GEOTECHNICAL INVESTIGATION

ORLEANS LEVEE DISTRICT

LONDON AVENUE CANAL

MIRABEAU AND FILMORE AVENUE BRIDGES

NEW ORLEANS, LOUISIANA

FOR
MEYER ENGINEERS, LTD.
METAIRIE, LOUISIANA

9 APRIL 1998

EUSTIS ENGINEERING COMPANY, INC.
GEOTECHNICAL ENGINEERS
CONSTRUCTION QUALITY CONTROL & MATERIALS TESTING
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9 April 1998

Meyer Engineers, Ltd.
Suite B
4937 Hearst Avenue
Metairie, Louisiana 70004

Attention Mr. Jitendra Shah

Gentlemen:

Geotechnical Investigation
Orleans Levee District
London Avenue Canal
Mirabeau and Filmore Avenue Bridges
New Orleans, Louisiana

Transmitted are three copies of our engineering report covering a geotechnical investigation for the subject project.

Thank you for asking us to perform these services.

Yours very truly,

EUSTIS ENGINEERING COMPANY, INC.

GWENDOLYN PHILIPS SANDERS, P.E.

STATE OF LOUISIANA
GWENDOLYN P. SANDERS
REG. NO. 27104
REGISTERED PROFESSIONAL ENGINEER IN CIVIL ENGINEERING

GPS:aln/ejg

EE 15166

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By
Eustis Engineering Company, Inc.
Metairie, Louisiana

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LONDON AVENUE CANAL
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NEW ORLEANS, LOUISIANA

INTRODUCTION

1. This report contains the results of a geotechnical investigation performed for the proposed bridges over the London Avenue Canal at Mirabeau and Filmore Avenues in New Orleans, Louisiana. The investigation was performed in general accordance with Eustis Engineering Company, Inc.'s letters of proposal dated 17 October 1995 and 11 December 1997. Authorization to proceed with the investigation was given by Meyer Engineers, Ltd.

2. This report has been prepared in accordance with generally accepted geotechnical engineering practice for the exclusive use of Meyer Engineers for specific application to the sites. In the event any changes in the nature, design, or location of the proposed bridges are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report are modified or verified in writing. Should these data be used by anyone other than Meyer Engineers, they should contact Eustis Engineering for interpretation of data and to secure any other information which may be pertinent to this project.

3. Recommendations and conclusions contained in this report are to some degree subjective. The report in its entirety should not be included in contract plans and specifications. However, the results of the soil borings and laboratory tests contained in Appendix I of this report may be included in plans and specifications.

4. It should further be noted the analyses and recommendations contained in this report are based in part on data obtained from soil borings. The nature and extent of subsurface variations and subsoil conditions between and away from the boring locations may not become evident until construction. If variations then appear, it will be necessary to reevaluate the recommendations contained in this report. .

PREVIOUS GEOTECHNICAL INVESTIGATION

5. Borings and laboratory test results made for a previous geotechnical investigation at the project site were used to supplement our analyses. The previous investigation is contained in Eustis Engineering's report entitled "Geotechnical Investigation, London Avenue Canal, Levee and Floodwall Improvements, Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana," dated 4 March 1986.

SCOPE

6. The analyses made for this report used existing soil borings and laboratory test data. Engineering analyses were performed to determine estimates of the allowable pile load capacities for support of the proposed bridges. Analyses were also made to evaluate slope stability, cantilever I-wall stability, seepage, and settlement. In addition, recommendations were formulated regarding site preparation, and foundation construction procedures. Engineering analyses were performed in accordance with the U.S. Army Corps of Engineers' criteria.

SOIL BORINGS

7. Previous Borings. Borings 15 and 50 made for the previous investigation were used for the analyses at the bridge at Mirabeau Avenue. Borings 18, 19, and 53 made for the previous investigation were used for the analyses at the bridge at Filmore Avenue. Detailed descriptive logs of the borings are shown in tabular and graphical form in Appendix I. Summaries of the laboratory test results are included on separate sheets following the boring logs in Appendix I.

FOUNDATION ANALYSES

Available and Furnished Information

8. Plan drawings of the proposed bridge and I-wall tie-in configurations were provided by Meyer Engineers. Profile drawings of the existing adjacent I-wall

flood protection were also provided. The top of the proposed I-wall is el 14.4 NGVD. No levee degrading is anticipated at the proposed tie-in sections.

9. The design still water level (SWL) was furnished as el 11.85. The design low water level within the London Avenue Canal is at approximate el -3, with a low storm water level at el -5. The normal water level within the canal is at approximate el 1.

10. Square, precast concrete piles are proposed for support of the intermediate bridge bents within the London Avenue Canal. HP14x73 steel H-Piles are proposed for support of the bridge abutments. Existing piles at the bridge bents will be pulled prior to installation of the concrete piles. These existing piles penetrate to el -24 and -23 at Mirabeau and Filmore Avenues, respectively.

11. Consideration is being given to using a factor of safety of 3 to estimate the allowable pile load capacities in lieu of verification by a load test. If a load test is performed, a factor of safety of 2 will be used to estimate the allowable pile load capacities.

Foundation Recommendations

12. We recommend existing piles not be pulled from the foundations. These piles should remain in place and be cut off below the mudline. Attempts should be made to relocate new piles so that existing piles may remain in place. New piles should be located no closer than 2 feet from existing piles. If existing

piles are pulled, new piles driven at existing pile locations will exhibit a reduction in load carrying capacity. Reduced allowable pile load capacities have been provided for piles at the bridge bents which do not meet these criteria.

13. We recommend a pile load test be performed to verify the allowable pile load capacities. The test pile will also provide valuable information regarding vibrations from pile driving, and requirements for predrilling. Pile integrity and hammer efficiency can also be evaluated during the driving of the test pile with the performance of a dynamic load test. Further discussion of the requirements for a pile load test follow in this report.

Design Parameters

14. Analyses for the proposed I-walls are based on borings made adjacent to the London Avenue Canal for the previous investigation. The USACE requires after construction (Q-case) factors of safety of 1.5 and 1.0, for cantilever I-wall analyses for the SWL and SWL + 2, respectively. No wave loads are anticipated and, therefore, long term (S-case) analyses are not required for the floodwall.

15. An SWL at el 11.85 was used in our slope stability analyses for failure toward the land side. A low water level at el -5 was used for our stability analyses for failure toward the canal. Corresponding opposite side water levels for these analyses were taken at the existing ground surface.

I-Wall Analyses

16. Methods. Based on the furnished cross-sections of the existing floodwalls and plan drawings of the proposed I-wall, analyses were performed to determine the penetration and factor of safety with regard to slope stability. Overall slope stability analyses were performed by the LMVD Method of Planes Analysis. Penetration and structural requirements were determined using the Corps of Engineers' computer program, CWALSHT. Underseepage was evaluated using Lane's Weighted Creep Ratio analysis.

17. Cantilever Analyses. I-wall analyses were performed for Q-case conditions to determine penetration requirements. A summary of the cases analyzed and results are tabulated on Figure 1. The governing cases are shown in bold on this figure. Based on our I-wall analyses, a minimum pile tip at el -27 is required for the proposed typical I-wall tie-in at Mirabeau Avenue. A minimum pile tip at el -33 is required for the proposed typical I-wall tie-in at Filmore Avenue.

18. The results of our analyses and configurations analyzed are shown on Figures 2 and 3. If any degrading is proposed adjacent to the walls, Eustis Engineering should be contacted to reevaluate the recommended tip embedments. Computer output corresponding to the governing I-wall analyses are given as Appendices II and III for Mirabeau and Filmore Avenues, respectively.

19. Slope Stability Analyses. Slope stability analyses were performed by the LMVD Method of Planes Analysis for the configurations shown on Figures 2

through 5. Analyses for stability toward the protected side assume a high water level within the canal at el 11.85, (Figures 2 and 3). Analyses for stability toward the floodside assume the low water level within the canal is at el -5, (Figures 4 and 5). Our analyses indicate factors of safety of at least 1.3 for both high and low water conditions at each bridge. Eustis Engineering should be contacted to reevaluate stability if any degrading operations are proposed.

Seepage Cutoff at Bridge Abutments

20. Underseepage analyses were performed to evaluate the penetrations selected based on I-wall and stability analyses. These analyses were performed using Lane's Weighted Creep Ratio analysis. We have assumed a value of 7 is required in fine sands for Lane's Weighted Creep Ratio. Our analyses indicate the proposed sheetpile tip elevations are adequate. A summary of the parameters used for our seepage analyses is shown on Figure 6.

Pile Foundations

21. Allowable Pile Load Capacities. Analyses have been made to determine the estimated allowable single pile load capacities for various embedments and sizes of square, precast concrete piles used in tension and compression for support of the proposed bridge bents. The results of these analyses are shown graphically on Figures 7 through 10. These analyses are based on mudlines at el -10 at Mirabeau Avenue and at -9 at Filmore Avenue.

22. Analyses have also been made to determine the estimated allowable single pile load capacities for various embedments of HP14x73 steel H-Piles used in tension and compression for support of the proposed bridge abutments. The results of these analyses are shown graphically on Figures 11 through 14. These analyses are based on the abutment ground surface at el 3 at Mirabeau Avenue and at el 2 at Filmore Avenue.

23. Full allowable pile capacities shown on Figures 7 through 14 include a 2-ft cutoff below the ground surface or mudline for the pile cap or scour. Full allowable pile capacities should be used for piles driven in "new" locations. If piles are pulled or existing piles interfere with the installation of new piles, reduced allowable capacities should be used. Reduced allowable capacities for concrete piles at the bridge bents are provided on Figures 15 through 18. These capacities neglect skin friction on the pile section for the full depth of the pulled pile. Existing piles should not be used for support.

24. <u>Structural Capacity.</u> Our analyses also contain estimated factors of safety of 2 or 3 against failure of a single pile through the soil. 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 should be determined by a structural engineer. Depending on the pile length selected, a spliced pile may be required. A tension connector must be used at the splice for the piles to develop uplift capacities.

25. Batter Piles. The estimated pile load capacities are for piles driven vertically and may be used to determine the pile load capacities for batter piles. The vertical component of the batter pile will be equal to the capacity for a vertical 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 geometric relationship is shown on Figure 19. Two section piles driven on a batter should splice with a connector sufficient to transfer tension and moments for both anticipated applied loads ***and*** loads associated with installation.

26. Modulus of Horizontal Subgrade Reaction. The modulus of horizontal subgrade reaction (k_h) may be estimated using the tables given on Figures 20 and 21 for Mirabeau and Filmore Avenues, respectively.

Where: k_h = modulus of horizontal subgrade reaction (lbs/in.^{3})

B = width or diameter of pile (in.)

C = reduction factor for cyclic loading

C = 0.5 for cyclic loading

C = 1.0 for initial loading

D = reduction factor for effect of group action

27. Pile Group Capacity and Spacing. Piles used in tension and compression will derive their supporting capacity through skin friction. It will be necessary to consider group effects for these piles. In this regard, the group perimeter shear formula shown on Figure 22 should be used. 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 22. New piles should be positioned no closer than 2 feet (clear distance) from existing piles.

28. Estimated Settlement. The estimated settlement of pile foundations is ½ to ¾ inch for pile lengths recommended in this report used to support structural loads from the bridges. This estimate of settlement assumes piles will be single rows of vertical piles supporting individual pile bents.

29. Piles for each feature should be driven to the same tip embedments unless excessive driving resistances are encountered in order to minimize differential foundation settlement. We have also assumed less than 1 foot of additional fill will be required. In the event any of our assumptions are not met, Eustis Engineering should be contacted to evaluate the potential settlement of pile foundations.

30. Pile Driving. Close field supervision should be maintained by experienced personnel to ensure proper procedures are followed and accurate records are kept for all pile driving operations. The driving record should include the type, size, length and embedment of piles, the number of blows per foot of penetration, the depth and diameter of predrill operations, and the equipment used to drive the piles. An accurate driving record is especially important to verify the piles are driven to the required tip embedment and to give an indication of any unusual driving characteristics which may indicate pile breakage.

31. The hammer used to install the piles will depend on the pile's full allowable compressive capacity. A single acting air hammer with a minimum

manufacturer's rated energy of 24,500 ft-lbs per blow should be used to drive the concrete piles with allowable compressive capacities of 70 tons or less, based on a factor of safety of 2. Piles having allowable compressive capacities between 70 and 120 tons (factor of safety ≈ 2) should be driven with a single acting air hammer with a minimum manufacturer's rated energy of 30,000 ft-lbs per blow. Piles with allowable compressive capacities over 120 tons may be driven with a single acting air hammer delivering 48,750 ft-lbs of energy per blow. The hammer used to install precast concrete piles should have a ram stroke of no more than 3 feet and a ram weight of approximately one-third to one-half of the weight of the pile driven.

32. Predrilling. Concrete piles may require a predrilled pilot hole in order to minimize vibrations during pile driving operations. Predrilling may be accomplished by wet rotary methods using a fishtail bit. The diameter of the predrilled hole should not exceed 75% of the pile side dimension. Assuming the ground surface is at el 5 at the borings, predrilling should be anticipated to be no deeper than approximately el -55 at both sites. For this predrill depth, a minimum pile tip embedment at el -65 is required. We do not recommend pile embedments above el -65. Requirements for predrilling should be determined during the driving of probe piles for the test pile program.

33. Jetting. ***We do not recommend piles for this project be installed with the aid of jetting unless jetting is necessary to reduce vibration to adjacent structures.*** Jetting should not be used to install steel H-Piles. If jetting is necessary for concrete piles, jet nozzles should be cast into the precast concrete pile section and the requirements for jetting should be determined during the installation of probe

piles for the test pile program. Eustis Engineering should be contacted to reevaluate the allowable pile load capacities presented in this report if jetting is considered.

34. Dynamic Analyses. The concrete piles should have cross-sections which are structurally sufficient to facilitate driving of the piles without damage. Dynamic analysis (WEAP) can be performed to evaluate driving stresses and driveability once the hammer and appurtenant equipment have been selected. Structural requirements can then be verified by a structural engineer and installation criteria can be established.

35. Vibrations. Pile driving will cause vibrations which may affect nearby structures, pavements, and utilities. We recommend peak particle velocities be monitored at critical structures or pavements with a seismograph during pile driving operations. The record of peak particle velocities will provide information in assessing potential damage and the need for changes in driving operations.

36. Peak particle velocities in excess of 0.5 in./sec measured at a structure may induce damage to ground supported structures. Pile supported structures will be limited to cosmetic damage for peak particle velocities of 0.5 in./sec. Peak particle velocities on the order of 1.5 to 2 in./sec are generally regarded as the threshold for structural damage to pile supported features.

37. Peak particle velocities of 0.25 in./sec as measured by the seismograph are generally regarded as a vibration level uncomfortable to human perception. In addition, peak particle velocities in excess of 0.25 in./sec may densify loose

cohesionless deposits resulting in settlement of structures or pavements founded in these deposits. Such deposits exist at these sites. For sustained peak particle velocities in excess of 0.25 in./sec at a pavement or structure of concern, Eustis Engineering should be notified, driving operations terminated, and consideration given to altering installation criteria.

38. Dynamic Pile Test. We recommend WEAP analysis be supplemented by a DPT using a Pile Driving Analyzer® to monitor selected test piles or job piles during installation. The performance of a DPT is used to evaluate actual driving stresses, penetration resistance, and the integrity and capacity of the job piles. The PDA can also monitor energy transferred to the pile from the hammer and evaluate installation efficiency.

39. Test Piles and Load Tests. Once the design pile types and lengths have been selected, we recommend at least two probe piles of each type and length proposed for use be driven near the proposed abutments at each bridge. The test piles should be the same type used for construction and should be driven with the same equipment and techniques used to drive the job piles. These test piles will provide more definitive information on predrill requirements and vibrations from pile driving.

40. The test piles should be monitored with a PDA to evaluate driving stresses, installation efficiency, and static capacity during driving. This monitoring is particularly critical for square, precast concrete piles to determine the hammer stroke required to minimize damage to the piles and reach the required tip

embedment. The results of the dynamic tests should be evaluated by Eustis Engineering to verify the estimated pile load capacities presented in this report.

41. Existing load test data at Orleans Avenue Outfall Canal and Robert E. Lee Boulevard may be used to evaluate the installation of the probe piles and results of the dynamic pile tests at the London Avenue Canal. However, additional static load tests may be required depending on the pile type, embedments, and design loads selected. Once more information is available, Eustis Engineering should be contacted to evaluate the applicability of existing load test data.

42. Test piles should be allowed to set for at least 14 days subsequent to the installation of the reaction system. The test pile should then be load tested to failure in accordance with ASTM D 1143. The results of the static and dynamic pile load tests should be evaluated by Eustis Engineering to verify the estimated pile load capacities presented in this report.

GEOTECHNICAL SERVICES DURING CONSTRUCTION

43. To provide continuity between the investigation, design, and construction phases, Eustis Engineering should be retained to provide additional services which may include inspection and testing during fill placement, compaction and density testing of structural fill, concrete inspection and testing, inspection of piles and pile caps, performance of the test pile program and pile load tests, logging the driving of test piles and job piles, monitoring and evaluation of vibrations, and any other soil and materials testing services which will provide quality control

during construction and conformance to design specifications. Eustis Engineering can also provide the inspection and documentation of existing structures and obtain and evaluate settlement points to monitor construction.

44. Once the pile driving equipment is established, Eustis Engineering can perform dynamic analyses to evaluate driveability of the pile and driving stresses in the pile cross-section. Eustis Engineering can also perform DPT during installation and evaluate PDA data with respect to driving stresses, load capacity, and pile integrity.

45. If any construction problems arise, Eustis Engineering should be notified immediately so appropriate action can be taken. Such notification will permit the geotechnical engineer to be available quickly, evaluate unanticipated conditions, conduct additional tests, if required, and recommend alternative solutions to problems when necessary.

