GEOTECHNICAL INVESTIGATION

LONDON AVENUE OUTFALL CANAL

FRONTAL PROTECTION AT PUMPING STATION NO. 3

NEW ORLEANS, LOUISIANA

FOR
PEPPER AND ASSOCIATES, INC.
METAIRIE, LOUISIANA

31 JANUARY 1995

FEB 2 1995

EUSTIS ENGINEERING COMPANY, INC.
GEOTECHNICAL ENGINEERS
CONSTRUCTION QUALITY CONTROL AND MATERIALS TESTING
3011 28th Street • Metairie, Louisiana 70002 • 504-834-0157

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31 January 1995

Pepper and Associates, Inc.
3012 26th Street
Metairie, Louisiana 70002

Attention Mr. Jerome Pepper

Gentlemen:

Geotechnical Investigation
London Avenue Outfall Canal
Frontal Protection at Pumping Station No. 3
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.~~

STATE OF LOUISIANA
JOHN R. EUSTIS
REG. NO. 22807
REGISTERED PROFESSIONAL ENGINEER IN CIVIL ENGINEERING

Yours very truly,

EUSTIS ENGINEERING COMPANY, INC.

John R. Eustis

JOHN R. EUSTIS, P.E.

PEPPER & ASSOCIATES, INC.
RECEIVED
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Per
N. W ORLEANS, LA.

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NEW ORLEANS, LOUISIANA

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By
Eustis Engineering Company, Inc.
Metairie, Louisiana

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INTRODUCTION

1. This report contains the results of a geotechnical investigation performed for the proposed improvements to Pumping Station No. 3 located at the southern end of London Avenue Outfall Canal in New Orleans, Louisiana. The investigation was performed in accordance Eustis Engineering Company, Inc.'s (Eustis Engineering) proposal dated 12 January 1993. Authorization to proceed with the investigation was received on 3 August 1994 from Mr. Jerome Pepper, representing Pepper and Associates, Inc., Consulting Engineers, Metairie, Louisiana.

2. This report has been prepared in accordance with generally accepted ~~geotechnical engineering practice for the exclusive use of Pepper and Associates,~~ Inc., for specific application to the subject site. In the event any changes in the nature, design or location of the proposed improvements 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 or modified or verified in writing. Should these data be used by anyone other than Pepper and Associates, Inc., they should contact Eustis Engineering for interpretation of data and to secure other information which may be pertinent to this project.

3. The analyses and recommendations contained in this report 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 contained in this report.

4. Recommendations and conclusions contained in this report are to some degree subjective and should only be used for design purposes. This report should not be included in the contract plans and specifications. However, the results of the soil boring and laboratory tests contained in Appendix I of this report may be included in the plans and specifications.

PREVIOUS GEOTECHNICAL INVESTIGATIONS

5. 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 our 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 1 and 36. The boring locations are shown on Figure 1.

SCOPE

6. The study included a review of the previous geotechnical investigations and the drilling of an additional undisturbed boring 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.

SOIL BORING

7. One undisturbed sample type soil test boring, 125 feet in depth, was made on 4 August 1994 at the location shown on 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 I.

8. The boring was made with a truck mounted rotary type drill rig, and samples of cohesive or semi-cohesive subsoils were obtained at close intervals or changes in stratum using a 3-in. diameter thinwall Shelby tube sampling barrel. The 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.

9. Samples of cohesionless soil were recovered during the performance of in situ Standard Penetration Tests. This test consists of driving a 2-in. diameter splitspoon sampler 1 foot into the soil after it is first seated 6 inches. A 140-lb weight dropped 30 inches is used to advance the sampler. The number of blows required to drive the sampler 1 foot is recorded and is indicative of the relative density of the subsoils tested. The 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.

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

LABORATORY TESTS

11. 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.

12. 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 I following the boring log.

