DISCHARGE PIPING TO MARIGNY GATE CLOSURE LOCATION.
    (e) INSTALL LOW SILL DAM ON EAST SIDE OF MARIGNY GATE TO KEEP WATER FROM BACKING UP FROM THE FLORIDA AVE. CANAL. REMOVE EXISTING BUTTERFLY GATE (MARIGNY GATE). SEAL OPENING WITH A CONCRETE RETAINING WALL AND PROVIDE SLEEVE FOR CD PIPING.
    (f) REMOVE LOW SILL DAM FROM EAST SIDE OF MARIGNY GATE.
    (g) REMOVE EXISTING LONDON AVE. GATE AND RELATED STRUCTURES.
    (h) REMOVE EXISTING WEST RETAINING WALL FROM LONDON AVE. GATE TO STATION BUILDING.
17. (a) CONSTRUCT CONCRETE DISCHARGE TUBE FOR PUMPS "A" AND "B" AND INSTALL STEEL TRANSITION SECTIONS BETWEEN PUMP FLANGES AND CONCRETE DISCHARGE TUBES.
    (b) RESTORE PUMPS "A" AND "B" TO SERVICE.
18. (a) RELOCATE 48" DIA. SFM TO PERMANENT LOCATION ON PROTECTED (SOUTH) SIDE OF SLUICE GATE STRUCTURE.
    (b) REMOVE TEMPORARY 48" DIA. SFM INCLUDING PILE BENTS. RESTORE CONCRETE LINING.
19. (a) CONSTRUCT MONOLITH T-3, WEST SIDE
    (b) BREAK OUT EXISTING BOTTOM SLAB TO PERMIT REMOVAL OF EXISTING TIMBER PILES IN CONFLICT WITH NEW STEEL H-PILES.
    (c) DRIVE NEW STEEL H-PILES.
    (d) CONSTRUCT FOUNDATION AND STEM.
    (e) RESTORE CONCRETE CANAL BOTTOM.
20. CONSTRUCT I-WALL WEST SIDE OF DISCHARGE BASIN AND TIE TO EXISTING FLOOD PROTECTION I-WALL.
21. REMOVE TEMPORARY SHEET PILE DAM AT TREASURE ST. AND REPAIR CONCRETE LINING.



NOTES: 1. ALL ELEVATIONS REFER TO N.G.V.D.