ORLEANS LEVEE DISTRICT
LONDON AVENUE CANAL
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA

SUMMARY OF I-WALL ANALYSES AND RESULTS

LOCATION	CASE	HIGH WATER LEVEL FEET NGVD	FACTOR OF SAFETY	REQUIRED SHEETPILE TIP ELEVATION NGVD	MAXIMUM BENDING MOMENT IN FT-KIPS PER FOOT OF WALL
MIRABEAU	Q	11.85	1.5	-25	29
		13.85	1.0	**-27**	**46**
	Penetration to Head Ratio*	11.85	—	-20	—
		13.85		-21	
FILMORE	Q	11.85	1.5	**-33**	**67**
		13.85	1.0	-31	98
	Penetration to Head Ratio*	11.85	—	-28	—
		13.85		-28	

Governing cases shown in bold.
* Ratio = 3:1 for SWL and 2.5:1 for SWL+2

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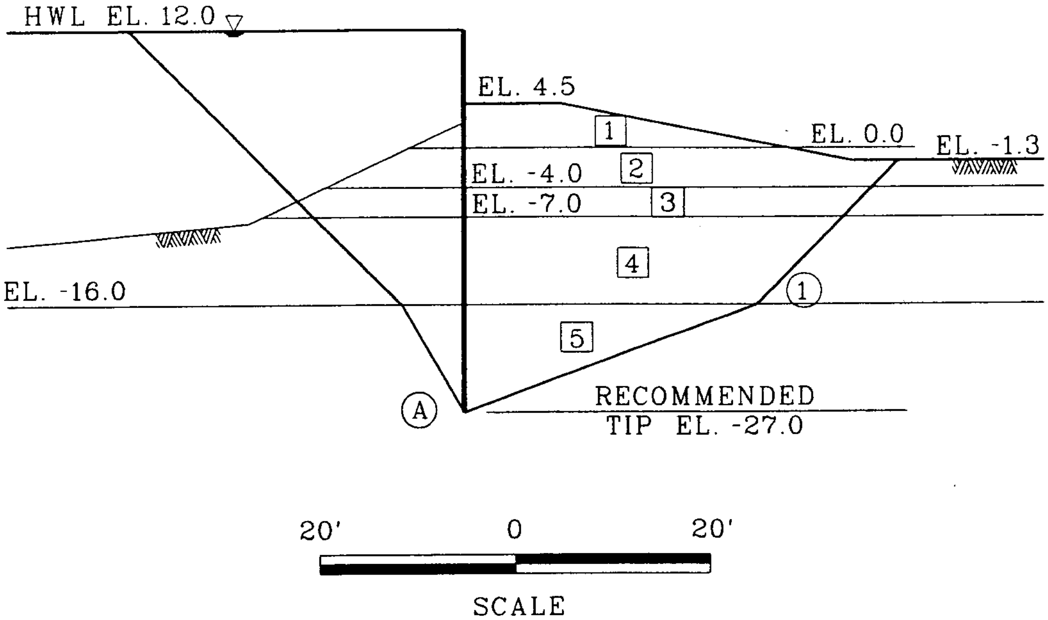
FIGURE 1



STRATUM NUMBER	UNIT WEIGHT (PCF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)
1	104	0	900
2	95	0	300
3	87	0	350
4	96	0	300
5	120	30	0

FAILURE SURFACE	RESISTING FORCES			DRIVING FORCES			FACTOR OF SAFETY
	R_A	R_B	R_P	D_A	D_B	D_P	
(A) (1)	12,851	16,583	9,081	59,418	27,500	10,992	1.84

SLOPE STABILITY AND I-WALL ANALYSES
MIRABEAU AVENUE
HIGH WATER

ORLEANS LEVEE DISTRICT
LONDON AVENUE CANAL
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA

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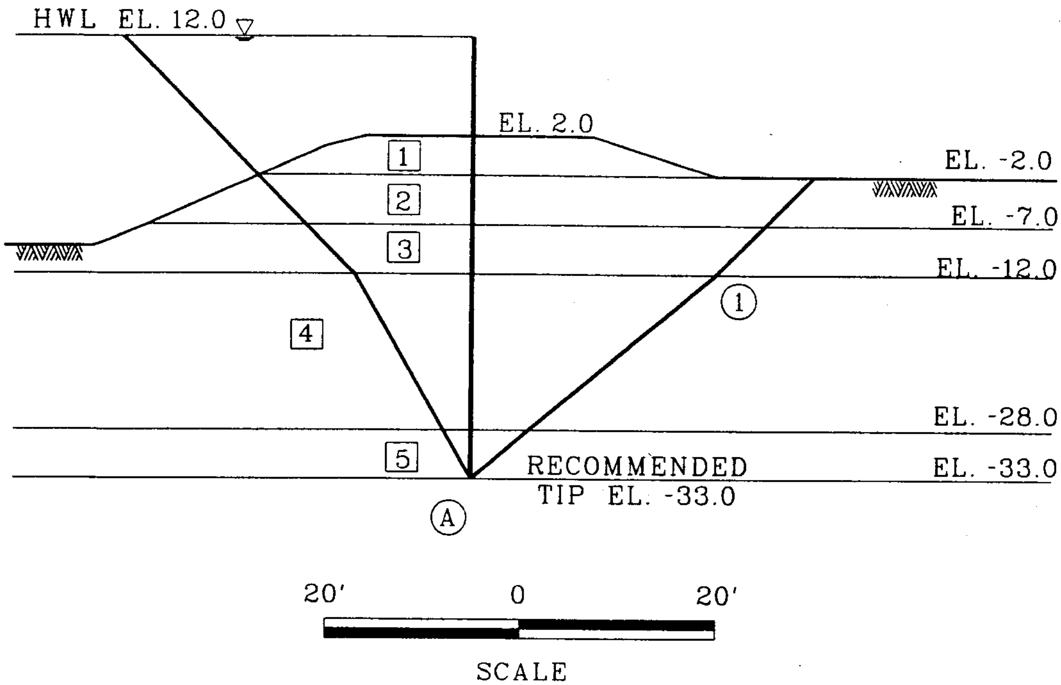
FIGURE 2

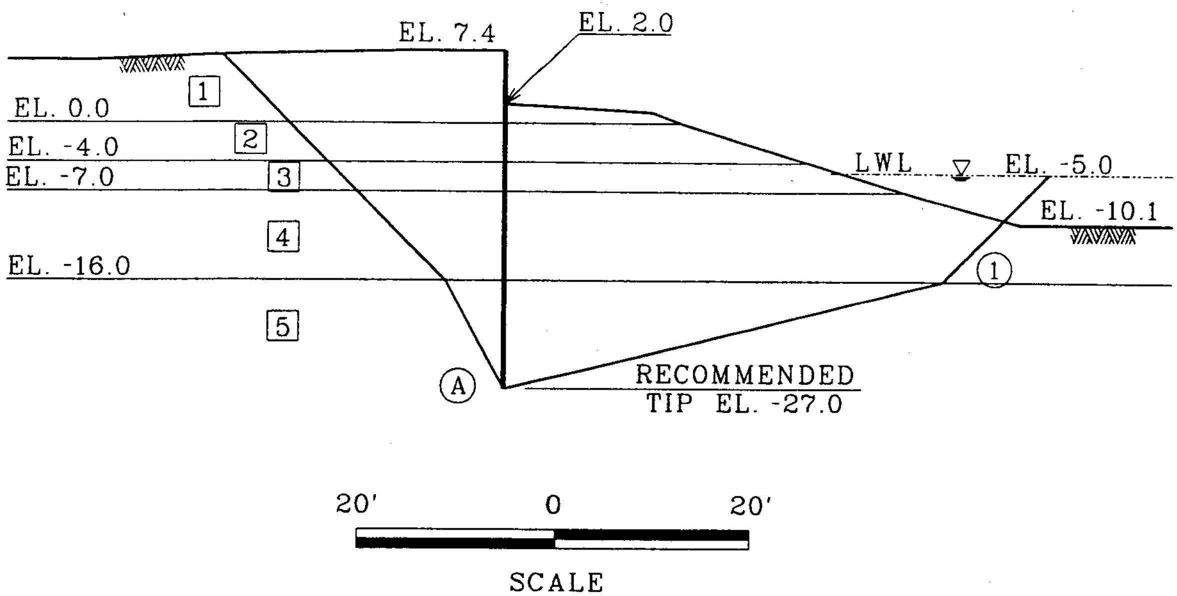


STRATUM NUMBER	UNIT WEIGHT (PCF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)
1	112	0	740
2	80	0	225
3	100	0	430
4	120	30	0
5	120	33	0

FAILURE SURFACE	RESISTING FORCES			DRIVING FORCES			FACTOR OF SAFETY
	R_A	R_B	R_P	D_A	D_B	D_P	
(A) (1)	21,501	14,861	6,479	87,423	52,416	4,187	1.39

SLOPE STABILITY AND I-WALL ANALYSES
FILMORE AVENUE
HIGH WATER

ORLEANS LEVEE DISTRICT
LONDON AVENUE CANAL
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA



STRATUM NUMBER	UNIT WEIGHT (PCF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)
1	104	0	900
2	95	0	300
3	87	0	350
4	96	0	300
5	120	30	0

FAILURE SURFACE	RESISTING FORCES			DRIVING FORCES			FACTOR OF SAFETY
	R_A	R_B	R_P	D_A	D_B	D_P	
A 1	30,542	19,944	3,799	58,574	21,810	4,622	1.69

SLOPE STABILITY ANALYSES
MIRABEAU AVENUE
LOW WATER

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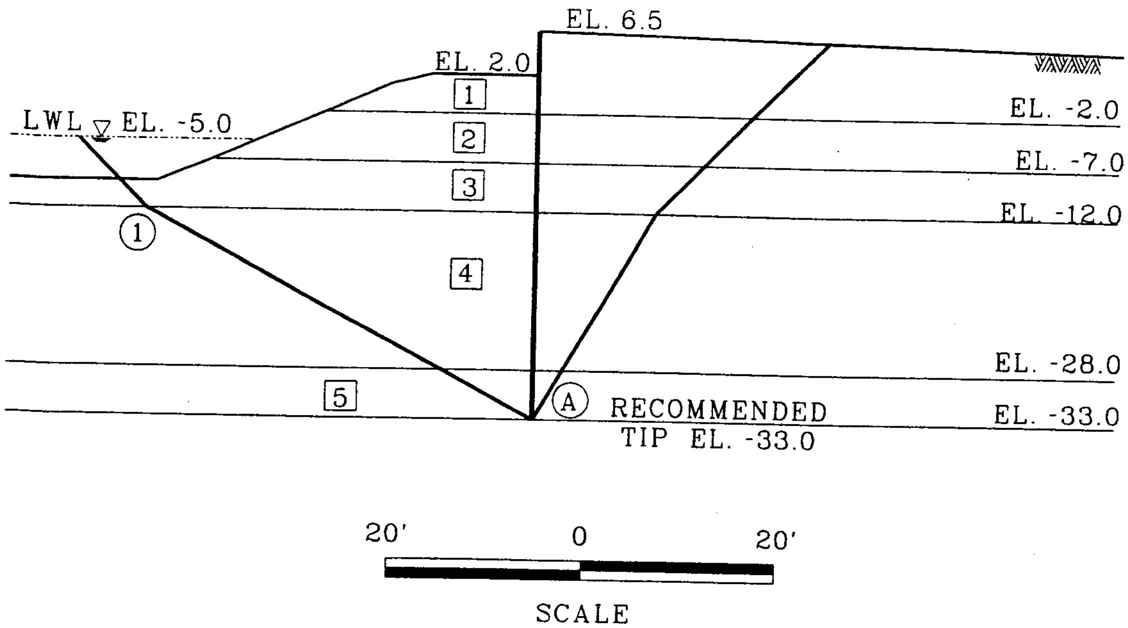
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA



STRATUM NUMBER	UNIT WEIGHT (PCF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)
1	112	0	740
2	80	0	225
3	100	0	430
4	120	30	0
5	120	33	0

FAILURE SURFACE	RESISTING FORCES			DRIVING FORCES			FACTOR OF SAFETY
	R_A	R_B	R_P	D_A	D_B	D_P	
(A) (1)	35,843	21,488	2,408	80,669	46,344	1,676	1.83

SLOPE STABILITY ANALYSES
FILMORE AVENUE
LOW WATER

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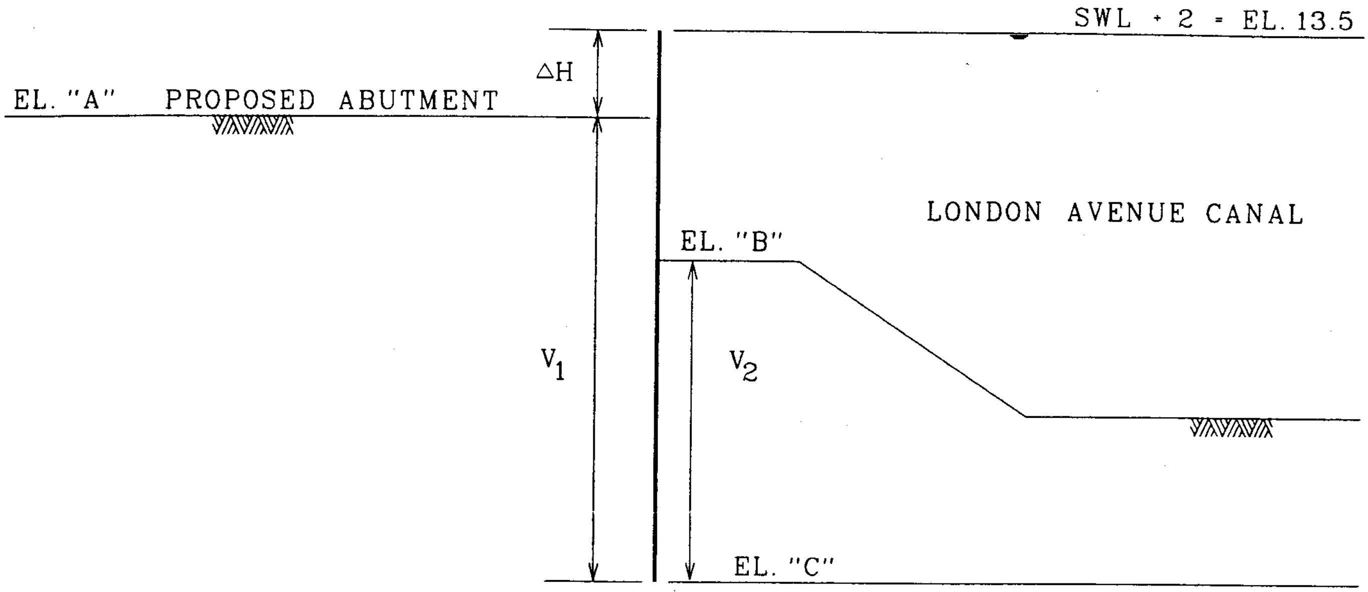
FIGURE 5



SCHEMATIC - NOT TO SCALE

LOCATION	EL. "A" (FT-NGVD)	EL. "B" (FT-NGVD)	EL. "C" (FT-NGVD)	ΔH (FEET)	H (FEET)	ΣV (FEET)	LANE'S WEIGHTED CREEP RATIO	
							ACTUAL	REQUIRED
MIRABEAU	7	2	-27	6.85	0	63	9	7
FILMORE	6	2	-33	7.85	0	74	9	7

$$LWCR = \frac{(1/3)H + \Sigma V}{\Delta H}$$

$$\Sigma V = (EL. "C" + EL. "A") + (EL. "C" + EL. "B")$$

SEEPAGE CUTOFF AT BRIDGE ABUTMENTS

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FIGURE 6

ORLEANS LEVEE DISTRICT
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ESTIMATED FULL ALLOWABLE PILE LOAD CAPACITIES
MIRABEAU AVENUE
MUDLINE EL -10 NGVD AT CANAL BENTS

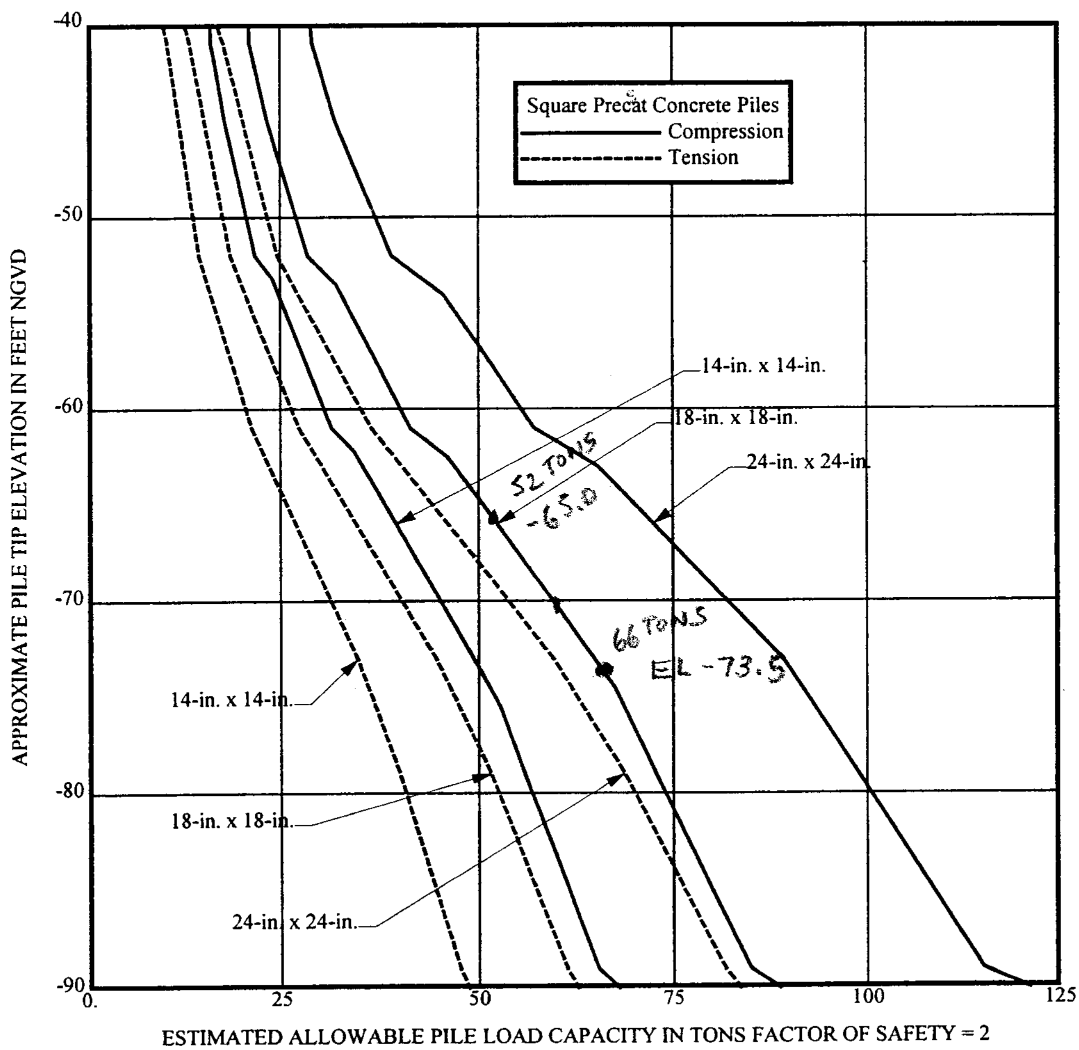

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FIGURE 7

ORLEANS LEVEE DISTRICT
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Partial Existing piles 30' deep 12 inch piles New piles 18" 2/3 18" = 12" 3/4 18" = 13.5"