DESCRIPTION OF SUBSOIL CONDITIONS

Topography

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

Geology

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

Stratigraphy

15. Holocene Deposits. Based on the five available soil borings, Holocene deposits can be divided into five distinct strata. The first stratum consists of artificial fill and natural levee deposits to el -13 to -17. This stratum is composed predominantly of CH and CL soils. These soils are oxidized and precompressed. The second stratum contains intradelta deposits of ML, SM and SP soil ranging from el -23.5 to -27.5. The third stratum consists of prodeltaic deposits of CH soil to el -40 to -43. Deposits to these depths form the deltaic plain. Deltaic plain deposits appear normally consolidated. The deltaic plain is underlain by nearshore Gulf deposits of SP, SM, SC and CL soils to el -57 to -62. Beneath this,

nearshore Gulf deposits of predominantly CH soil continue to el -63.5 to -67.5. Nearshore Gulf deposits appear slightly precompressed.

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

Ground Water

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

FOUNDATION ANALYSIS

Furnished Information

18. Information provided by Pepper and Associates, Inc., indicates the existing discharge pipes will be extended approximately 107 feet north. A sluice

gate structure will be placed at the northern end of the discharge tubes to form a barrier across the canal. A 25 ft long portion of the sluice gate structure will have a T-wall monolith between discharge pump "B" and discharge pump "C." The east and west ends of the sluice gates will tie into T-wall structures running north and then into I-wall structures to the Norfolk Southern Railroad embankment. Low water level in the canal is el -1 and hurricane level is el 13.9. The bottom of the discharge basin is at el -9.18.

Soil Design Parameters

19. 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 trixial compression shear (R) test. The soil design parameters are tabulated on Figure 2.

Lateral Earth Pressures

20. 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, we recommend a design lateral earth pressure of 95 psf per linear foot of depth. For clay backfill,

we recommend a design lateral earth pressure of 110 psf per linear foot of depth. These values include the effects of soil and water acting on the walls.

Pile Foundations

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

22. 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.

23. 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 a structural engineer.

24. Pile Group Capacity and Spacing. Furnished information indicates a 60-ton design load capacity will be used for construction. This will require piles being driven to a tip of el -77. Piles driven to this tip elevation will derive their supporting capacity primarily through skin friction, and it will be necessary to consider the effect of group action for piles driven in groups. In this regard, the

supporting value of the friction piles driven in groups should be investigated on the basis of group perimeter shear by the formula shown on Figure 5. For pile groups used in tension, the second term of the formula is deleted. The minimum center to center pile spacing within a pile group or row of piles should be determined in accordance with the pile spacing formula also shown on Figure 5.

25. 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.

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

27. Dynamic Pile Test (DPT). The steel H-piles should have a cross section which is structurally sufficient to facilitate driving of the piles without damage. Driving stresses and drivability of the piles with the selected hammer and appurtenant driving equipment should be evaluated by dynamic analysis (WEAP). Structural requirements can then be verified by a structural engineer and installation criteria can be established.

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

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

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

31. <u>Test Piles and Pile Load Test.</u> A test pile should be installed within the excavation cofferdam. The test pile program can be used to establish

installation criteria for the job piles and will give an indication of the driving resistance and vibrations. The test pile should be allowed to "set" for at least 28 days after driving, and then should be load tested to failure in accordance with the New Orleans Building Code. If DPT is considered for job pile evaluation, pile load tests should be coordinated with DPT to establish relationships between dynamic and static tests.

32. Alternately, a test pile program outside of the excavation may be considered because of construction time constraints. Eustis Engineering should be consulted to select a test pile site and recommend test capacities. Eustis Engineering should also be consulted to evaluate the load test and make appropriate adjustments to the test capacities.

33. Estimated Settlement. For pile foundations embedded in the underlying Pleistocene formation at tip el -77, it is estimated that settlement of the sluice gate structure and T-walls will be ¼ to ¾ of an inch. This estimate of settlement does not include the elastic deformation of the piles or settlement due to the placement of fill near pile foundations. This estimate of settlement is based on the assumption that the foundation design will utilize single rows of piles on relatively wide spacings of 8 to 10 feet with 3 to 4 feet between piles in each row. We have also assumed small isolated pile groups with two to three piles per group. 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, Eustis Engineering should be retained to perform additional settlement analyses.