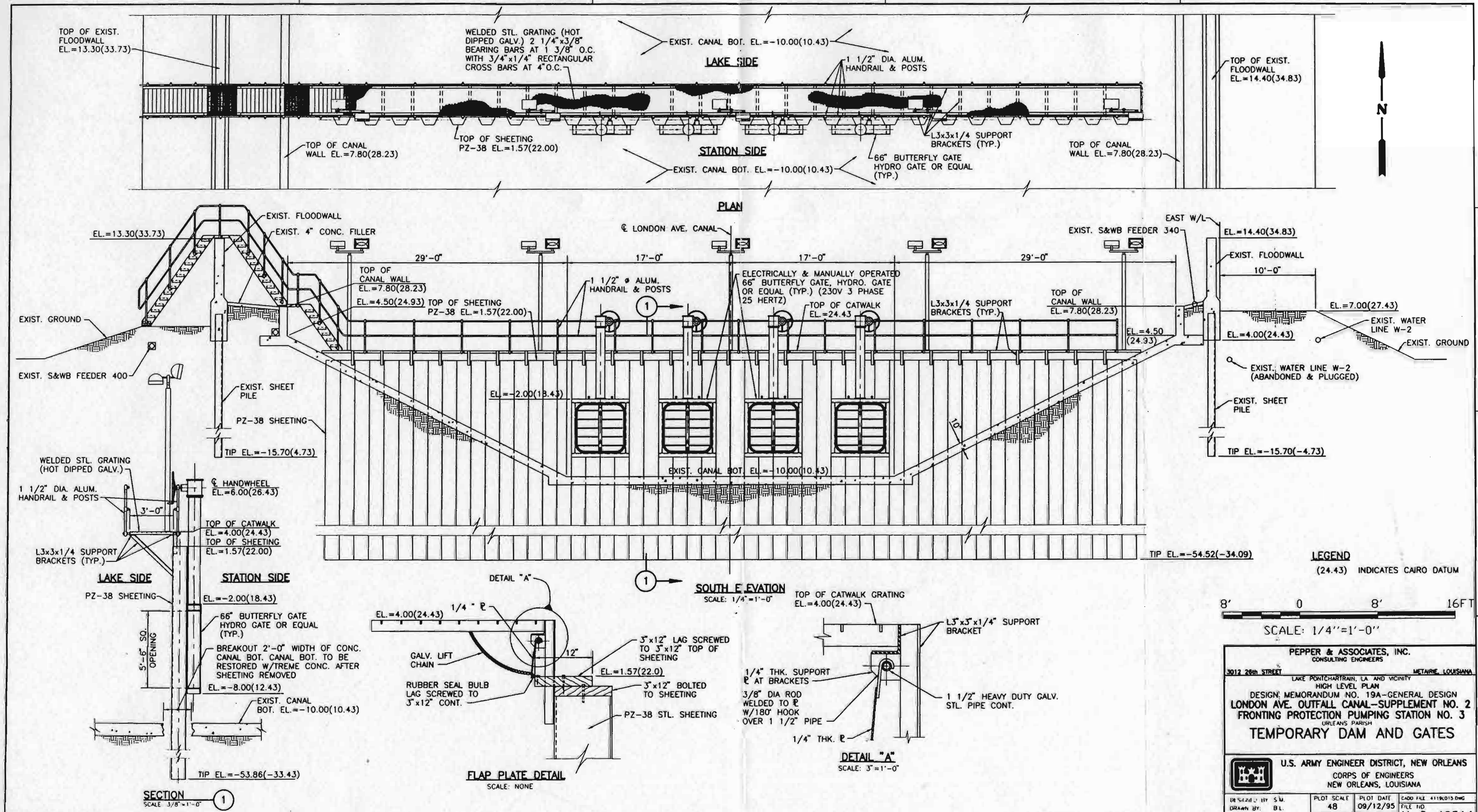
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

TEMPORARY DAM AND CONSTRUCTION COFFERDAM PLAN

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

DESIGNED BY S.M. DRAWN BY M.E.C. CHECKED BY S.M.	PLOT SCALE 240	PLOT DATE: 09/18/95	CADD FILE 4119L06B.DWG