ESTIMATED ~~FULL~~ ALLOWABLE PILE LOAD CAPACITIES
MIRABEAU AVENUE
MUDLINE EL -10 NGVD AT CANAL BENTS

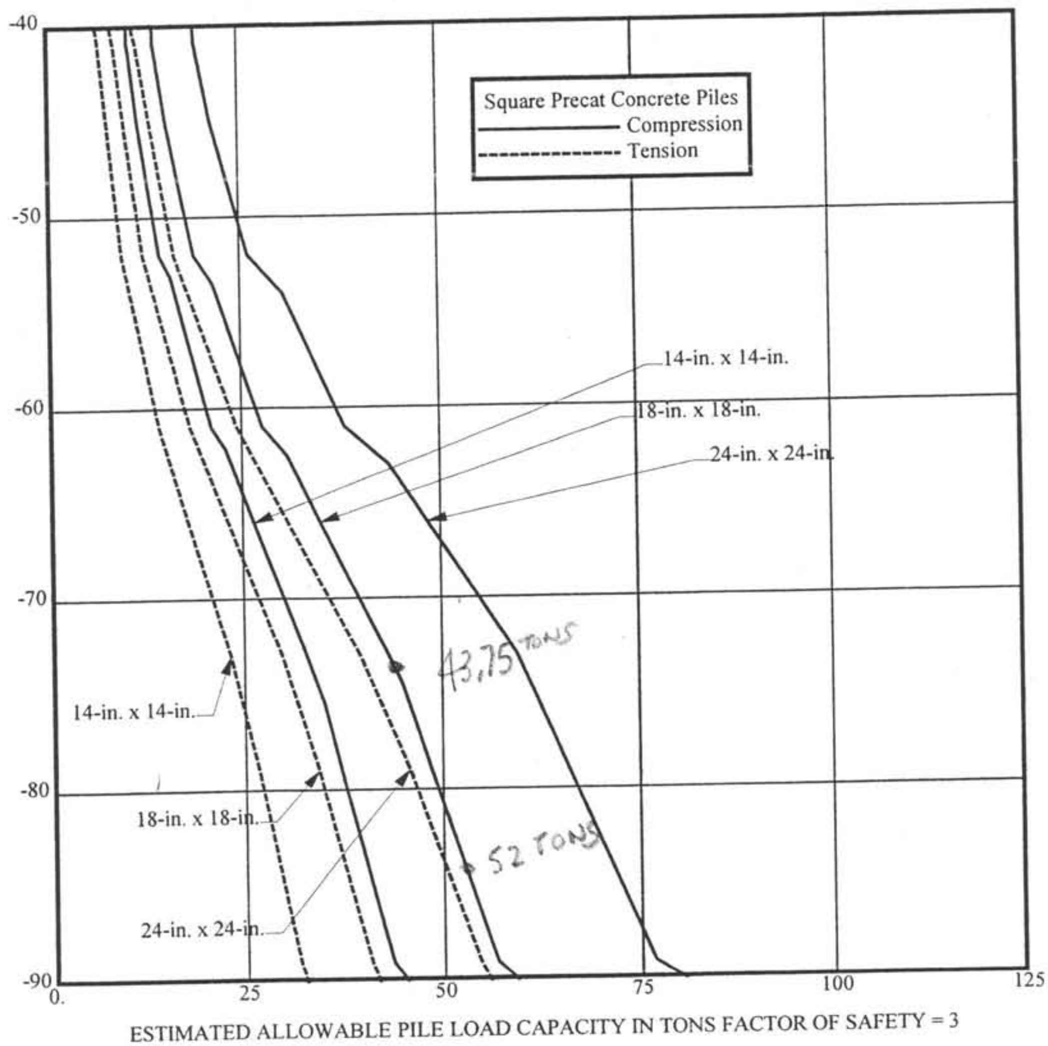

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FIGURE 8

ORLEANS LEVEE DISTRICT
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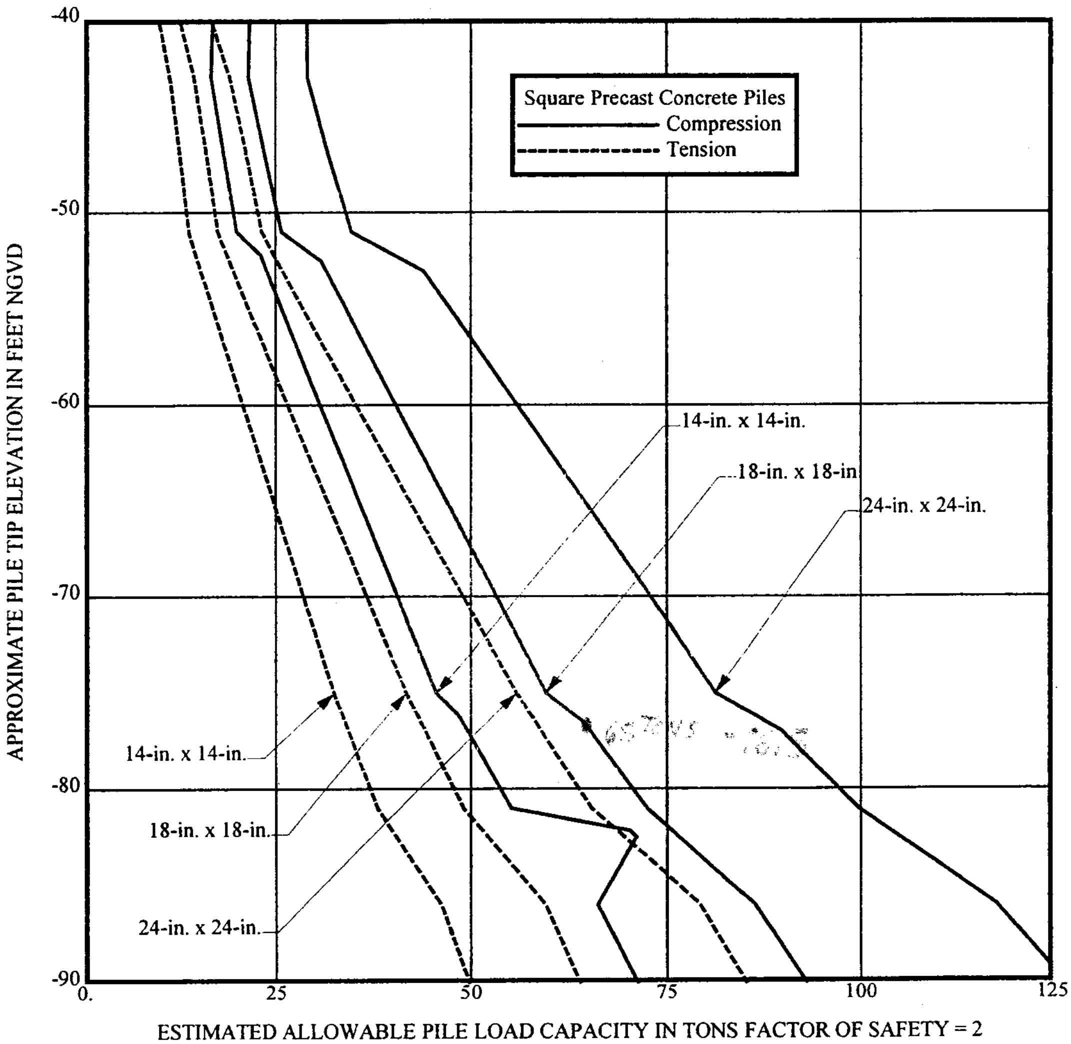
NEW ORLEANS, LOUISIANA

ESTIMATED FULL ALLOWABLE PILE LOAD CAPACITIES
FILMORE AVENUE
MUDLINE EL -9 NGVD AT CANAL BENTS



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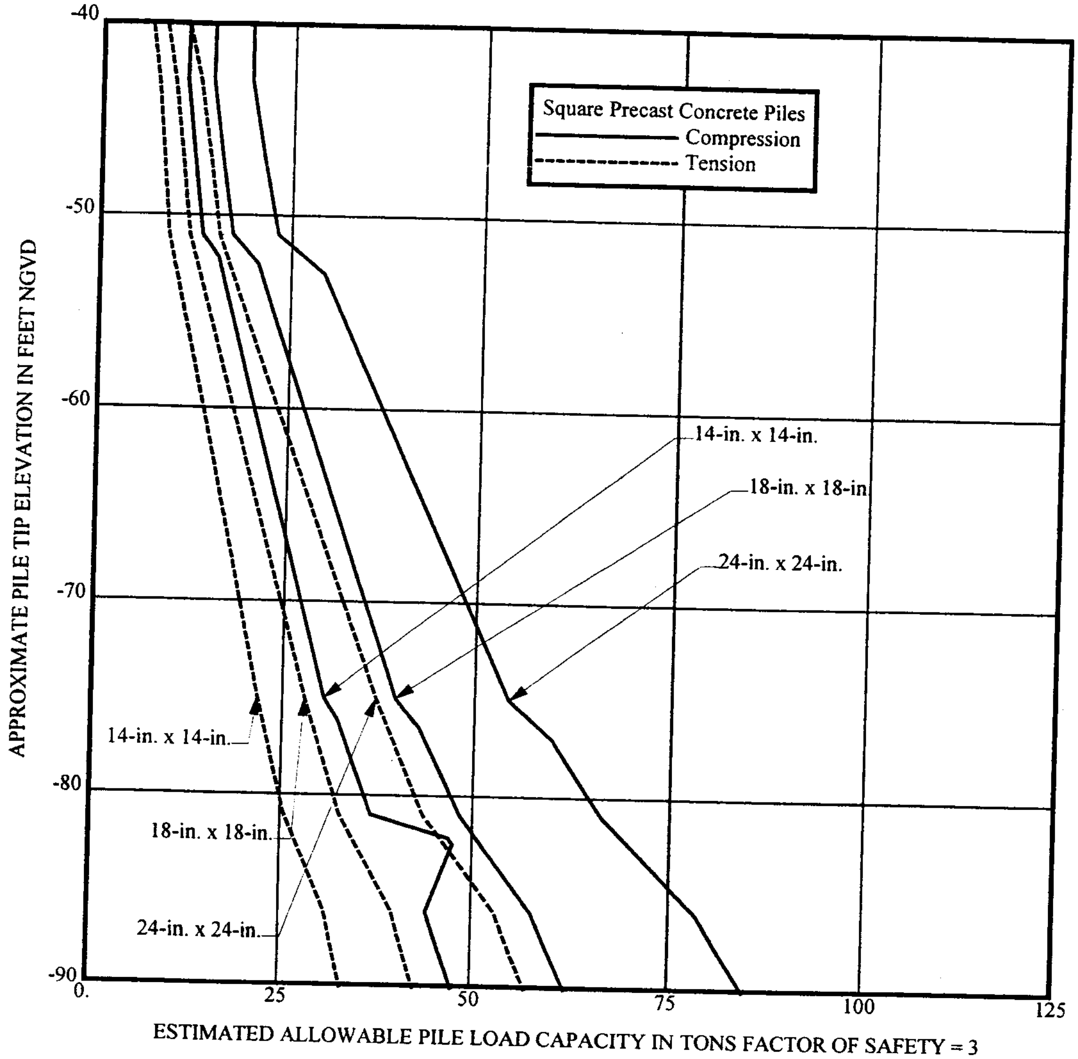
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA

ESTIMATED FULL ALLOWABLE PILE LOAD CAPACITIES
FILMORE AVENUE
MUDLINE EL -9 NGVD AT CANAL BENTS



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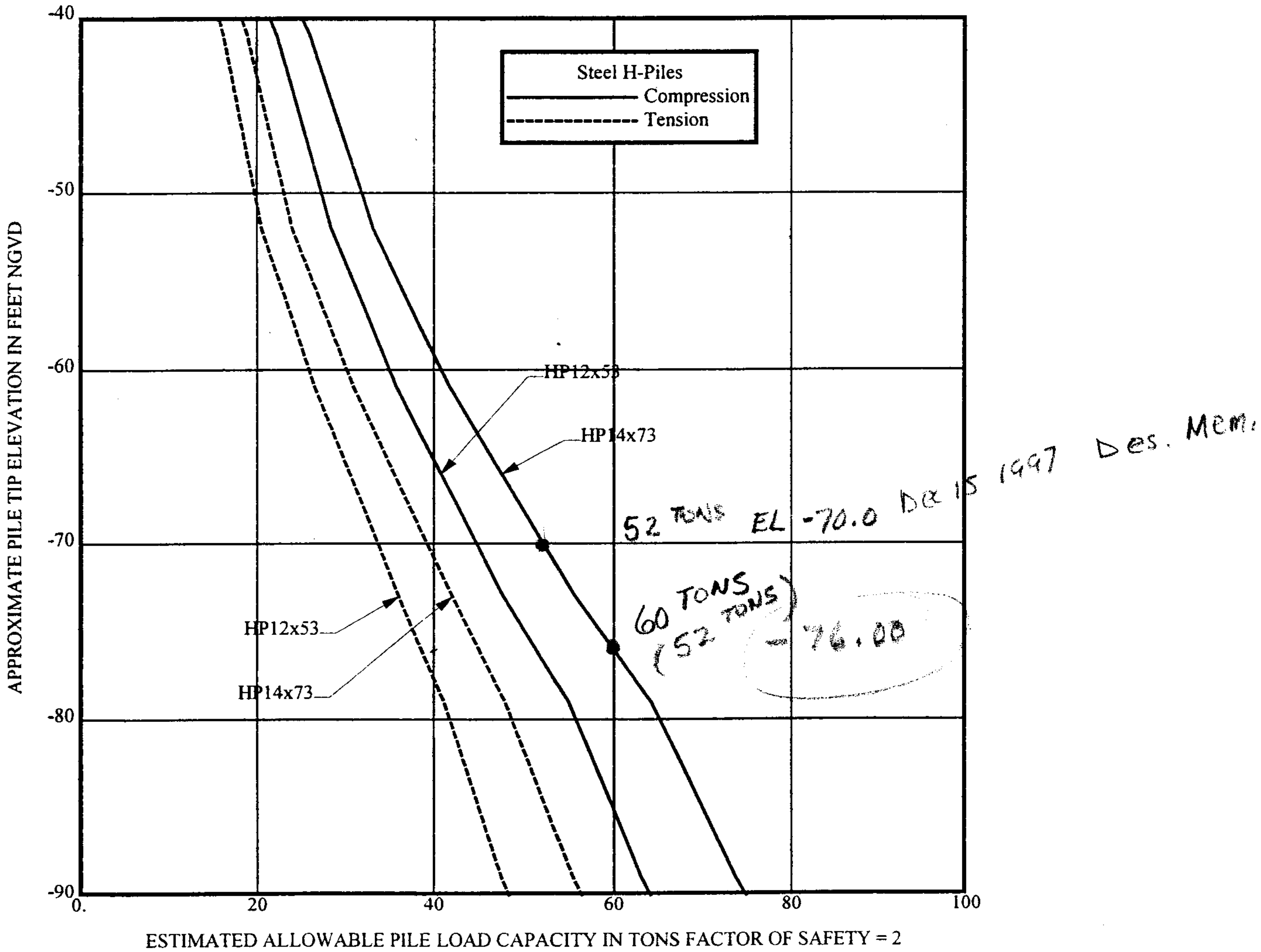
NEW ORLEANS, LOUISIANA

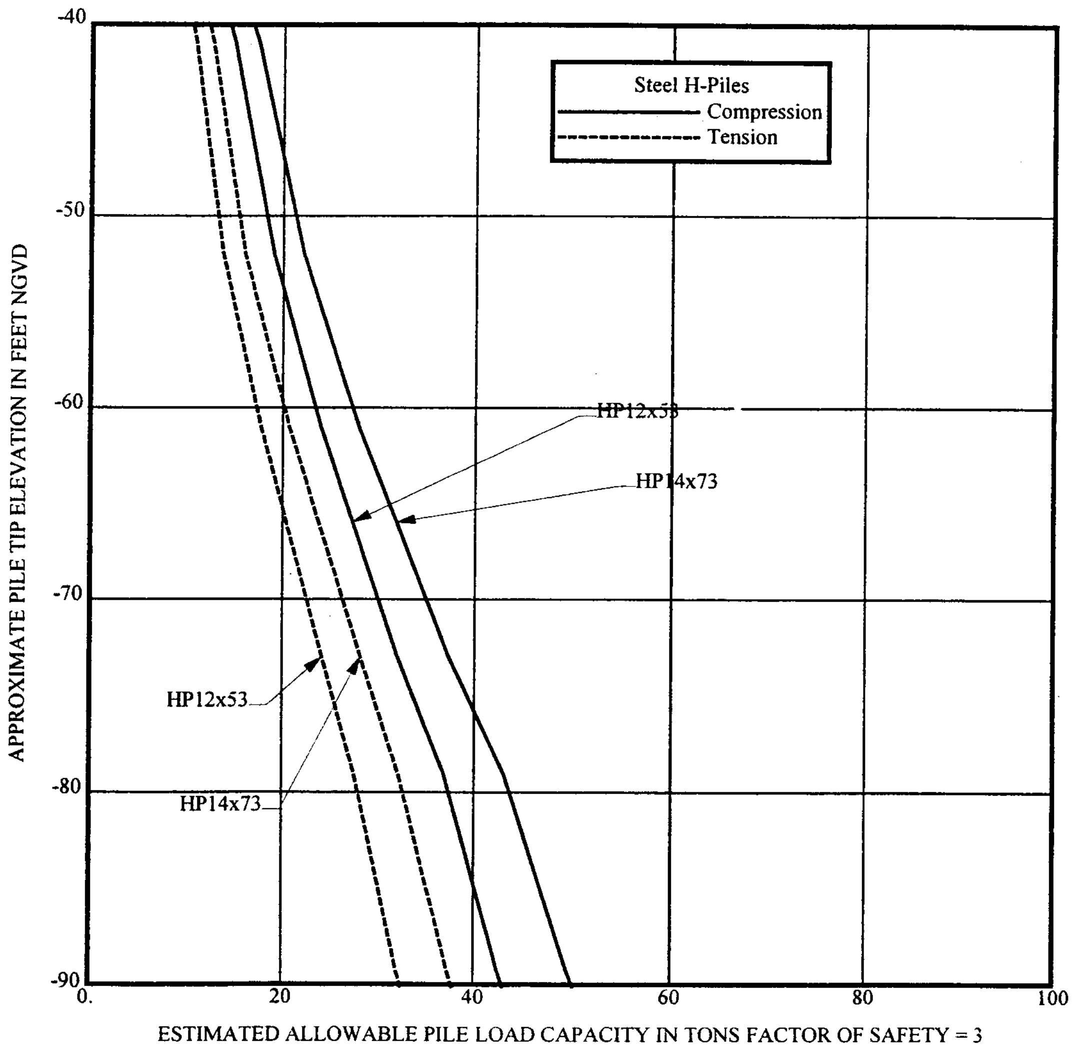
ESTIMATED FULL ALLOWABLE PILE LOAD CAPACITIES
MIRABEAU AVENUE
MUDLINE EL 3 NGVD AT ABUTMENTS