34. Subgrade Moduli. Analyses were made to estimate the modulus of horizontal subgrade reaction for laterally loaded piles. We have estimated the modulus of horizontal subgrade reaction between el -10 and -90. The 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.

Temporary Cofferdam

35. Design Conditions. Furnished information indicates a temporary cofferdam will be installed in the canal in sections. The Sewerage and Water Board of New Orleans will only allow one pump to be taken out of service at a time. This will require a separate cofferdam for each discharge tube and sluice gate. The bottom of the canal was assumed at el -10 with water elevations in the canal at el 0, 2 and 4.

36. Stability. Preliminary analyses were made for anchored and cantilevered sheetpile walls using Q-case and S-case soil conditions. The point of support for an anchored wall was assumed to be el 2. Based on the preliminary analyses, Pepper and Associates, Inc., chose an anchored sheetpile wall for the temporary cofferdam. The final design for construction assumes water in the canal at el 2 and mudline el -10. A factor of safety of 1 was applied to the soil shear strengths to determine the maximum bending moment and anchor force. A factor of safety of 1.5 was applied to the soil shear strengths to determine the tip embedment.

37. For the S-case condition, our analyses indicate a maximum bending moment of 36 ft-kips per linear foot of wall, anchor force of 4.5 kips per linear foot and tip el -33. For the Q-case condition, the maximum bending moment is 19 ft-kips per linear foot, the anchor force is 3 kips per linear foot, and sheetpiles should be driven to el -30. Due to the temporary duration of loading, we recommend structural requirements for the cofferdam be based on Q-case conditions. Computer printouts of our analyses for both the S-case and Q-case conditions are included in Appendix II.

38. <u>Dewatering and Pressure Relief.</u> Our analyses assume hydrostatic pressures on the cohesionless intradeltaic deposits occurring between el -13 and -27.5 do not exceed el -15. Hydrostatic pressures in the cohesionless nearshore Gulf deposits between el -40 and -62 are assumed not to exceed el 4.5. In order to achieve these hydrostatic pressures, it will be necessary to install a dewatering and hydrostatic pressure relief system.

39. The pressure relief system should be comprised of a series of wells or wellpoints capable of lowering the hydrostatic heads to the levels assumed for our 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. Eustis Engineering should be retained to perform these services.

40. We should note 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.

I-Wall Structure

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

42. 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 III together with a flow chart developed by the USACE. In addition, we have included the computer output for the program "CWALSHT" for the design condition. The results indicate the sheetpile wall should be installed to tip el -0.80. The maximum bending moment is 2,398 ft-lbs. Shear, moment and deflection information is also included in Appendix III.