	DATE 11/10/94		FILE NO. H-2-40514



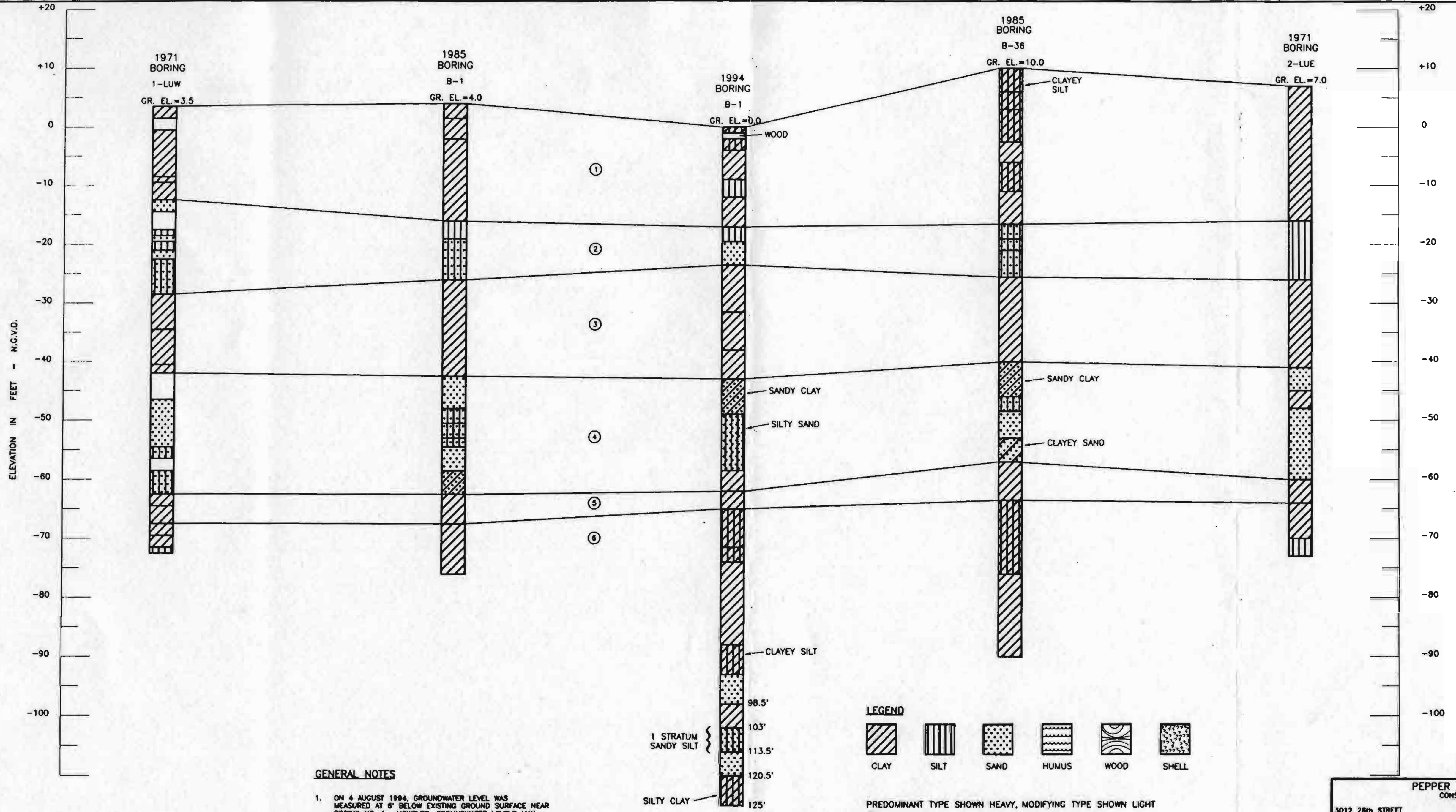
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A–GENERAL DESIGN
LONDON AVE. OUTFALL CANAL–SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