ORLEANS LEVEE DISTRICT
LONDON AVENUE CANAL
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA

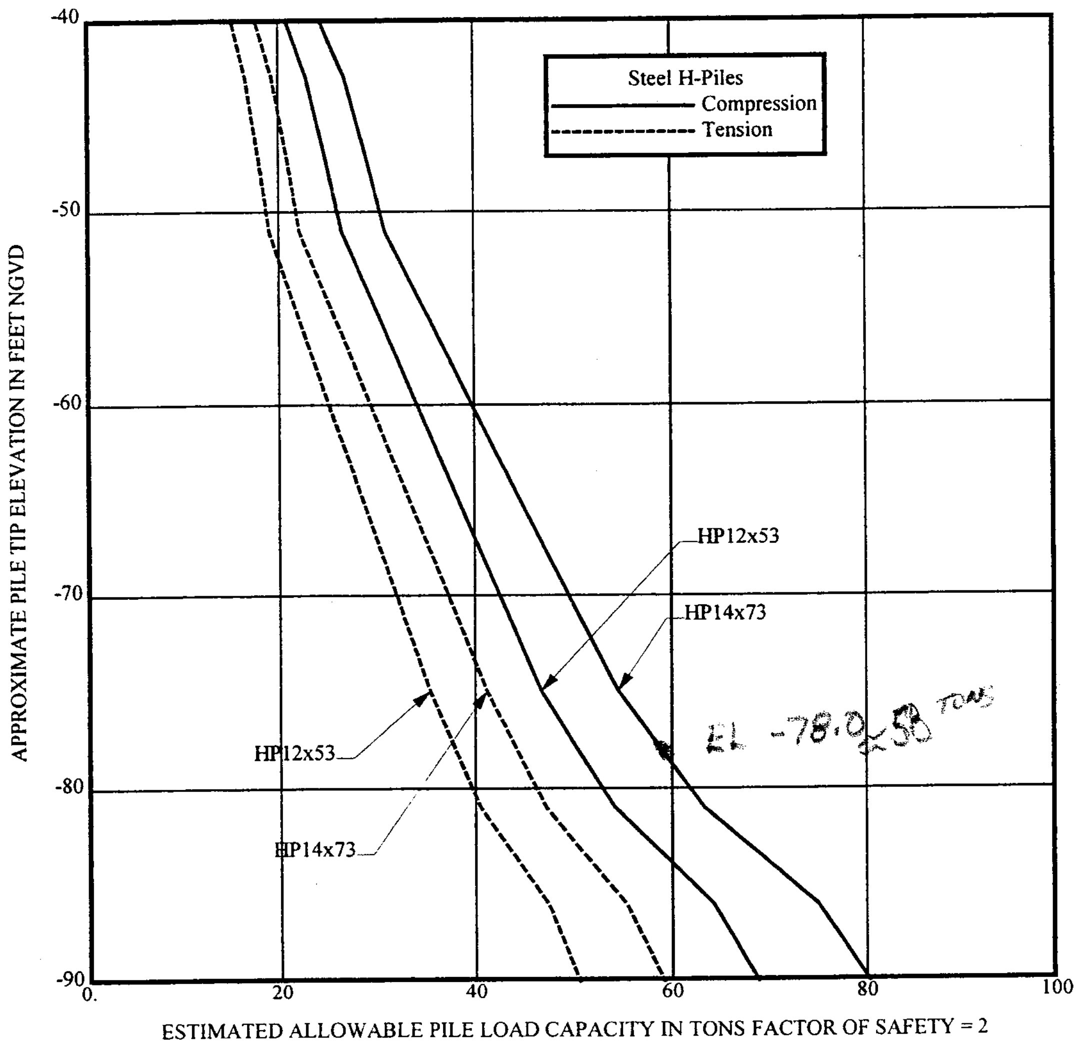
Driven next to existing piles.

ESTIMATED ~~FULL~~ ALLOWABLE PILE LOAD CAPACITIES
MIRABEAU AVENUE
MUDLINE EL 3 NGVD AT ABUTMENTS



ORLEANS LEVEE DISTRICT
LONDON AVENUE CANAL
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA

ESTIMATED FULL ALLOWABLE PILE LOAD CAPACITIES
FILMORE AVENUE
MUDLINE EL 2 NGVD AT ABUTMENTS

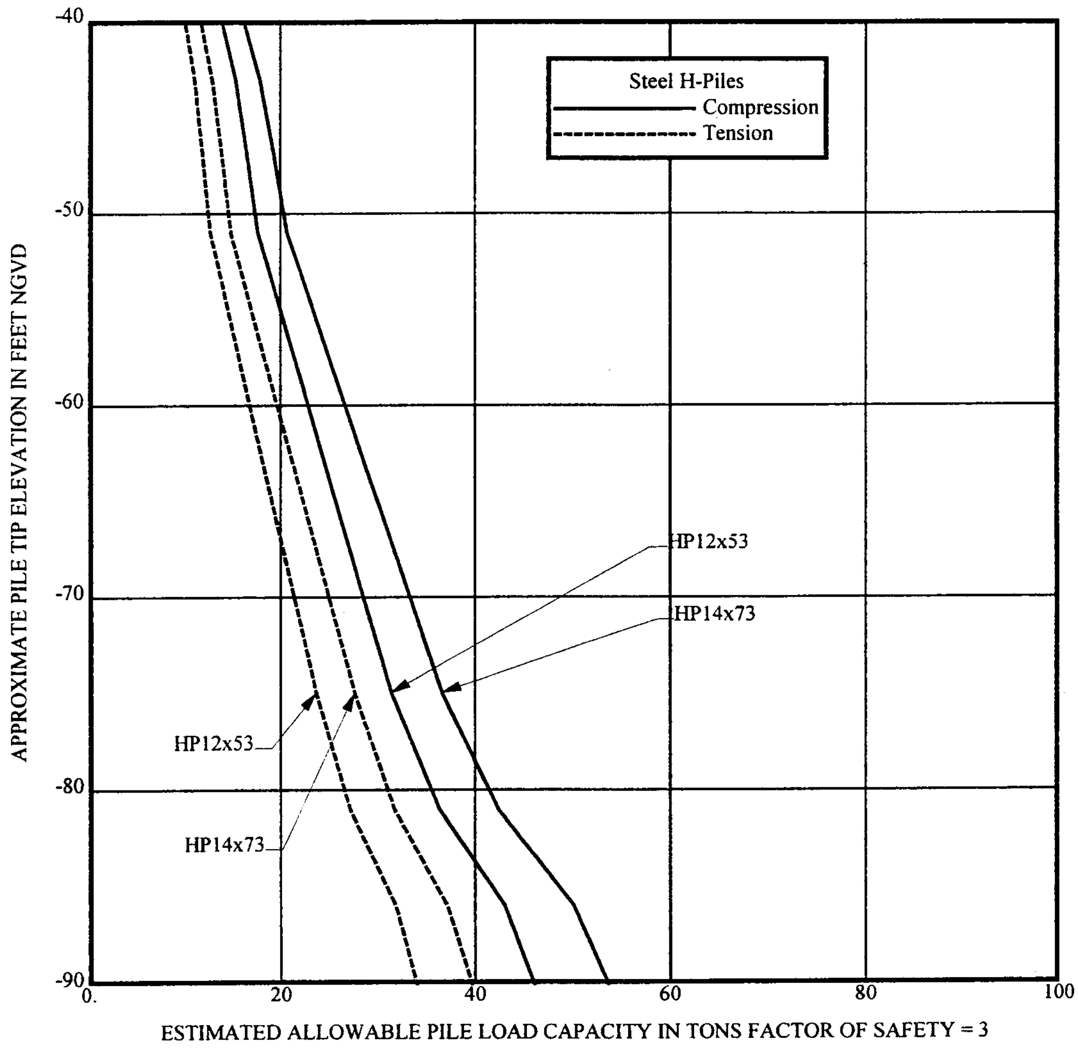


EUSTIS ENGINEERING COMPANY, INC.

FIGURE 13

ORLEANS LEVEE DISTRICT
LONDON AVENUE CANAL
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA

ESTIMATED FULL ALLOWABLE PILE LOAD CAPACITIES
FILMORE AVENUE
MUDLINE EL 2 NGVD AT ABUTMENTS

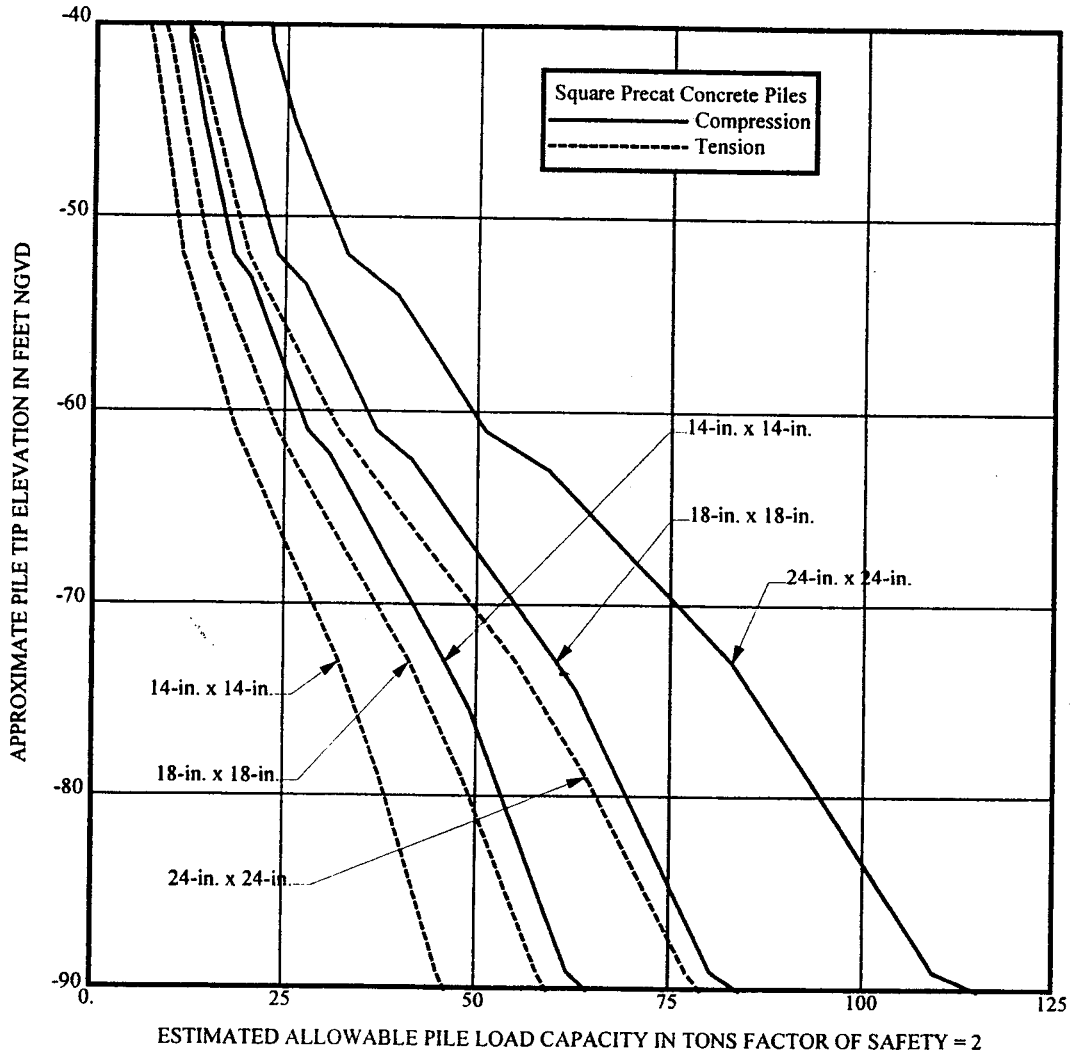


EUSTIS ENGINEERING COMPANY, INC.

FIGURE 14

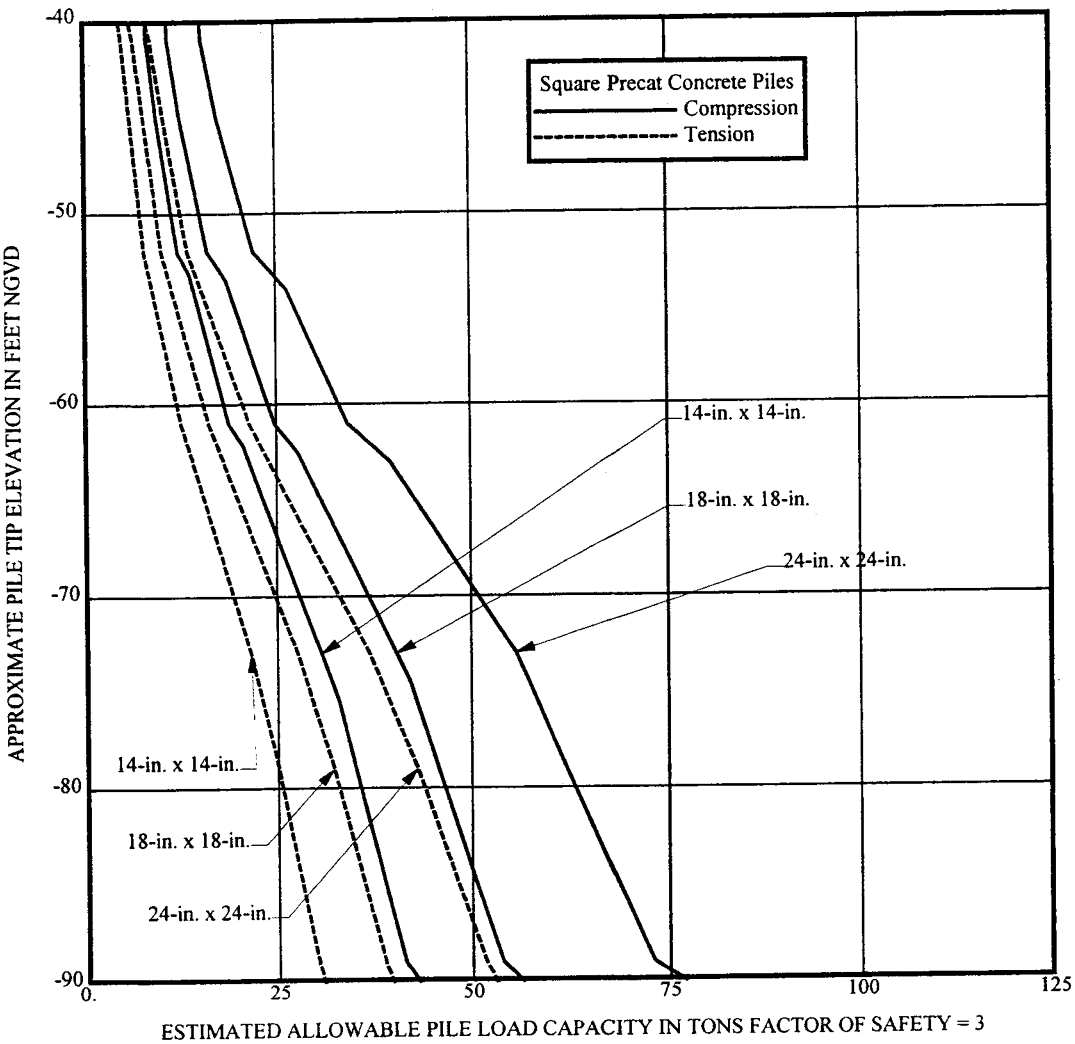
ORLEANS LEVEE DISTRICT
LONDON AVENUE CANAL
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA

ESTIMATED REDUCED ALLOWABLE PILE LOAD CAPACITIES
MIRABEAU AVENUE
MUDLINE EL -10 NGVD AT CANAL BENTS



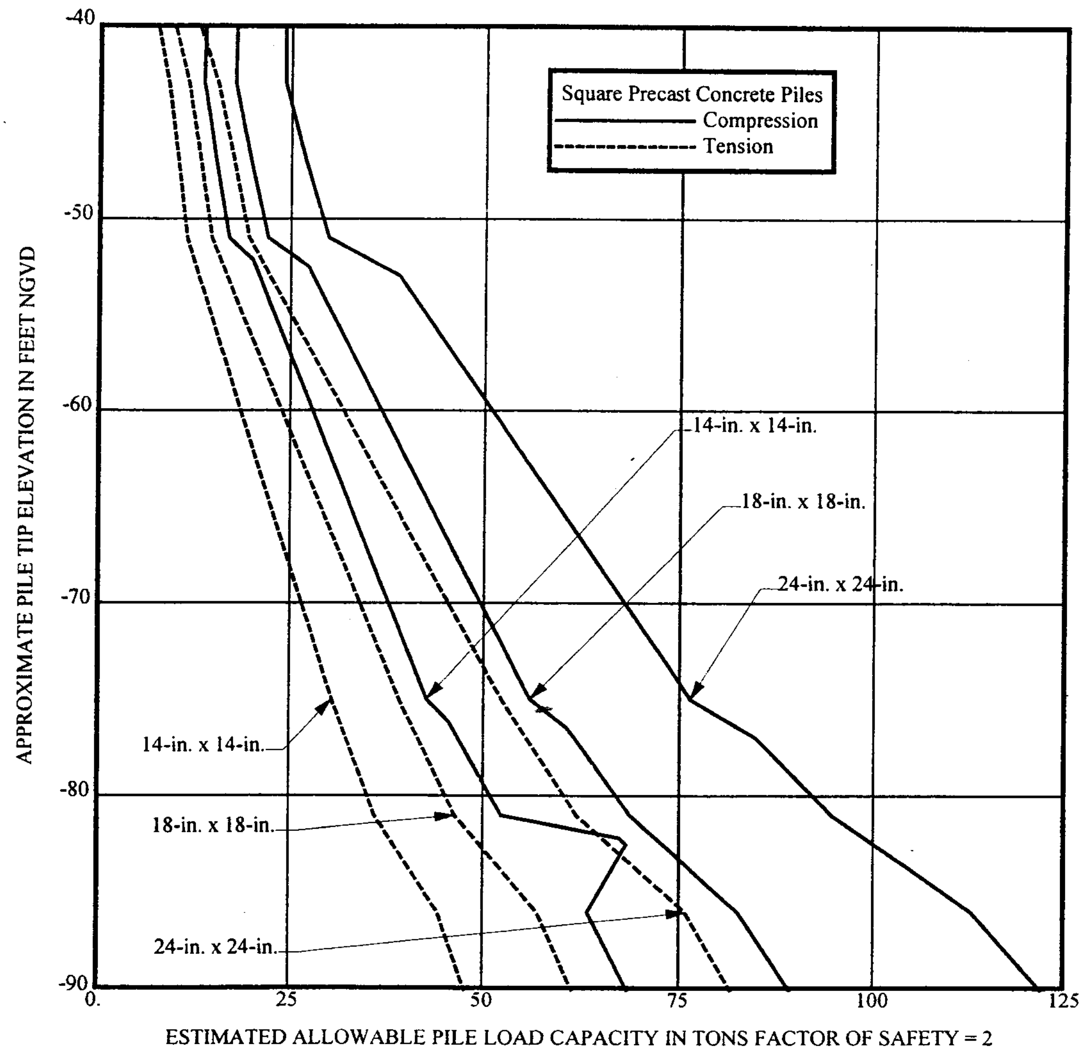
ORLEANS LEVEE DISTRICT
LONDON AVENUE CANAL
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA