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

Documentation of Existing Conditions

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

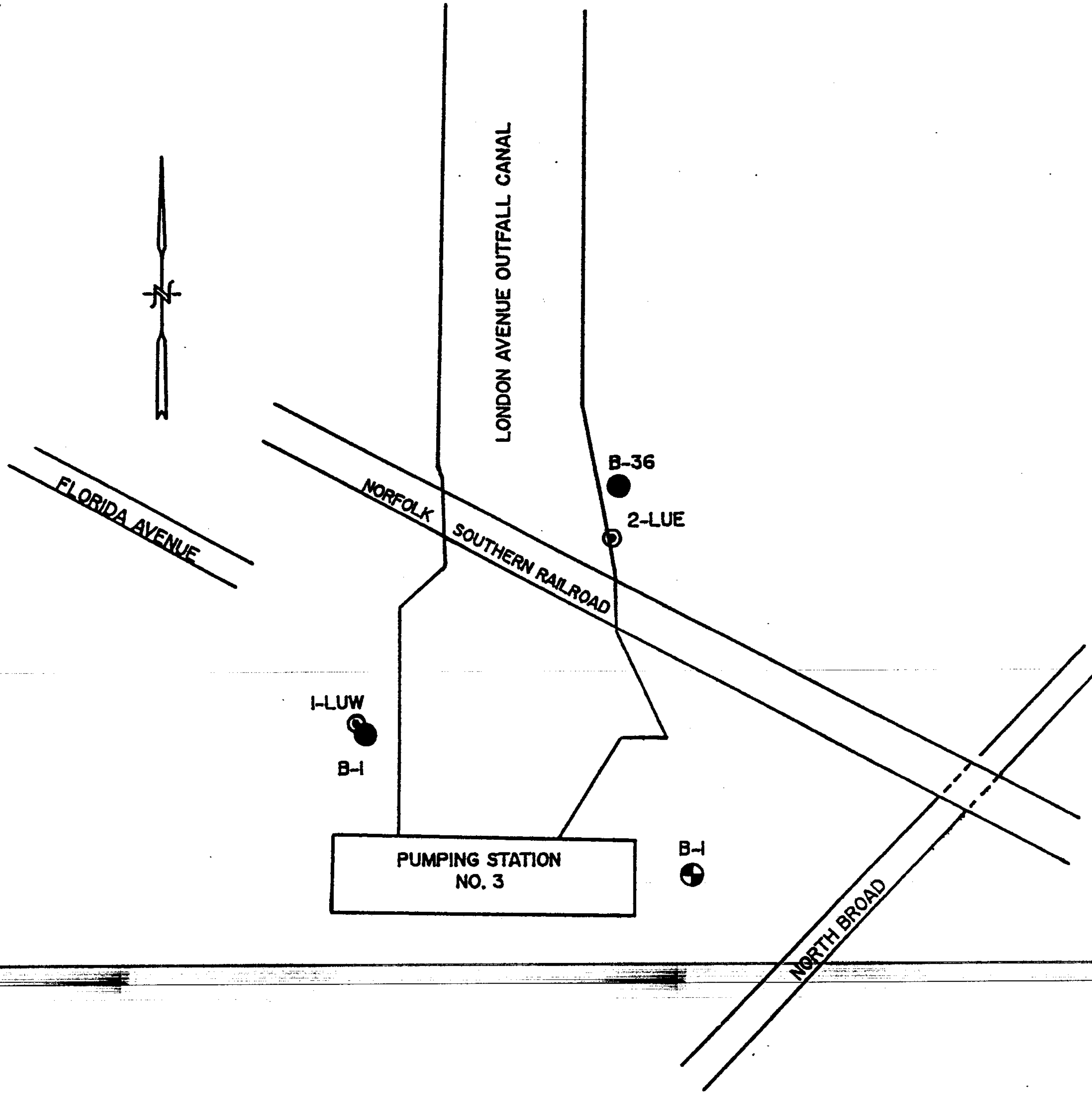
Stability Analyses

45. 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. Our 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 "B" and "C." For the T-wall structure at Station 1 + 57 to 2+07, our analyses indicate a factor of safety of 1.66 during low water. These factors of safety are considered adequate for the structures. Results for our stability analyses are shown on Figures 7 and 8.

GEOTECHNICAL SERVICES DURING CONSTRUCTION

46. To provide continuity between the investigation, design and construction phases, Eustis Engineering should be retained to provide WEAP and DPT using a PDA. Because of our knowledge of the subsoils at this site, Eustis Engineering should be involved with the testing and inspection of all foundation piles for the project. This includes inspection of piles, measuring vibrations, logging the driving of test piles and job piles, and the performance of dynamic and static pile load tests. Also, Eustis Engineering should be retained to provide additional services which may include compaction and inplace density tests of structural fill, asphalt and concrete testing and inspection, and any other soil and materials testing services which will provide quality control during construction and conformance to design specifications.

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



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

EUSTIS ENGINEERING BORINGS (1985)

UNDISTURBED BORING DRILLED 4 AUGUST 1994

LOCATION OF BORINGS

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

EUSTIS ENGINEERING COMPANY, INC.