TEMPORARY DAM AND GATES

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

DESIGNED BY S.M. DRAWN BY: B.L. CHECKED BY S.M.	PLOT SCALE 48	PLOT DATE 09/12/95	CADD FILE 4119L015.DWG
	DATE 02/22/95		FILE NO H-2-40514



SUBSOIL PROFILE

PREDOMINANT TYPE SHOWN HEAVY, MODIFYING TYPE SHOWN LIGHT

1. ARTIFICAL FILL AND NATURAL LEVEE (CH AND CL)
2. INTRADELTA (ML, SM AND SP)
3. PRODELTA (CH)
4. NEARSHORE GULF (SP, SM, SC AND CL)
5. NEARSHORE GULF (CH)
6. PLEISTOCENE (CH AND CL)

GENERAL NOTES

1. ON 4 AUGUST 1994, GROUNDWATER LEVEL WAS MEASURED AT 6' BELOW EXISTING GROUND SURFACE NEAR BORING NO. 1. HOWEVER, GROUNDWATER LEVELS MAY VARY DUE TO SEASONAL AND OTHER FACTORS. IF IMPORTANT TO CONSTRUCTION, THE DEPTH TO GROUNDWATER 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.
3. REFER TO PLATE 2 FOR BORING LOCATIONS.

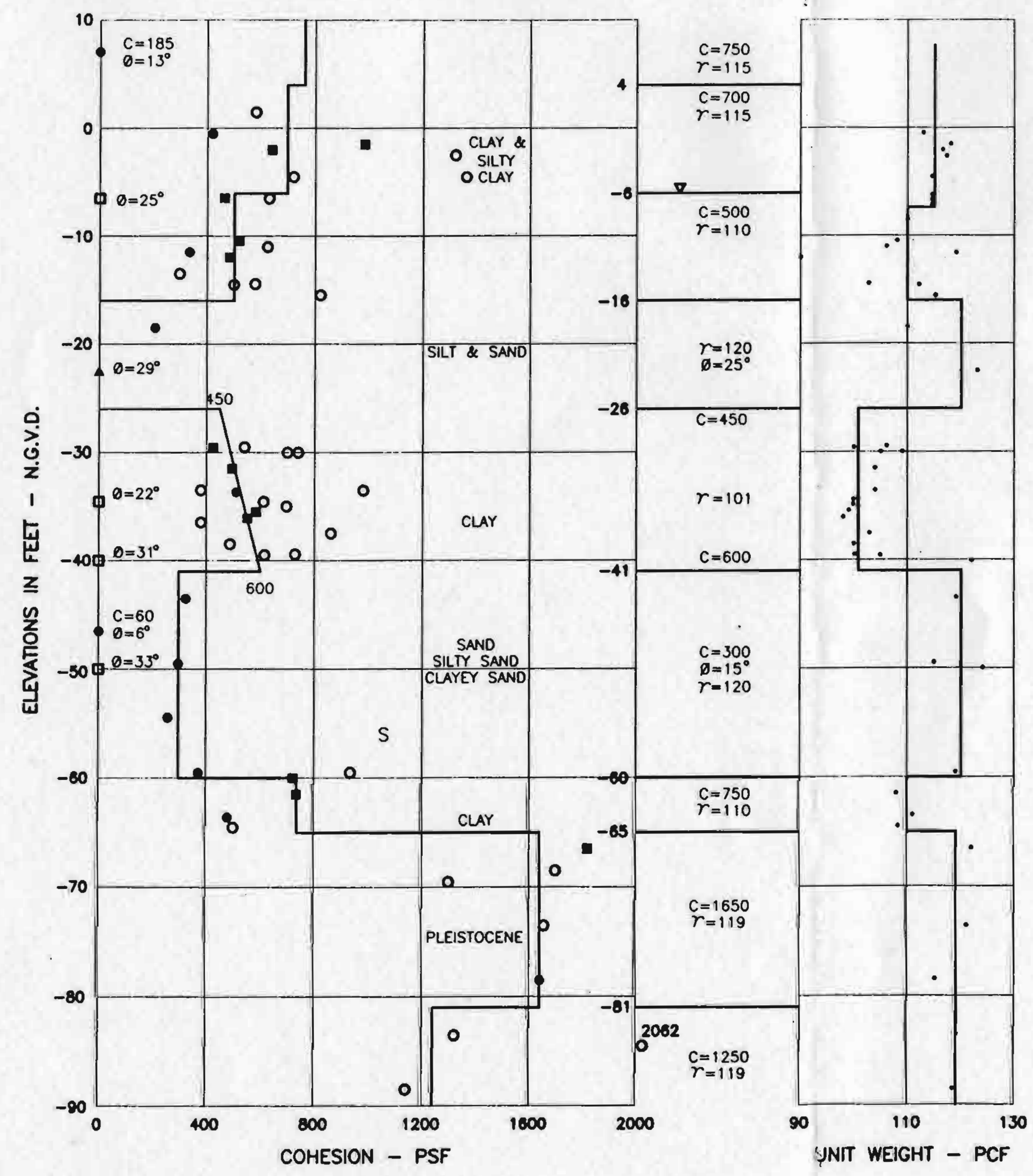
PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 28th STREET METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
FRONTING PROTECTION PUMPING STATION NO. 3
ORLEANS PARISH

SOIL BORING PROFILES

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

DESIGNED BY: S.M. DRAWN BY: K.B.B. CHECKED BY: S.M.	PLOT SCALE: 96	PLOT DATE: 09/12/95	CADD FILE: 41198012.DWG
	DATE: 01/16/95		FILE NO. H-2-40514



GROUND EL.		BORING
7.0	-	2-LUE (1971)
3.5	-	1-LUW (1971)
0.0	-	B-1 (1994)
4.0	-	B-1 (1985)
10.0	-	B-36 (1985)

LAB TESTS

- O UC
- ● OB
- ◘ S
- ■ Q
- ▲ R

PEPPER & ASSOCIATES, INC.
CONSULTING ENGINEERS
3012 26th STREET — METAIRIE, LOUISIANA

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A-GENERAL DESIGN
LONDON AVE. OUTFALL CANAL-SUPPLEMENT NO. 2
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ORLEANS PARISH

SOIL DESIGN PARAMETERS
PUMPING STATION NO. 3

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

DESIGNED BY DRAWN BY KEM CHECKED BY	PLOT SCALE 120	PLOT DATE 09/18/95	CADD FILE 4119SOP.DWG
	DATE: 01/13/95		FILE NO. H-2-40514