ESTIMATED REDUCED ALLOWABLE PILE LOAD CAPACITIES
MIRABEAU AVENUE
MUDLINE EL -10 NGVD AT CANAL BENTS



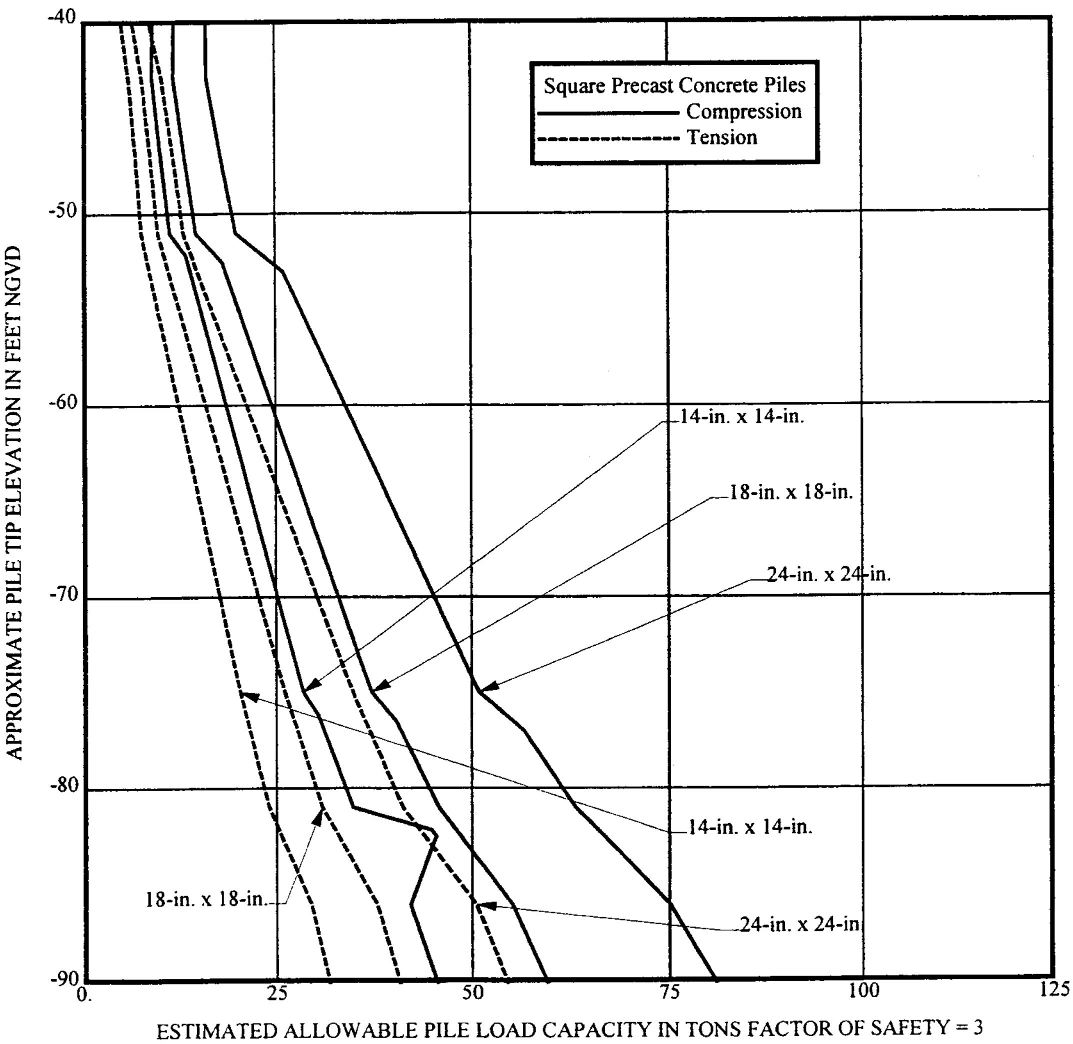
ORLEANS LEVEE DISTRICT
LONDON AVENUE CANAL
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA

ESTIMATED REDUCED ALLOWABLE PILE LOAD CAPACITIES
FILMORE AVENUE
MUDLINE EL -9 NGVD AT CANAL BENTS



ORLEANS LEVEE DISTRICT
LONDON AVENUE CANAL
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA

ESTIMATED REDUCED ALLOWABLE PILE LOAD CAPACITIES
FILMORE AVENUE
MUDLINE EL -9 NGVD AT CANAL BENTS



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.

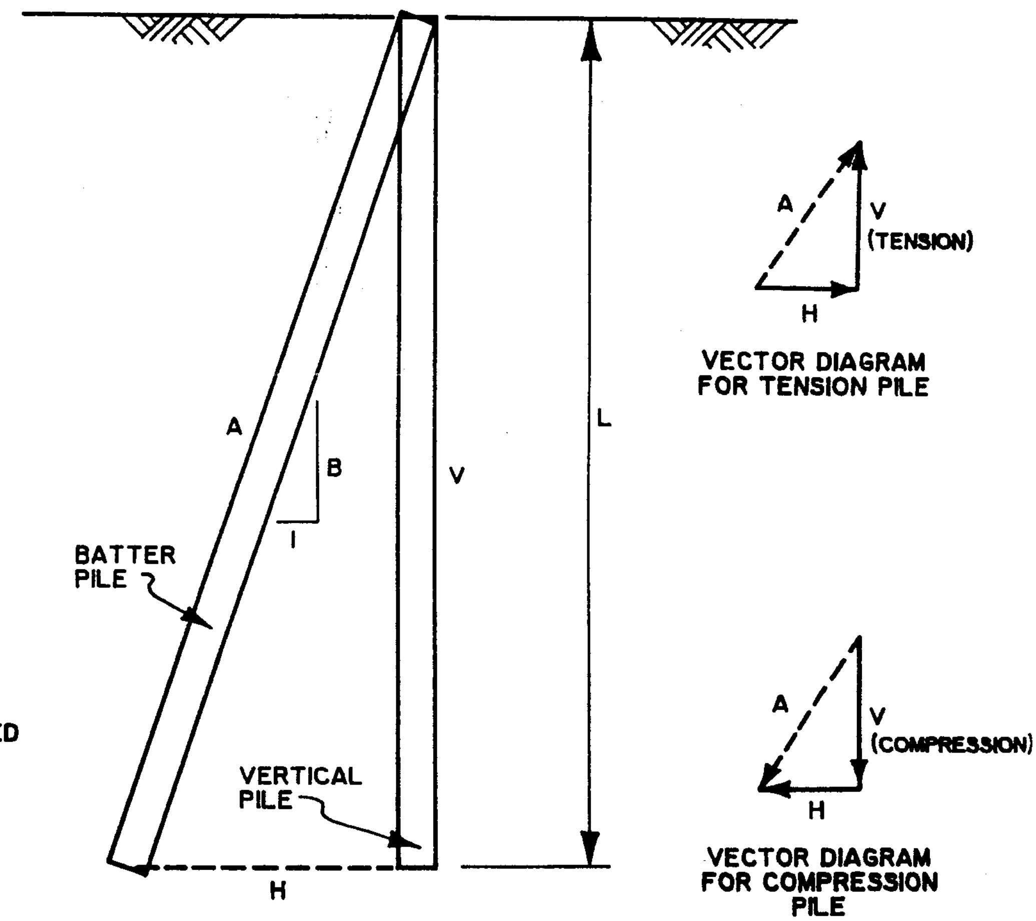
B = BATTER OF PILE EXPRESSED AS A RATIO OF VERTICAL DISTANCE TO ONE FOOT HORIZONTAL DISTANCE.

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.

ORLEANS LEVEE DISTRICT
LONDON AVENUE CANAL
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA

MODULUS OF HORIZONTAL SUBGRADE REACTION
MIRABEAU AVENUE

ELEVATION IN FEET NGVD	$(k_h \times B)/(C \times D)$
G.S.E.* to 0	400
0 to -4	133
-4 to -7	155
-7 to -16	133
-16 to -29	4,200
-29 to -34	9,200
-34 to -41	8,775
-41 to -45	4,100
-45 to -52	311
-52 to -61	366
-61 to -73	533
-73 to -79	25,500
-79 to -89	360
-89 to -95	533

* G.S.E. = Ground Surface Elevation (varies)

D	PILE SPACING IN DIRECTION OF LOADING
1.0	8B
0.85	7B
0.70	6B
0.55	5B
0.40	4B
0.25	3B

ORLEANS LEVEE DISTRICT
LONDON AVENUE CANAL
MIRABEAU AND FILMORE AVENUE BRIDGES
NEW ORLEANS, LOUISIANA

MODULUS OF HORIZONTAL SUBGRADE REACTION
FILMORE AVENUE

ELEVATION IN FEET NGVD	$(k_h \times B)/(C \times D)$
G.S.E.* to -2	329
-2 to -7	100
-7 to -12	191
-12 to -28	2,880
-28 to -32	14,400
-32 to -43	7,020
-43 to -47	2,400
-47 to -51	222
-51 to -75	378
-75 to -81	10,500
-81 to -86	39,500
-86 to -91	14,000
-91 to -95	444

* G.S.E. = Ground Surface Elevation (varies)

D	PILE SPACING IN DIRECTION OF LOADING
1.0	8B
0.85	7B
0.70	6B
0.55	5B
0.40	4B
0.25	3B

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.

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

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 WITHIN 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

APPENDIX I

LOG OF BORING

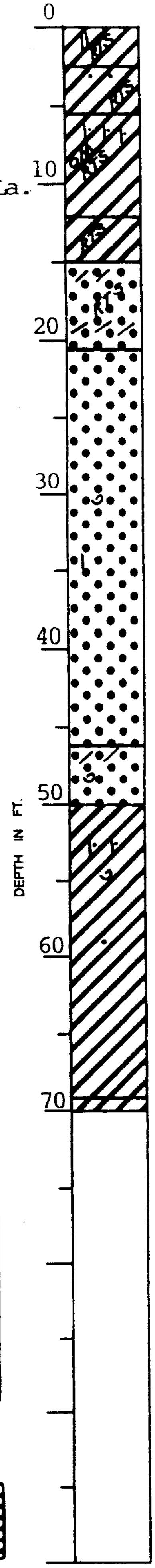
EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

Name of Project: London Avenue Canal, Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana
For: The Board of Levee Commissioners of the Orleans Levee District, New Orleans, La.
Burk & Associates, Inc., New Orleans, Louisiana

Boring No. 15 Soil Technician A. Croal, Jr. Date 17 October 1985

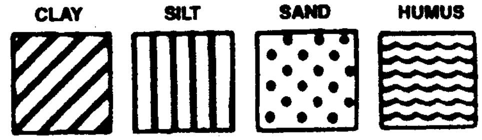
Ground Elev. ______ Datum ______ Gr. Water Depth See Text

Sample No.	SAMPLE Depth — Feet From	SAMPLE Depth — Feet To	DEPTH STRATUM Feet From	DEPTH STRATUM Feet To	VISUAL CLASSIFICATION	*STANDARD PENETRATION TEST	
1	1.7	2.5	0.0	2.5	Medium stiff tan & gray clay w/silt		
					pockets & grass roots		
2	4.7	5.5	2.5	5.5	Medium stiff tan & gray clay w/sand		
					pockets & roots		
3	7.7	8.5	5.5		Soft dark gray clay w/silty sand		
					layers, organic matter & roots		
4	10.7	11.5	\	12.0	Soft dark gray clay w/organic matter		
					& roots		
5	13.7	14.5	12.0	15.0	Soft gray clay w/roots		
6	15.5	17.0	15.0		Medium dense gray fine sand w/clay	3	20
					pockets & roots		
7	18.0	19.5		20.5	Medium dense gray fine sand w/clay	10	14
					layers		
8	20.5	22.0	20.5		Very dense gray fine sand	50=6" (Seat)	
9	23.5	25.0			Ditto	25	50=8"
10	28.5	30.0			Ditto	20	50=6"
11	33.5	35.0			Very dense gray fine sand w/few shell	18	50=8"
					fragments & trace of silt		
12	38.5	40.0			Very dense gray fine sand	23	50=6"
13	43.5	45.0		46.0	Very dense gray fine sand w/few shell	2	50=10"
					fragments		
14	48.5	50.0	46.0	50.0	Medium dense gray fine sand w/clay	3	12
					pockets & shell fragments		
15	53.2	54.0	50.0		Medium stiff gray clay w/silty sand		
					pockets & few shell fragments		
					(Continued)		



*Number in first column indicates number of blows of 140-lb. hammer dropped 30 in. required to seat 2-in. O. D. splitspoon sampler 6 in. Number in second column indicates number of blows of 140-lb. hammer dropped 30 in. required to drive 2-in. O. D. splitspoon sampler 1 ft. after seating 6 in.

WHILE THIS LOG OF BORING IS CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT ITS RESPECTIVE LOCATION ON THE DATE SHOWN, IT IS NOT WARRANTED THAT IT IS REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.



Remarks: ______

Predominant type shown heavy. Modifying type shown light.

LOG OF BORING

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

Name of Project: London Avenue Canal, Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District, New Orleans, La.
Burk & Associates, Inc., New Orleans, Louisiana

Boring No. 15 (Cont'd) Soil Technician A. Croal, Jr. Date 17 October 1985

Ground Elev. ______ Datum ______ Gr. Water Depth See Text

Sample No.	SAMPLE Depth — Feet From	SAMPLE Depth — Feet To	DEPTH STRATUM Feet From	DEPTH STRATUM Feet To	VISUAL CLASSIFICATION	*STANDARD PENETRATION TEST	
16	58.2	59.0			Medium stiff gray clay w/few silty sand pockets & few shell fragments		
17	63.2	64.0		66.0	Medium stiff gray clay w/shell fragments		
18	68.2	69.0	66.0	69.0	Stiff gray clay w/shell fragments & trace of sand		
19	69.5	70.0	69.0	70.0	Stiff green clay		

DEPTH IN FT.

*Number in first column indicates number of blows of 140-lb. hammer dropped 30 in. required to seat 2-in. O. D. splitspoon sampler 6 in. Number in second column indicates number of blows of 140-lb. hammer dropped 30 in. required to drive 2-in. O. D. splitspoon sampler 1 ft. after seating 6 in.

WHILE THIS LOG OF BORING IS CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT ITS RESPECTIVE LOCATION ON THE DATE SHOWN, IT IS NOT WARRANTED THAT IT IS REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

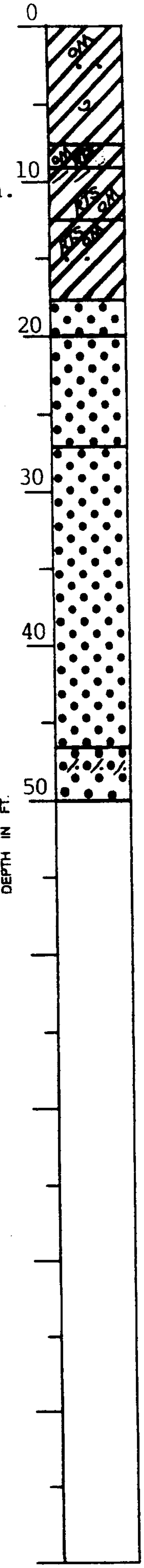
Remarks: ______

CLAY SILT SAND HUMUS

Predominant type shown heavy. Modifying type shown light.

LOG OF BORING

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.



Name of Project: London Avenue Canal, Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District, New Orleans, La.
Burk & Associates, Inc., New Orleans, Louisiana

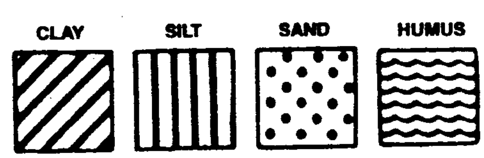
Boring No. 18 **Soil Technician** George Hardee **Date** 25 October 1985

Ground Elev. ______ **Datum** ______ **Gr. Water Depth** See Text

Sample No.	SAMPLE Depth — Feet From	SAMPLE Depth — Feet To	DEPTH STRATUM Feet From	DEPTH STRATUM Feet To	VISUAL CLASSIFICATION	*STANDARD PENETRATION TEST	
1	1.5	2.5	0.0		Very stiff gray clay w/organic matter		
					& sand pockets		
2	5.0	5.5		7.5	Stiff gray clay w/organic matter, sand		
					pockets & shells		
3	8.0	8.5	7.5	9.0	Medium stiff brown & gray clay w/roots		
					& organic matter		
4	10.5	11.5	9.0	12.5	Soft brown & gray clay w/clay pockets,		
					roots & wood		
5	13.5	14.5	12.5		Soft gray clay w/roots & organic matter		
6	16.5	17.5		17.5	Soft gray clay w/sand pockets		
7	17.5	19.0	17.5	20.0	Dense gray fine sand	11	42
8	20.5	22.0	20.0		Medium dense gray fine sand	3	18
9	23.5	25.0		27.0	Ditto	5	25
10	28.5	30.0	27.0		Dense gray fine sand	12	50
11	33.5	35.0			Ditto	16	50
12	38.5	40.0			Ditto	11	30
13	43.5	45.0		46.5	Ditto	5	32
14	48.5	50.0	46.5	50.0	Loose gray fine sand w/clayey sand	3	9
					layers		

*Number in first column indicates number of blows of 140-lb. hammer dropped 30 in. required to seat 2-in. O. D. splitspoon sampler 6 in. Number in second column indicates number of blows of 140-lb. hammer dropped 30 in. required to drive 2-in. O. D. splitspoon sampler 1 ft. after seating 6 in.