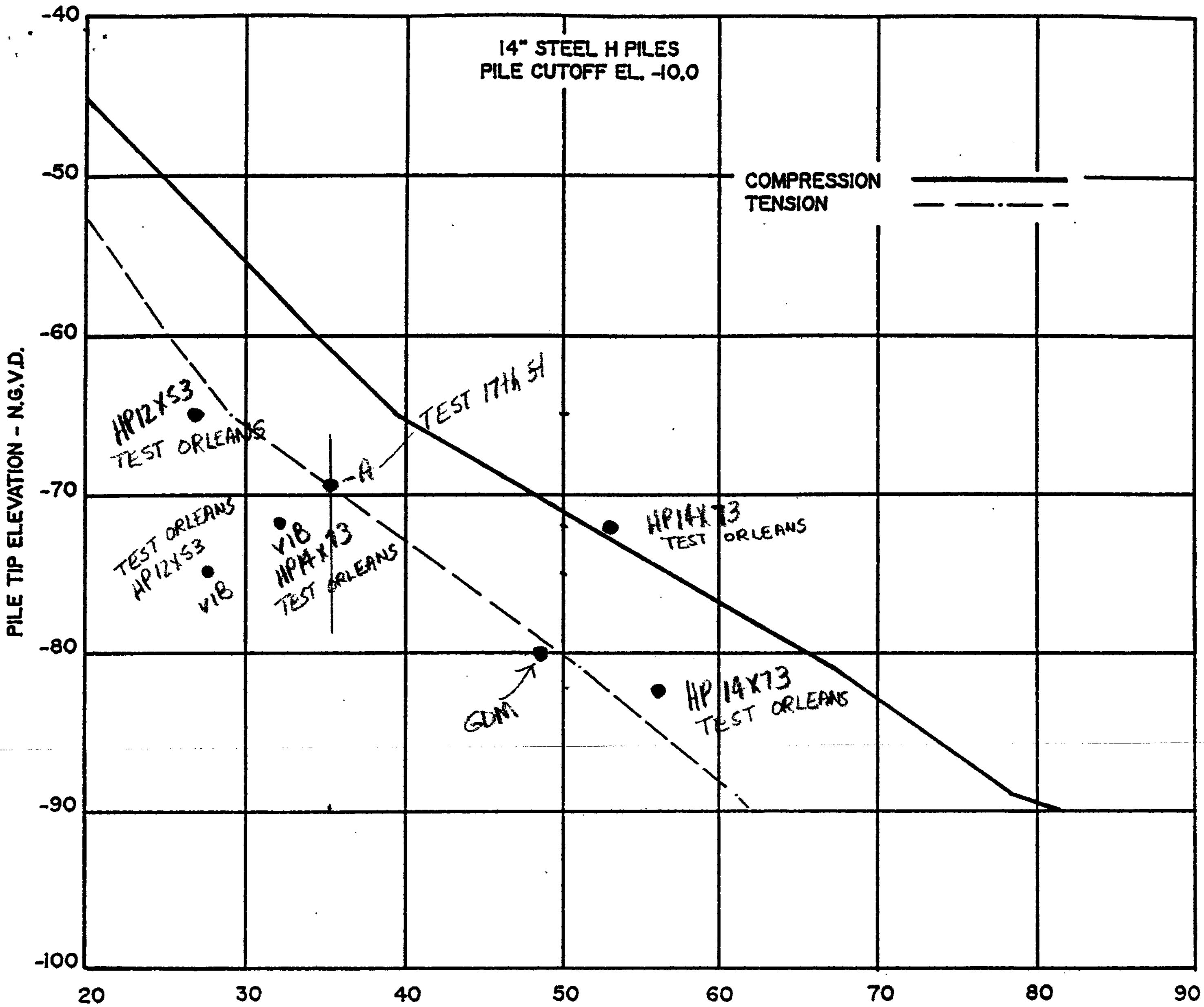
FIGURE 1

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

SOIL DESIGN PARAMETERS

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

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



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

A. HP12X53 Assume ground elev 0.0
69' penetration
17th St Canal

PILE LOAD CAPACITIES

LONDON AVENUE OUTFALL CANAL
FRONTAL PROTECTION AT
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EUSTIS ENGINEERING COMPANY, INC.