WHILE THIS LOG OF BORING IS CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT ITS RESPECTIVE LOCATION ON THE DATE SHOWN, IT IS NOT WARRANTED THAT IT IS REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.



Remarks: ______

Predominant type shown heavy. Modifying type shown light.

LOG OF BORING

EUSTIS ENGINEERING COMPANY

SOIL AND FOUNDATION CONSULTANTS

METAIRIE, LA.

Name of Project: London Avenue Canal, Levee and Floodwall Improvements

Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana

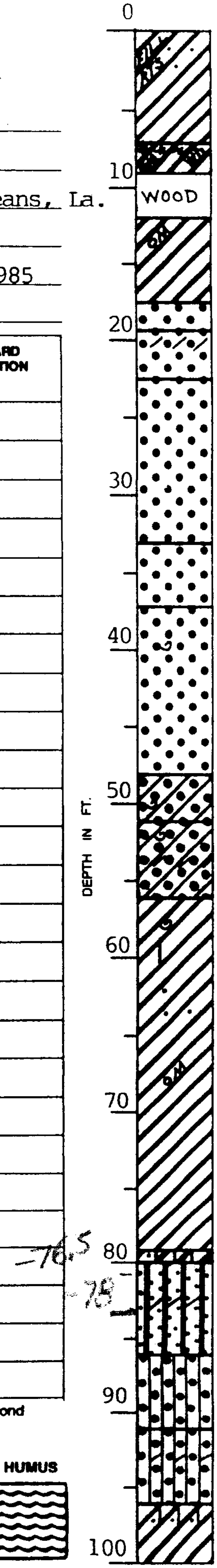
For: The Board of Levee Commissioners of the Orleans Levee District, New Orleans, La.

Burk & Associates, Inc., New Orleans, Louisiana

Boring No. 19 **Soil Technician** A. J. Mayeux **Date** 18 October 1985

Ground Elev. ________ **Datum** ________ **Gr. Water Depth** See Text

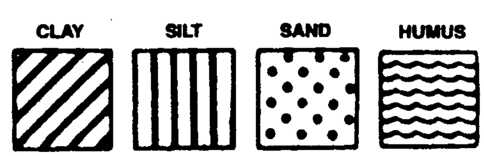
Sample No.	Sample Depth — Feet From	Sample Depth — Feet To	Depth Stratum Feet From	Depth Stratum Feet To	Visual Classification	*Standard Penetration Test	
			0.0		Stiff gray & tan clay w/fill		
1	2.0	2.5			Stiff gray & tan clay w/sand pockets		
					& roots		
2	5.0	5.5		7.0	Medium stiff gray & tan clay w/sand		
					lenses, pockets & roots		
3	8.0	8.5	7.0	9.0	Very soft gray clay w/organic clay		
					layers, wood & organic matter		
	11.0	11.5	9.0	12.0	Wood w/organic matter & clay		
4	14.0	14.5	12.0	17.5	Soft gray clay w/some organic matter		
5	17.5	19.0	17.5	19.5	Medium dense gray sand	5	21
6	20.0	21.5	19.5	22.5	Loose gray sand w/clay layers	3	9
7	22.5	24.0	22.5		Medium dense gray sand	3	23
8	25.0	26.5			Ditto	2	24
9	28.5	30.0		33.0	Ditto	6	21
10	33.5	35.0	33.0	37.0	Dense gray sand	13	40
11	38.5	40.0	37.0		Medium dense gray sand w/shell	7	21
					fragments		
12	43.5	45.0		48.0	Ditto	4	22
13	48.5	50.0	48.0	51.0	Very loose gray clayey sand w/shell	2	5
					fragments		
14	53.5	55.0	51.0	56.0	Very loose gray clayey sand w/shell	1	3
					fragments & clay layers		
15	58.5	60.0	56.0		Medium stiff gray clay w/shell	3	7
					fragments & silt lenses		
16	64.0	64.5			Medium stiff gray clay w/trace of sand		
17	69.0	69.5			Stiff gray clay w/sand pockets		



*Number in first column indicates number of blows of 140-lb. hammer dropped 30 in. required to seat 2-in. O. D. splitspoon sampler 6 in. Number in second column indicates number of blows of 140-lb. hammer dropped 30 in. required to drive 2-in. O. D. splitspoon sampler 1 ft. after seating 6 in.

WHILE THIS LOG OF BORING IS CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT ITS RESPECTIVE LOCATION ON THE DATE SHOWN, IT IS NOT WARRANTED THAT IT IS REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Remarks: ________



Predominant type shown heavy. Modifying type shown light.

LOG OF BORING

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

Name of Project: London Avenue Canal, Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District, New Orleans, La.
Burk & Associates, Inc., New Orleans, Louisiana

Boring No. 19 (Cont'd) **Soil Technician** A. J. Mayeux **Date** 18 October 1985

Ground Elev. ______ **Datum** ______ **Gr. Water Depth** See Text

Sample No.	Sample Depth — Feet From	Sample Depth — Feet To	Depth Stratum Feet From	Depth Stratum Feet To	Visual Classification	*Standard Penetration Test	
18	74.0	74.5		79.0	Stiff gray clay w/some organic matter & shell fragments		
19	79.0	79.5	79.0	80.0	Medium stiff greenish-gray silty clay w/fine sand		
20	80.0	81.5	80.0		Medium compact greenish-gray sandy silt	6	19
21	82.5	84.0	\	86.0	Medium compact greenish-gray sandy silt w/clay layers	7	10
22	88.5	90.0	86.0	91.0	Dense gray silty sand	10	42
23	93.5	95.0	91.0	96.0	Medium dense gray silty sand w/clay layers	7	17
24	98.5	100.0	96.0	100.0	Medium stiff gray clay w/sandy silt layers	5	12

DEPTH IN FT.

*Number in first column indicates number of blows of 140-lb. hammer dropped 30 in. required to seat 2-in. O. D. splitspoon sampler 6 in. Number in second column indicates number of blows of 140-lb. hammer dropped 30 in. required to drive 2-in. O. D. splitspoon sampler 1 ft. after seating 6 in.

WHILE THIS LOG OF BORING IS CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT ITS RESPECTIVE LOCATION ON THE DATE SHOWN, IT IS NOT WARRANTED THAT IT IS REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Remarks: ______

CLAY SILT SAND HUMUS

Predominant type shown heavy. Modifying type shown light.

LOG OF BORING

EUSTIS ENGINEERING COMPANY Sheet 1 of 2

SOIL AND FOUNDATION CONSULTANTS

METAIRIE, LA.

Name of Project: London Avenue Canal, Levee and Floodwall Improvements

Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana

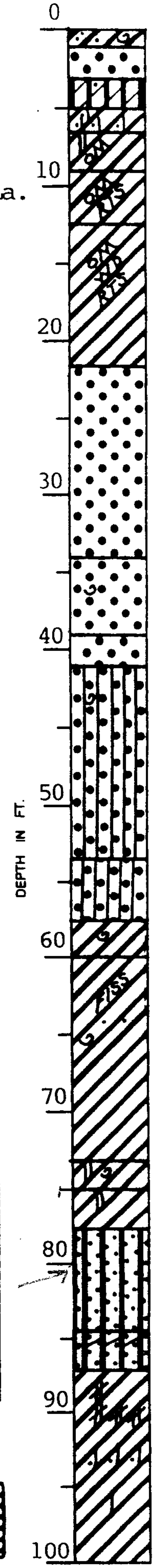
For: The Board of Levee Commissioners of the Orleans Levee District, New Orleans, La.

Burk & Associates, Inc., New Orleans, Louisiana

Boring No. 50 **Soil Technician** A. Croal, Jr. **Date** 13 November 1985

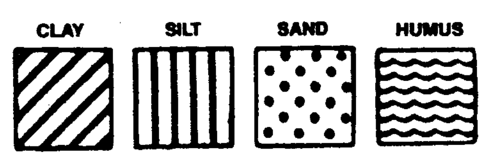
Ground Elev. ______ **Datum** ______ **Gr. Water Depth** See Text

Sample No.	SAMPLE Depth — Feet From	SAMPLE Depth — Feet To	DEPTH STRATUM Feet From	DEPTH STRATUM Feet To	VISUAL CLASSIFICATION	*STANDARD PENETRATION TEST	
1	0.0	0.5	0.0	1.0	Very stiff gray & brown clay w/fine		
					sand lenses, pockets & shell		
					fragments		
2	1.7	2.5	1.0	3.0	Loose tan fine sand		
3	2.5	4.0	3.0	5.0	Medium compact brown & gray clayey silt	3	14
					w/fine sand lenses		
4	5.0	6.5	5.0	6.5	Medium stiff to stiff gray clay w/sandy	2	8
					silt lenses & layers		
5	8.2	9.0	6.5	9.0	Soft dark gray clay w/silt pockets &		
					trace of organic matter		
6	10.7	11.5	9.0	12.5	Soft dark gray clay w/organic matter		
					& roots		
7	13.7	14.5	12.5		Very soft gray clay w/organic matter		
					& wood		
8	18.2	19.0		21.8	Soft gray clay w/organic matter & roots		
9	21.7	22.5	21.8		Loose to medium dense gray fine sand		
10	22.5	24.0			Medium dense gray fine sand	4	26
11	25.0	26.5			Ditto	4	23
12	27.5	29.0			Ditto	3	19
13	30.0	31.5		34.0	Ditto	8	25
14	33.5	35.0	34.0	39.0	Dense gray fine sand w/shell fragments	9	32
15	38.5	40.0	39.0	41.0	Very dense gray fine sand	12	50=11"
16	43.5	45.0	41.0		Medium dense gray silty sand w/few	6	26
					shell fragments		
17	48.5	50.0		53.5	Medium dense gray silty sand	5	27
18	53.5	55.0	53.5	57.5	Loose gray silty sand	3	10



*Number in first column indicates number of blows of 140-lb. hammer dropped 30 in. required to seat 2-in. O. D. splitspoon sampler 6 in. Number in second column indicates number of blows of 140-lb. hammer dropped 30 in. required to drive 2-in. O. D. splitspoon sampler 1 ft. after seating 6 in.

WHILE THIS LOG OF BORING IS CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT ITS RESPECTIVE LOCATION ON THE DATE SHOWN, IT IS NOT WARRANTED THAT IT IS REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.



Remarks: ______

Predominant type shown heavy. Modifying type shown light.

LOG OF BORING

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

Name of Project: London Avenue Canal, Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District, New Orleans, La.
Burk & Associates, Inc., New Orleans, Louisiana

Boring No. 50 (Cont'd) **Soil Technician** A. Croal, Jr. **Date** 13 November 1985

Ground Elev. ______ **Datum** ______ **Gr. Water Depth** See Text

Sample No.	SAMPLE Depth — Feet		DEPTH STRATUM Feet		VISUAL CLASSIFICATION	*STANDARD PENETRATION TEST	
	From	To	From	To			
19	58.5	60.0	57.5	60.0	Soft gray clay w/shell fragments	2	4
20	63.2	64.0	60.0	66.0	Medium stiff gray fissured clay w/sand		
					pockets & few shell fragments &		
					vertical fissures.		
21	68.2	69.0	66.0	73.0	Stiff gray clay w/few shell fragments		
22	73.2	74.0	73.0	75.0	Stiff greenish-gray clay w/silt		
					pockets & shells		
23	76.7	77.5	75.0	77.5	Very stiff greenish-gray & tan clay		
					w/few silt pockets		
24	77.5	79.0	77.5		Compact gray sandy silt	9	44
25	80.0	81.5			Ditto	8	35
26	82.5	84.0		84.5	Medium compact gray sandy silt	6	21
27	85.0	86.5	84.5	87.0	Very loose gray sandy silt w/clay	2	2
					layers		
28	88.5	90.0	87.0		Medium stiff gray clay w/clayey silt	2	6
					lenses & layers		
29	91.7	92.5			Medium stiff gray clay w/sandy silt		
				94	layers		
30	96.7	97.5	94	100.0	Stiff gray clay w/silt lenses		

*Number in first column indicates number of blows of 140-lb. hammer dropped 30 in. required to seat 2-in. O. D. splitspoon sampler 6 in. Number in second column indicates number of blows of 140-lb. hammer dropped 30 in. required to drive 2-in. O. D. splitspoon sampler 1 ft. after seating 6 in.

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Remarks: ______

CLAY SILT SAND HUMUS

Predominant type shown heavy. Modifying type shown light.

DEPTH IN FT.

LOG OF BORING

EUSTIS ENGINEERING COMPANY

SOIL AND FOUNDATION CONSULTANTS

METAIRIE, LA.

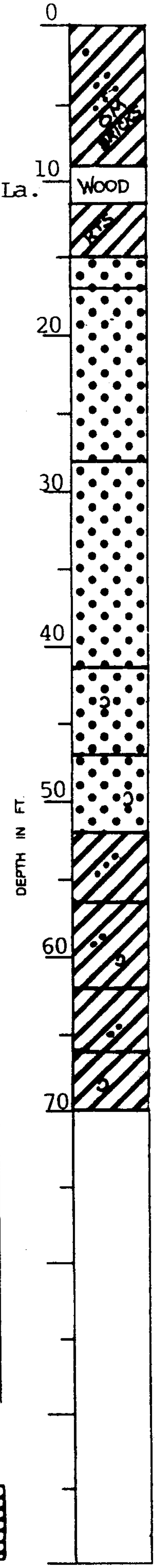
Name of Project: London Avenue Canal, Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District, New Orleans, La.
Burk & Associates, Inc., New Orleans, Louisiana

Boring No. 53 Soil Technician A. J. Mayeux Date 4 December 1985

Ground Elev. ________ Datum ________ Gr. Water Depth See Text

Sample No.	Sample Depth — Feet From	Sample Depth — Feet To	Depth Stratum Feet From	Depth Stratum Feet To	Visual Classification	*Standard Penetration Test	
1	2.0	2.5	0.0		Medium stiff gray & tan clay w/sand		
					lenses & pockets & trace of organic		
					matter		
2	5.5	6.0			Medium stiff gray & tan clay w/sand		
					pockets		
3	8.0	8.5		9.0	Medium stiff gray & tan clay w/vertical		
					sand layers, organic matter & bricks		
	11.0	11.5	9.0	11.5	Wood w/organic matter & clay		
4	14.0	14.5	11.5	15.0	Soft gray & tan clay w/decayed roots		
5	15.0	16.5	15.0	17.0	Loose gray sand	1	5
6	17.5	19.0	17.0		Medium dense gray sand	2	13
7	20.0	21.5			Ditto	4	19
8	23.5	25.0		28.0	Ditto	2	15
9	28.5	30.0	28.0		Dense gray sand	5	35
10	33.5	35.0			Ditto	10	48
11	38.5	40.0		41.5	Ditto	7	32
12	43.5	45.0	41.5	47.0	Medium dense gray sand w/shell	5	19
					fragments		
13	48.5	50.0	47.0	52.0	Loose gray sand w/shell fragments	2	8
14	53.5	55.0	52.0	56.5	Medium stiff gray clay w/sand layers	1	4
15	59.0	59.5	56.5	62.0	Stiff gray & tan clay w/sand pockets &		
					shell fragments		
16	64.0	64.5	62.0	66.0	Stiff gray clay w/sand pockets		
17	69.0	69.5	66.0	70.0	Medium stiff gray clay w/shell		
					fragments		



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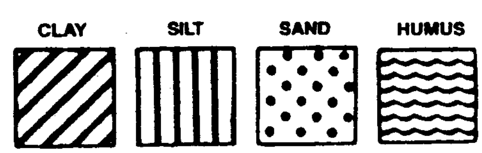

Predominant type shown heavy. Modifying type shown light.

Remarks: ________

Geotechnical Investigation
London Avenue Canal
Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana

Burk & Associates, Inc., Engineers, Planners & Environmental Scientists
New Orleans, Louisiana

SUMMARY OF LABORATORY TEST RESULTS

BORING 15

Sample No.	Depth In Feet	Classification	Water Content Percent	Density PCF Dry	Density PCF Wet	Unconfined Compressive Strength PSF
1	1.7	Medium stiff gray & tan clay w/silt pockets & roots	43.7	73.9	106.2	1715
2	4.7	Medium stiff gray & tan clay w/sand pockets & roots	54.0	65.2	100.5	1935
3	7.7	Soft dark gray clay w/silty sand layers & roots	53.5	60.1	92.3	590*
4	10.7	Soft dark gray clay w/organic matter & roots	92.9	45.7	88.1	690
5	13.7	Soft gray clay w/roots	70.8	57.2	97.6	630
16	58.2	Medium stiff gray clay w/silty sand pockets & shell fragments	46.1	73.7	107.7	1755
18	68.2	Stiff gray clay w/trace of sand	47.8	73.3	108.3	2570

*Unconsolidated Undrained Triaxial Compression Test - One Specimen; Confined at the approximate overburden pressure.

Geotechnical Investigation
London Avenue Canal
Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana

Burk & Associates, Inc., Engineers, Planners & Environmental Scientists
New Orleans, Louisiana

SUMMARY OF LABORATORY TEST RESULTS

BORING 18

Sample No.	Depth In Feet	Classification	Water Content Percent	Density PCF Dry	Density PCF Wet	Unconfined Compressive Strength PSF	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI
2	5.0	Stiff gray clay w/sand pockets & shells	26.6	90.2	114.1	3240			
3	8.0	Medium stiff brown & gray clay w/organic matter & roots	74.1	42.9	74.8	1710*			
4	10.5	Soft brown & gray clay w/organic matter & many roots	76.4	----	-----	----			
5	13.5	Soft gray clay w/roots	58.4	64.1	101.5	755			
6	16.5	Soft gray clay w/sand pockets & organic matter	47.0	72.6	106.8	900	72	23	49

*Unconsolidated Undrained Triaxial Compression Test - One Specimen; Confined at the approximate overburden pressure.

Geotechnical Investigation
London Avenue Canal
Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana

Burk & Associates, Inc., Engineers, Planners & Environmental Scientists
New Orleans, Louisiana

SUMMARY OF LABORATORY TEST RESULTS

BORING 19

Sample No.	Depth In Feet	Classification	Water Content Percent	Density PCF Dry	Density PCF Wet	Unconfined Compressive Strength PSF
1	2.0	Stiff tan & gray clay w/sand pockets & roots	32.7	84.4	112.0	2050
2	5.0	Medium stiff gray & tan clay w/sand lenses, pockets & roots	32.3	86.7	114.8	1380
3	8.0	Very soft gray clay w/organic clay layers, roots & wood	104.7	42.3	86.5	445
4	14.0	Soft gray clay w/roots	68.3	58.6	98.5	945
16	64.0	Medium stiff gray clay w/trace of sand	51.8	69.3	105.1	1305
17	69.0	Stiff gray clay w/sand pockets	41.7	78.0	110.5	2055
18	74.0	Stiff gray clay w/trace of organic matter & shell fragments	60.1	63.0	100.9	3225
19	79.0	Medium stiff greenish-gray silty clay w/fine sand	21.2	103.5	125.4	1015

Geotechnical Investigation
London Avenue Canal
Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana

Burk & Associates, Inc., Engineers, Planners & Environmental Scientists
New Orleans, Louisiana

SUMMARY OF LABORATORY TEST RESULTS

BORING 50

Sample No.	Depth in Feet	Classification	Water Content Percent	Density PCF Dry	Density PCF Wet	Unconfined Compressive Strength PSF	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI
5	8.2	Soft dark gray clay w/silt pockets & organic matter	51.6	64.1	97.2	805			
6	10.7	Soft dark gray clay w/much organic matter & roots	104.2	41.7	85.2	700	161	45	116
7	13.7	Very soft gray clay w/organic matter & wood	80.7	----	-----	----			
8	18.2	Soft gray clay w/trace of organic matter	84.3	50.6	93.2	580			
20	63.2	Medium stiff gray fissured clay w/sand pockets & partings	44.6	72.7	105.2	1545			
21	68.2	Stiff gray clay w/shell fragments	44.5	75.3	108.8	2430	80	25	55
22	73.2	Stiff greenish-gray clay w/silt pockets & shell fragments	31.6	87.7	115.5	2300			
23	76.7	Stiff greenish-gray & tan clay w/silt pockets	28.8	89.3	115.1	2500	71	22	49
29	91.7	Medium stiff gray clay w/sandy silt layers	46.0	75.8	110.6	1625	74	23	51
30	96.7	Stiff gray clay w/silt lenses	37.9	83.6	115.3	2800			

Geotechnical Investigation
London Avenue Canal
Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana

Burk & Associates, Inc., Engineers, Planners & Environmental Scientists
New Orleans, Louisiana

SUMMARY OF LABORATORY TEST RESULTS

BORING 53

Sample No.	Depth in Feet	Classification	Water Content Percent	Density PCF Dry	Density PCF Wet	Unconfined Compressive Strength PSF
1	2.0	Medium stiff gray & tan clay w/sand lenses, pockets & trace of organic matter	35.3	82.3	111.4	1545*
2	5.5	Medium stiff gray & tan clay w/sand pockets	42.2	74.0	105.3	1510
3	8.0	Medium stiff gray & tan clay w/vertical sand layers, organic matter & brick	44.4	----	-----	----
4	14.0	Soft gray & tan clay w/decayed roots	87.0	----	-----	----
15	59.0	Stiff gray & tan clay w/sand pockets & shell fragments	45.2	75.3	109.3	2055
16	64.0	Stiff gray clay w/sand pockets	54.3	68.2	105.2	2155
17	69.0	Medium stiff gray clay w/shell fragments	54.6	67.8	104.8	1705

*Unconsolidated Undrained Triaxial Compression Test - One Specimen; Confined at the approximate overburden pressure.

APPENDIX II

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

DATE: 24-MARCH-98 TIME: 8:54:14

****
* INPUT DATA *
****

I.--HEADING
'LONDON AVENUE CANAL @ MIRABEAU AVE
'EE15166 (SOIL PARAMETERS EE9223)
'EAST BANK
'Q-CASE, SWL + 2 = 13.85, FS=1

II.--CONTROL
CANTILEVER WALL DESIGN
FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
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)
0.00	2.60
22.00	-7.60
47.00	-9.80
72.00	-10.16

IV.B.--LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
0.00	4.50
10.00	4.50
40.00	-1.30
140.00	-2.70

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE = 1.00
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = 1.00

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.
104.00	104.00	23.00	0.00	0.00	0.00	0.00	0.00	1.00	1.00
96.00	96.00	0.00	300.00	0.00	0.00	-16.00	0.00	1.00	1.00
120.00	120.00	30.00	0.00	0.00	0.00	-41.00	0.00	1.00	1.00
120.00	120.00	25.00	0.00	0.00	0.00			1.00	1.00

V.B.--LEFTSIDE
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE = 1.00
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = 1.00

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.
104.00	104.00	23.00	0.00	0.00	0.00	0.00	0.00	1.00	1.00
96.00	96.00	0.00	300.00	0.00	0.00	-16.00	0.00	1.00	1.00
120.00	120.00	30.00	0.00	0.00	0.00	-41.00	0.00	1.00	1.00
120.00	120.00	25.00	0.00	0.00	0.00			1.00	1.00

VI.--WATER DATA
UNIT WEIGHT = 62.40 (PCF)
RIGHTSIDE ELEVATION = 13.85 (FT)
LEFTSIDE ELEVATION = -3.00 (FT)
NO SEEPAGE

VII.--VERTICAL SURCHARGE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

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

DATE: 24-MARCH-98 TIME: 8:54:33

```
**************************
*   SOIL PRESSURES FOR   *
* CANTILEVER WALL DESIGN *
**************************
```

I.--HEADING
'LONDON AVENUE CANAL @ MIRABEAU AVE
'EE15166 (SOIL PARAMETERS EE9223)
'EAST BANK
'Q-CASE, SWL + 2 = 13.85, FS=1

II.--SOIL PRESSURES

RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE

LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE

ELEV. (FT)	NET WATER (PSF)	<---LEFTSIDE---> PASSIVE (PSF)	ACTIVE (PSF)	<------NET------> (SOIL + WATER) ACTIVE (PSF)	PASSIVE (PSF)	<--RIGHTSIDE---> ACTIVE (PSF)	PASSIVE (PSF)
14.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0

13.4	28.1	0.0	0.0	28.1	28.1	0.0	0.0
12.4	90.5	0.0	0.0	90.5	90.5	0.0	0.0
11.4	152.9	0.0	0.0	152.9	152.9	0.0	0.0
10.4	215.3	0.0	0.0	215.3	215.3	0.0	0.0
9.4	277.7	0.0	0.0	277.7	277.7	0.0	0.0
8.4	340.1	0.0	0.0	340.1	340.1	0.0	0.0
7.4	402.5	0.0	0.0	402.5	402.5	0.0	0.0
6.4	464.9	0.0	0.0	464.9	464.9	0.0	0.0
5.4	527.3	0.0	0.0	527.3	527.3	0.0	0.0
4.5	583.4	0.0	0.0	583.4	583.4	0.0	0.0
4.4	589.7	23.7	4.6	565.9	585.1	0.0	0.0
3.4	652.1	261.1	50.1	390.9	602.0	0.0	0.0
2.6	702.0	451.0	86.6	251.0	615.4	0.0	0.0
2.4	714.5	498.5	95.7	218.7	630.0	2.8	11.2
1.6	764.4	688.4	132.1	89.9	688.1	13.9	55.8
1.4	776.9	735.9	141.2	60.0	700.6	19.0	65.0
1.2+	788.5	780.3	151.4	8.2	797.6	0.0	225.9
1.2-	788.5	780.3	151.4	8.2	797.6	0.0	95.0
1.2	839.3	791.5	149.8	0.0	805.9	0.0	102.6
0.4	864.2	972.3	124.3	-133.0	940.9	0.0	225.9
0.0	884.9	1050.1	44.2	-185.8	1210.3	0.0	390.2
-0.3+	901.7	1125.6	0.0	-223.8	1390.3	0.0	505.5
-0.3-	901.7	1091.7	0.0	-223.8	1390.3	0.0	505.5
-0.6	943.5	1125.6	0.0	-224.0	1501.2	0.0	599.6
-1.3+	964.1	1221.9	0.0	-262.5	1602.7	0.0	659.3
-1.3-	964.1	1190.1	0.0	-262.5	1602.7	0.0	659.3
-1.6	1026.5	1221.9	17.8	-257.8	1635.0	0.0	688.7
-2.6	1051.4	1317.5	117.1	-291.0	1618.7	0.0	709.3
-3.0	1051.4	1346.8	148.0	-295.3	1623.3	0.0	719.8
-3.6	1051.4	1365.9	176.2	-314.5	1613.1	0.0	737.8
-4.6	1051.4	1377.1	209.8	-325.6	1586.1	0.0	744.5
-5.6	1051.4	1387.0	243.6	-335.5	1501.0	0.0	693.2
-6.6	1051.4	1397.1	275.8	-345.6	1415.0	0.0	639.3
-7.6	1051.4	1407.1	303.0	-355.7	1389.5	0.0	641.1
-8.6	1051.4	1417.1	325.8	-365.6	1392.7	0.0	667.1
-9.6	1051.4	1428.4	346.6	-376.9	1394.8	0.0	690.0
-10.6	1051.4	1444.2	364.7	-392.7	1399.7	0.0	713.0
-11.6	1051.4	1463.8	380.1	-412.4	1407.2	0.0	735.9
-12.6	1051.4	1484.3	395.1	-432.9	1415.3	0.0	758.9
-13.6	1051.4	1504.7	410.1	-453.2	1423.2	0.0	781.8
-14.6	1051.4	1525.0	425.1	-473.6	1426.1	0.0	799.7
-14.7+	1051.4	1523.9	426.0	-472.5	1483.3	0.0	910.2
-14.7-	1051.4	1523.9	426.0	-472.5	1483.3	0.0	805.5
-15.6	1051.4	1872.9	413.1	2220.7	1548.6	3042.1	910.2
-16.0	1051.4	2374.0	379.2	3103.0	1715.8	4425.6	1043.5
-16.6	1051.4	3058.9	336.0	732.4	1958.6	2739.9	1243.1
-17.6	1051.4	3395.3	332.2	-2309.0	2110.4	34.9	1391.1
-18.6	1051.4	3510.0	346.7	-2203.0	2190.5	255.5	1485.7
-19.6	1051.4	3643.1	359.7	-2320.7	2279.4	271.0	1587.7
-20.6	1051.4	3776.7	372.7	-2438.7	2372.0	286.5	1693.3
-21.6	1051.4	3901.3	385.7	-2547.9	2463.9	302.0	1798.1
-22.6	1051.4	4004.9	398.7	-2636.0	2566.6	317.5	1913.9
-23.6	1051.4	4095.7	411.7	-2711.3	2744.5	333.0	2104.8
-24.6	1051.4	4183.7	424.7	-2783.9	2972.0	348.4	2345.3
-25.6	1051.4	4223.1	437.7	-2807.5	3132.7	364.1	2519.0
-26.6	1051.4	4220.0	450.7	-2788.5	3244.5	380.1	2643.8
-27.6	1051.4	4266.3	463.7	-2818.6	3361.8	396.3	2774.1
-28.6	1051.4	4350.2	476.7	-2886.3	3485.3	412.5	2910.5