FIGURE 3

AXIAL AND HORIZONTAL RESISTANCE OF BATTER PILES

ESTIMATED FROM ALLOWABLE VERTICAL LOAD CAPACITY

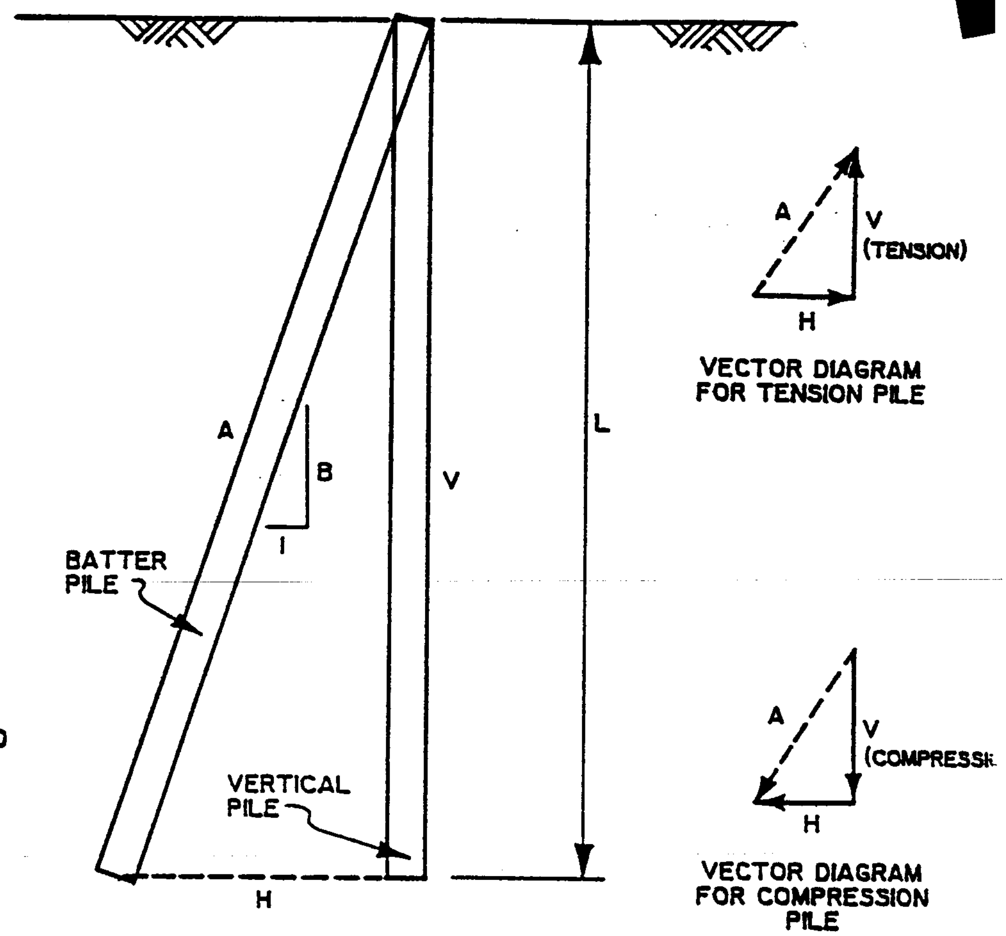
L = VERTICAL COMPONENT OF BATTER PILE EMBEDMENT LENGTH.

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.

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

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.