-29.6	1051.4	4448.9	489.7	-2968.8	3622.1	428.7	3060.3
-30.6	1051.4	4604.0	502.7	-3107.7	3766.8	444.9	3218.0
-31.6	1051.4	4780.4	515.7	-3267.9	3912.1	461.1	3376.4
-32.6	1051.4	4922.0	528.8	-3393.3	4057.1	477.3	3534.5
-33.6	1051.4	5044.4	542.9	-3499.5	4201.2	493.5	3692.7

```
PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                          BY CLASSICAL METHODS
DATE: 24-MARCH-98                                            TIME: 8:54:34

                         ****************************
                         *  SUMMARY OF RESULTS FOR  *
                         *  CANTILEVER WALL DESIGN  *
                         ****************************

     I.--HEADING
      'LONDON AVENUE CANAL @ MIRABEAU AVE
      'EE15166  (SOIL PARAMETERS EE9223)
      'EAST BANK
      'Q-CASE, SWL + 2 = 13.85,  FS=1

    II.--SUMMARY

         RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

         LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

         *****WARNING:  STANDARD WEDGE SOLUTION MAY NOT EXIST
                        ALL ELEVATIONS.  SEE COMPLETE OUTPUT.

    WALL BOTTOM ELEV. (FT)      :       -27.08
          PENETRATION (FT)      :        29.68

    MAX. BEND. MOMENT (LB-FT)   :   4.5664E+04
          AT ELEVATION (FT)     :       -11.07

    MAX. SCALED DEFL. (LB-IN^3) :   4.4088E+10
          AT ELEVATION (FT)     :        14.40

              NOTE:  DIVIDE SCALED DEFLECTION 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
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BY CLASSICAL METHODS

DATE: 24-MARCH-98 TIME: 8:54:35

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***************************
* COMPLETE OF RESULTS FOR  *
*  CANTILEVER WALL DESIGN  *
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I.--HEADING

'LONDON AVENUE CANAL @ MIRABEAU AVE
'EE15166 (SOIL PARAMETERS EE9223)
'EAST BANK
'Q-CASE, SWL + 2 = 13.85, FS=1

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
14.40	0.0000E+00	0.	4.4088E+10	0.00
13.85	2.4307E-08	0.	4.3121E+10	0.00
13.40	9.4770E-01	6.	4.2329E+10	28.08
12.40	3.1706E+01	66.	4.0570E+10	90.48
11.40	1.5294E+02	187.	3.8811E+10	152.88
10.40	4.2706E+02	371.	3.7052E+10	215.28
9.40	9.1646E+02	618.	3.5294E+10	277.68
8.40	1.6835E+03	927.	3.3537E+10	340.08
7.40	2.7907E+03	1298.	3.1784E+10	402.48
6.40	4.3003E+03	1732.	3.0035E+10	464.88
5.40	6.2749E+03	2228.	2.8294E+10	527.28
4.50	8.5010E+03	2728.	2.6736E+10	583.44
4.40	8.7766E+03	2785.	2.6563E+10	565.94
3.40	1.1815E+04	3263.	2.4848E+10	390.95
2.60	1.4536E+04	3520.	2.3491E+10	250.95
2.40	1.5245E+04	3567.	2.3154E+10	218.74
1.60	1.8155E+04	3691.	2.1817E+10	89.91
1.40	1.8895E+04	3706.	2.1486E+10	59.96
1.21	1.9587E+04	3712.	2.1178E+10	8.23
1.17	1.9763E+04	3712.	2.1100E+10	0.00
0.40	2.2594E+04	3661.	1.9850E+10	-133.03
0.00	2.4046E+04	3597.	1.9207E+10	-185.83
-0.33	2.5224E+04	3530.	1.8680E+10	-223.80
-0.60	2.6168E+04	3469.	1.8254E+10	-223.95
-1.27	2.8438E+04	3307.	1.7209E+10	-262.52
-1.60	2.9516E+04	3221.	1.6702E+10	-257.80
-2.60	3.2602E+04	2946.	1.5202E+10	-290.97
-3.00	3.3757E+04	2829.	1.4617E+10	-295.32
-3.60	3.5400E+04	2646.	1.3758E+10	-314.48
-4.60	3.7887E+04	2326.	1.2375E+10	-325.62
-5.60	4.0049E+04	1995.	1.1058E+10	-335.52
-6.60	4.1875E+04	1655.	9.8094E+09	-345.61
-7.60	4.3355E+04	1304.	8.6334E+09	-355.70
-8.60	4.4480E+04	944.	7.5322E+09	-365.61
-9.60	4.5239E+04	572.	6.5079E+09	-376.94
-10.60	4.5620E+04	187.	5.5617E+09	-392.72
-11.60	4.5608E+04	-215.	4.6942E+09	-412.36
-12.60	4.5183E+04	-638.	3.9055E+09	-432.86
-13.60	4.4325E+04	-1081.	3.1948E+09	-453.22

-14.60	4.3015E+04	-1544.	2.5607E+09	-473.58
-14.65	4.2934E+04	-1569.	2.5297E+09	-472.47
-15.60	4.1638E+04	-740.	2.0008E+09	2220.67
-16.00	4.1543E+04	324.	1.7971E+09	3103.03
-16.60	4.2154E+04	1475.	1.5132E+09	732.42
-17.60	4.3488E+04	687.	1.0985E+09	-2309.00
-18.60	4.3038E+04	-1569.	7.5875E+08	-2203.01
-19.55	4.0538E+04	-3714.	5.0468E+08	-2314.75
-19.60	4.0348E+04	-3830.	4.9301E+08	-2277.19
-20.60	3.5503E+04	-5734.	2.9667E+08	-1531.61
-21.60	2.9127E+04	-6893.	1.6146E+08	-786.03
-22.60	2.1966E+04	-7306.	7.6473E+07	-40.45
-23.60	1.4763E+04	-6974.	2.9432E+07	705.12
-24.60	8.2659E+03	-5896.	8.0045E+06	1450.70
-25.60	3.2193E+03	-4073.	1.0688E+06	2196.28
-26.60	3.6906E+02	-1504.	1.2513E+04	2941.86
-27.08	0.0000E+00	0.	0.0000E+00	3301.01

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

III.--WATER AND SOIL PRESSURES

		<-------------SOIL PRESSURES------------->			
	WATER	<----LEFTSIDE----->		<---RIGHTSIDE---->	
ELEVATION (FT)	PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
14.40	0.	0.	0.	0.	0.
13.85	0.	0.	0.	0.	0.
13.40	28.	0.	0.	0.	0.
12.40	90.	0.	0.	0.	0.
11.40	153.	0.	0.	0.	0.
10.40	215.	0.	0.	0.	0.
9.40	278.	0.	0.	0.	0.
8.40	340.	0.	0.	0.	0.
7.40	402.	0.	0.	0.	0.
6.40	465.	0.	0.	0.	0.
5.40	527.	0.	0.	0.	0.
4.50	583.	0.	0.	0.	0.
4.40	590.	24.	5.	0.	0.
3.40	652.	261.	50.	0.	0.
2.60	702.	451.	87.	0.	0.
2.40	714.	499.	96.	3.	11.
1.60	764.	688.	132.	14.	56.
1.40	777.	736.	141.	19.	65.
1.21+	789.	780.	151.	0.	226.
1.21+	789.	780.	151.	0.	95.
1.17	839.	791.	150.	0.	103.
0.40	864.	972.	124.	0.	226.
0.00	885.	1050.	44.	0.	390.
-0.33+	902.	1126.	0.	0.	505.
-0.33+	902.	1092.	0.	0.	505.
-0.60	943.	1126.	0.	0.	600.
-1.27+	964.	1222.	0.	0.	659.
-1.27+	964.	1190.	0.	0.	659.
-1.60	1026.	1222.	18.	0.	689.

-2.60	1051.	1317.	117.	0.	709.
-3.00	1051.	1347.	148.	0.	720.
-3.60	1051.	1366.	176.	0.	738.
-4.60	1051.	1377.	210.	0.	744.
-5.60	1051.	1387.	244.	0.	693.
-6.60	1051.	1397.	276.	0.	639.
-7.60	1051.	1407.	303.	0.	641.
-8.60	1051.	1417.	326.	0.	667.
-9.60	1051.	1428.	347.	0.	690.
-10.60	1051.	1444.	365.	0.	713.
-11.60	1051.	1464.	380.	0.	736.
-12.60	1051.	1484.	395.	0.	759.
-13.60	1051.	1505.	410.	0.	782.
-14.60	1051.	1525.	425.	0.	800.
-14.65+	1051.	1524.	426.	0.	910.
-14.65+	1051.	1524.	426.	0.	806.
-15.60	1051.	1873.	413.	3042.	910.
-16.00	1051.	2374.	379.	4426.	1044.
-16.60	1051.	3059.	336.	2740.	1243.
-17.60	1051.	3395.	332.	35.	1391.
-18.60	1051.	3510.	347.	256.	1486.
-19.55	1051.	3636.	359.	270.	1583.
-19.60	1051.	3643.	360.	271.	1588.
-20.60	1051.	3777.	373.	286.	1693.
-21.60	1051.	3901.	386.	302.	1798.
-22.60	1051.	4005.	399.	317.	1914.
-23.60	1051.	4096.	412.	333.	2105.
-24.60	1051.	4184.	425.	348.	2345.
-25.60	1051.	4223.	438.	364.	2519.
-26.60	1051.	4220.	451.	380.	2644.
-27.08	1051.	4266.	464.	396.	2774.
-28.60	1051.	4350.	477.	412.	2911.

APPENDIX III

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

DATE: 26-MARCH-98 TIME: 2:08:32

****
* INPUT DATA *
****

I.--HEADING
'LONDON AVENUE CANAL @ FILMORE AVE
'EE15166 (SOIL PARAMETERS EE9223)
'EAST BANK
'Q-CASE, SWL = 11.85, FS=1.5

II.--CONTROL
CANTILEVER WALL DESIGN
FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.50
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)
0.00	2.00
11.00	2.00
15.00	0.91
39.00	-9.34
64.00	-9.25

IV.B.--LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
0.00	3.00
10.00	3.00
25.00	-2.00
150.00	-4.00

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE = 1.50
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = 1.50

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.
112.00	112.00	23.00	0.00	0.00	0.00	-2.00	0.00	1.50	1.50
80.00	80.00	0.00	225.00	0.00	0.00	-7.00	0.00	1.50	1.50
100.00	100.00	0.00	430.00	0.00	0.00	-12.00	0.00	1.50	1.50
120.00	120.00	30.00	0.00	0.00	0.00	-28.00	0.00	1.50	1.50
120.00	120.00	33.00	0.00	0.00	0.00			1.50	1.50

V.B.--LEFTSIDE

LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE = 1.50
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = 1.50

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.
12.00	112.00	23.00	0.00	0.00	0.00	-2.00	0.00	1.50	1.50
80.00	80.00	0.00	225.00	0.00	0.00	-7.00	0.00	1.50	1.50
100.00	100.00	0.00	430.00	0.00	0.00	-12.00	0.00	1.50	1.50
20.00	120.00	30.00	0.00	0.00	0.00	-28.00	0.00	1.50	1.50
20.00	120.00	33.00	0.00	0.00	0.00			1.50	1.50

VI.--WATER DATA
UNIT WEIGHT = 62.40 (PCF)
RIGHTSIDE ELEVATION = 11.85 (FT)
LEFTSIDE ELEVATION = -3.00 (FT)
NO SEEPAGE

VII.--VERTICAL SURCHARGE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

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

DATE: 26-MARCH-98 TIME: 2:08:46

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*   SOIL PRESSURES FOR   *
* CANTILEVER WALL DESIGN *
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I.--HEADING
'LONDON AVENUE CANAL @ FILMORE AVE
'EE15166 (SOIL PARAMETERS EE9223)
'EAST BANK
'Q-CASE, SWL = 11.85, FS=1.5

II.--SOIL PRESSURES

RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE

LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE

ELEV.	NET WATER	<---LEFTSIDE---> PASSIVE	ACTIVE	<------NET------> (SOIL + WATER) ACTIVE	PASSIVE	<--RIGHTSIDE---> ACTIVE	PASSIVE

(FT)	(PSF)	(PSF)	(PSF)	(PSF)	(PSF)	(PSF)	(PSF)
14.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11.4	28.1	0.0	0.0	28.1	28.1	0.0	0.0
10.4	90.5	0.0	0.0	90.5	90.5	0.0	0.0
9.4	152.9	0.0	0.0	152.9	152.9	0.0	0.0
8.4	215.3	0.0	0.0	215.3	215.3	0.0	0.0
7.4	277.7	0.0	0.0	277.7	277.7	0.0	0.0
6.4	340.1	0.0	0.0	340.1	340.1	0.0	0.0
5.4	402.5	0.0	0.0	402.5	402.5	0.0	0.0
4.4	464.9	0.0	0.0	464.9	464.9	0.0	0.0
3.4	527.3	0.0	0.0	527.3	527.3	0.0	0.0
3.0	552.2	0.0	0.0	552.2	552.2	0.0	0.0
2.4	589.7	117.5	38.4	472.2	551.2	0.0	0.0
2.0	614.6	195.8	64.1	418.8	550.6	0.0	0.0
1.4	652.1	313.3	102.5	355.8	601.6	17.0	52.0
1.0	677.0	391.6	128.1	313.8	635.6	28.4	86.7
0.4	714.5	509.1	166.6	250.8	686.7	45.4	138.7
-0.6	776.9	705.1	230.7	148.2	769.9	76.5	223.7
-1.6	839.3	897.2	293.1	0.0	887.4	58.4	341.6
-1.6	864.2	898.5	293.5	-1.0	888.2	58.2	342.4
-2.0	865.6	919.7	292.9	-52.4	986.2	3.1	414.8
-2.0+	901.7	917.5	291.9	-51.9	1033.3	0.0	501.2
-2.0-	901.7	917.5	291.9	-51.9	1033.3	0.0	418.0
-2.6	926.6	907.5	307.6	-5.9	1095.3	0.0	501.2
-3.0	926.6	932.1	332.1	-5.5	1113.4	0.0	518.9
-3.6	926.6	950.7	350.5	-24.1	1102.7	0.0	526.6
-4.6	926.6	967.4	368.2	-40.7	1102.6	0.0	544.2
-5.6	926.6	969.4	389.1	-42.8	1095.8	0.0	558.3
-6.6	926.6	1019.4	346.5	-92.7	1216.7	0.0	636.6
-7.0	926.6	1095.0	276.0	-168.4	1376.9	0.0	726.3
-7.6	926.6	1199.1	190.4	-272.4	1585.1	0.0	848.9
-8.6	926.6	1237.0	183.1	-310.4	1654.8	0.0	911.2
-9.6	926.6	1237.2	204.0	-310.6	1655.3	0.0	932.7
-10.6	926.6	1240.5	218.7	-313.9	1665.8	0.0	957.8
-10.8+	926.6	1239.3	221.1	-312.7	1669.6	0.0	968.2
-10.8-	926.6	1239.3	221.1	-312.7	1669.6	0.0	960.0
-11.6	926.6	1305.1	256.8	377.7	1638.0	756.2	968.2
-12.0	926.6	1418.2	301.5	726.4	1591.8	1218.0	966.7
-12.6	926.6	1583.2	358.1	227.3	1557.3	883.8	988.8
-13.6	926.6	1699.5	386.0	-557.7	1615.0	215.2	1074.4
-14.6	926.6	1770.8	396.8	-549.9	1698.8	294.3	1168.9
-15.6	926.6	1845.8	409.0	-597.8	1781.7	321.4	1264.1
-16.6	926.6	1920.9	421.3	-645.7	1867.1	348.6	1361.8
-17.6	926.6	1996.4	433.5	-694.1	1953.7	375.7	1460.6
-18.6	926.6	2065.6	445.7	-736.1	2043.9	402.9	1563.0
-19.6	926.6	2118.1	458.0	-761.5	2095.6	430.0	1626.9
-20.6	926.6	2158.3	470.2	-774.5	2105.1	457.2	1648.7
-21.6	926.6	2197.7	482.4	-786.7	2156.1	484.3	1711.9
-22.6	926.6	2250.7	494.7	-812.6	2242.7	511.5	1810.7
-23.6	926.6	2325.2	506.9	-859.9	2322.1	538.7	1902.4
-24.6	926.6	2409.2	519.1	-916.7	2400.3	565.8	1992.7
-25.6	926.6	2492.6	531.4	-973.0	2478.3	593.0	2083.0
-26.6	926.6	2572.1	545.9	-1024.6	2552.7	620.8	2172.0
-27.6	926.6	2723.2	548.9	-1161.0	2701.7	635.6	2324.0
-28.0	926.6	2846.7	537.5	-1290.8	2827.5	629.3	2438.4

-28.6	926.6	3019.6	525.2	-1471.8	3001.1	621.2	2599.6
-29.6	926.6	3169.3	573.9	-1607.7	3099.6	634.9	2746.9
-30.6	926.6	3291.6	654.1	-1708.5	3109.9	656.4	2837.4
-31.6	926.6	3421.4	725.3	-1817.5	3090.1	677.3	2888.8
-32.6	926.6	3550.7	803.9	-1925.9	3069.8	698.1	2947.1
-33.6	926.6	3679.9	849.0	-2034.5	3116.6	718.8	3039.0
-34.6	926.6	3807.6	859.5	-2142.5	3200.2	738.5	3133.1
-35.6	926.6	3932.6	873.2	-2248.7	3310.8	757.3	3257.3

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

DATE: 26-MARCH-98 TIME: 2:08:49

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****************************
*  SUMMARY OF RESULTS FOR  *
*  CANTILEVER WALL DESIGN  *
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I.--HEADING
'LONDON AVENUE CANAL @ FILMORE AVE
'EE15166 (SOIL PARAMETERS EE9223)
'EAST BANK
'Q-CASE, SWL = 11.85, FS=1.5

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

*****WARNING: STANDARD WEDGE SOLUTION MAY NOT EXIST
ALL ELEVATIONS. SEE COMPLETE OUTPUT.

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WALL BOTTOM ELEV. (FT)       :      -33.26
     PENETRATION (FT)        :       35.26

MAX. BEND. MOMENT (LB-FT)    :  6.7417E+04
     AT ELEVATION (FT)       :      -18.09

MAX. SCALED DEFL. (LB-IN^3)  :  7.6355E+10
     AT ELEVATION (FT)       :       14.40
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NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

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

DATE: 26-MARCH-98 TIME: 2:08:49

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* COMPLETE OF RESULTS FOR *
* CANTILEVER WALL DESIGN *
****

I.--HEADING
'LONDON AVENUE CANAL @ FILMORE AVE
'EE15166 (SOIL PARAMETERS EE9223)
'EAST BANK
'Q-CASE, SWL = 11.85, FS=1.5

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
14.40	0.0000E+00	0.	7.6355E+10	0.00
13.40	1.3970E-09	0.	7.3782E+10	0.00
12.40	1.3970E-09	0.	7.1210E+10	0.00
11.85	4.0802E-08	0.	6.9795E+10	0.00
11.40	9.4770E-01	6.	6.8637E+10	28.08
10.40	3.1706E+01	66.	6.6065E+10	90.48
9.40	1.5294E+02	187.	6.3492E+10	152.88
8.40	4.2706E+02	371.	6.0920E+10	215.28
7.40	9.1646E+02	618.	5.8349E+10	277.68
6.40	1.6835E+03	927.	5.5779E+10	340.08
5.40	2.7907E+03	1298.	5.3212E+10	402.48
4.40	4.3003E+03	1732.	5.0650E+10	464.88
3.40	6.2749E+03	2228.	4.8096E+10	527.28
3.00	7.2088E+03	2444.	4.7077E+10	552.24
2.40	8.7696E+03	2751.	4.5552E+10	472.19
2.00	9.9063E+03	2929.	4.4539E+10	418.82
1.40	1.1735E+04	3162.	4.3024E+10	355.80
1.00	1.3027E+04	3295.	4.2018E+10	313.78
0.40	1.5057E+04	3465.	4.0516E+10	250.75
-0.60	1.8631E+04	3664.	3.8034E+10	148.24
-1.59	2.2319E+04	3738.	3.5601E+10	0.00
-1.60	2.2344E+04	3738.	3.5585E+10	-0.99
-2.00	2.3838E+04	3727.	3.4615E+10	-52.44
-2.02	2.3920E+04	3726.	3.4562E+10	-51.92
-2.60	2.6068E+04	3709.	3.3174E+10	-5.86
-3.00	2.7551E+04	3707.	3.2221E+10	-5.47
-3.60	2.9773E+04	3698.	3.0808E+10	-24.10
-4.60	3.3457E+04	3666.	2.8493E+10	-40.71
-5.60	3.7102E+04	3624.	2.6236E+10	-42.81
-6.60	4.0696E+04	3556.	2.4044E+10	-92.73
-7.00	4.2109E+04	3504.	2.3186E+10	-168.38
-7.60	4.4175E+04	3372.	2.1921E+10	-272.41
-8.60	4.7405E+04	3081.	1.9875E+10	-310.36
-9.60	5.0330E+04	2770.	1.7911E+10	-310.56
-10.60	5.2944E+04	2458.	1.6034E+10	-313.90
-10.81	5.3443E+04	2393.	1.5659E+10	-312.67

-11.60	5.5318E+04	2419.	1.4248E+10	377.74
-12.00	5.6325E+04	2640.	1.3560E+10	726.39
-12.60	5.8010E+04	2926.	1.2558E+10	227.26
-13.60	6.0919E+04	2761.	1.0968E+10	-557.68
-14.60	6.3403E+04	2207.	9.4832E+09	-549.89
-15.60	6.5327E+04	1633.	8.1080E+09	-597.76
-16.60	6.6653E+04	1012.	6.8456E+09	-645.66
-17.60	6.7334E+04	342.	5.6982E+09	-694.07
-18.60	6.7322E+04	-373.	4.6671E+09	-736.13
-19.60	6.6576E+04	-1122.	3.7523E+09	-761.48
-20.60	6.5071E+04	-1890.	2.9524E+09	-774.48
-21.60	6.2792E+04	-2671.	2.2648E+09	-786.74
-22.60	5.9723E+04	-3470.	1.6856E+09	-812.55
-23.60	5.5839E+04	-4307.	1.2095E+09	-859.95
-24.60	5.1093E+04	-5195.	8.2970E+08	-916.71
-25.60	4.5430E+04	-6140.	5.3811E+08	-972.99
-26.50	3.9497E+04	-7037.	3.4280E+08	-1019.53
-26.60	3.8795E+04	-7135.	3.2488E+08	-959.24
-27.60	3.1282E+04	-7790.	1.7855E+08	-349.92
-28.00	2.8144E+04	-7881.	1.3590E+08	-106.19
-28.60	2.3419E+04	-7835.	8.6234E+07	259.39
-29.60	1.5815E+04	-7271.	3.4419E+07	868.71
-30.60	9.0794E+03	-6098.	1.0057E+07	1478.02
-31.60	3.8222E+03	-4315.	1.5966E+06	2087.34
-32.60	6.5234E+02	-1923.	4.1971E+04	2696.65
-33.26	0.0000E+00	0.	0.0000E+00	3100.89

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

III.--WATER AND SOIL PRESSURES

		<-------------SOIL PRESSURES------------->			
	WATER	<----LEFTSIDE----->		<---RIGHTSIDE---->	
ELEVATION (FT)	PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
14.40	0.	0.	0.	0.	0.
13.40	0.	0.	0.	0.	0.
12.40	0.	0.	0.	0.	0.
11.85	0.	0.	0.	0.	0.
11.40	28.	0.	0.	0.	0.
10.40	90.	0.	0.	0.	0.
9.40	153.	0.	0.	0.	0.
8.40	215.	0.	0.	0.	0.
7.40	278.	0.	0.	0.	0.
6.40	340.	0.	0.	0.	0.
5.40	402.	0.	0.	0.	0.
4.40	465.	0.	0.	0.	0.
3.40	527.	0.	0.	0.	0.
3.00	552.	0.	0.	0.	0.
2.40	590.	117.	38.	0.	0.
2.00	615.	196.	64.	0.	0.
1.40	652.	313.	102.	17.	52.
1.00	677.	392.	128.	28.	87.
0.40	714.	509.	167.	45.	139.
-0.60	777.	705.	231.	76.	224.

-1.59	839.	897.	293.	58.	342.
-1.60	864.	899.	294.	58.	342.
-2.00	866.	920.	293.	3.	415.
-2.02+	902.	918.	292.	0.	501.
-2.02+	902.	918.	292.	0.	418.
-2.60	927.	908.	308.	0.	501.
-3.00	927.	932.	332.	0.	519.
-3.60	927.	951.	351.	0.	527.
-4.60	927.	967.	368.	0.	544.
-5.60	927.	969.	389.	0.	558.
-6.60	927.	1019.	347.	0.	637.
-7.00	927.	1095.	276.	0.	726.
-7.60	927.	1199.	190.	0.	849.
-8.60	927.	1237.	183.	0.	911.
-9.60	927.	1237.	204.	0.	933.
-10.60	927.	1241.	219.	0.	958.
-10.81+	927.	1239.	221.	0.	968.
-10.81+	927.	1239.	221.	0.	960.
-11.60	927.	1305.	257.	756.	968.
-12.00	927.	1418.	302.	1218.	967.
-12.60	927.	1583.	358.	884.	989.
-13.60	927.	1699.	386.	215.	1074.
-14.60	927.	1771.	397.	294.	1169.
-15.60	927.	1846.	409.	321.	1264.
-16.60	927.	1921.	421.	349.	1362.
-17.60	927.	1996.	434.	376.	1461.
-18.60	927.	2066.	446.	403.	1563.
-19.60	927.	2118.	458.	430.	1627.
-20.60	927.	2158.	470.	457.	1649.
-21.60	927.	2198.	482.	484.	1712.
-22.60	927.	2251.	495.	511.	1811.
-23.60	927.	2325.	507.	539.	1902.
-24.60	927.	2409.	519.	566.	1993.
-25.60	927.	2493.	531.	593.	2083.
-26.50	927.	2564.	544.	618.	2163.
-26.60	927.	2572.	546.	621.	2172.
-27.60	927.	2723.	549.	636.	2324.
-28.00	927.	2847.	538.	629.	2438.
-28.60	927.	3020.	525.	621.	2600.
-29.60	927.	3169.	574.	635.	2747.
-30.60	927.	3292.	654.	656.	2837.
-31.60	927.	3421.	725.	677.	2889.
-32.60	927.	3551.	804.	698.	2947.
-33.26	927.	3680.	849.	719.	3039.
-34.60	927.	3808.	859.	738.	3133.