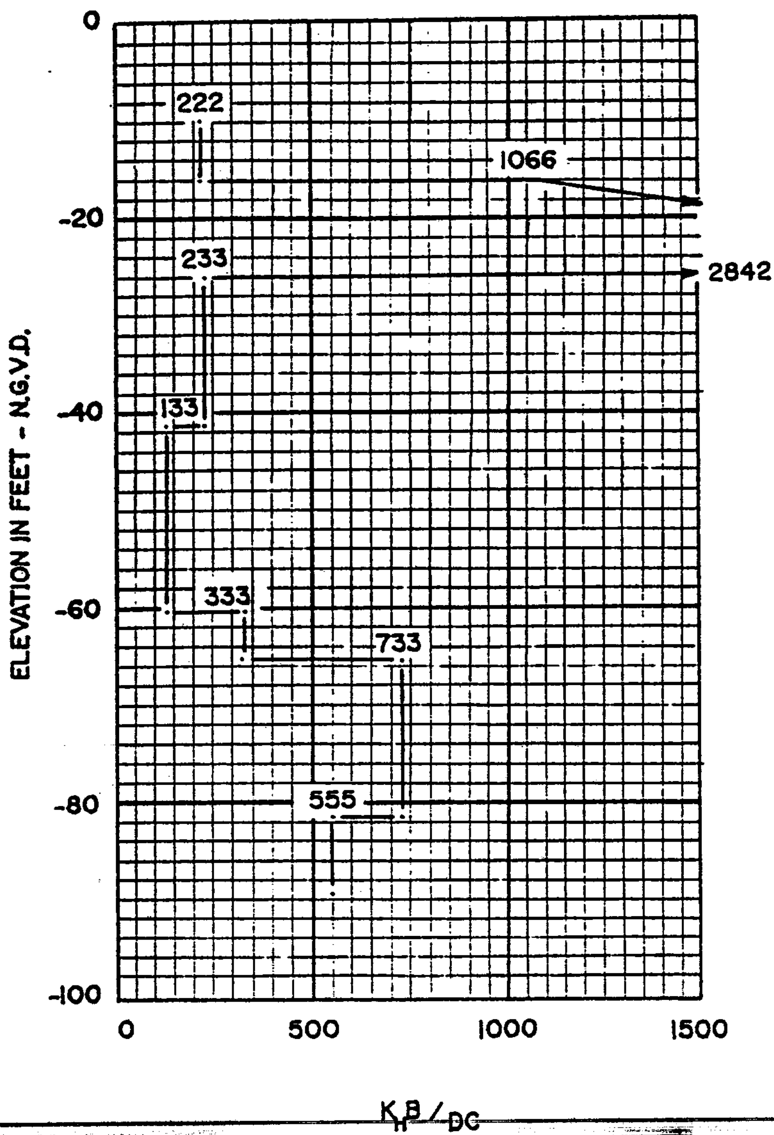
SPACING OF PILE GROUPS

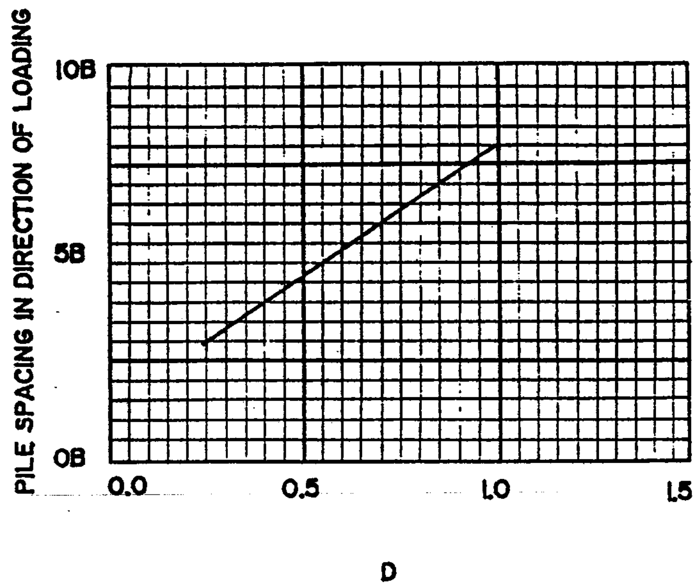
$$SPAC = 0.05\,(L_1) + 0.025\,(L_2) + 0.0125\,(L_3)$$

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
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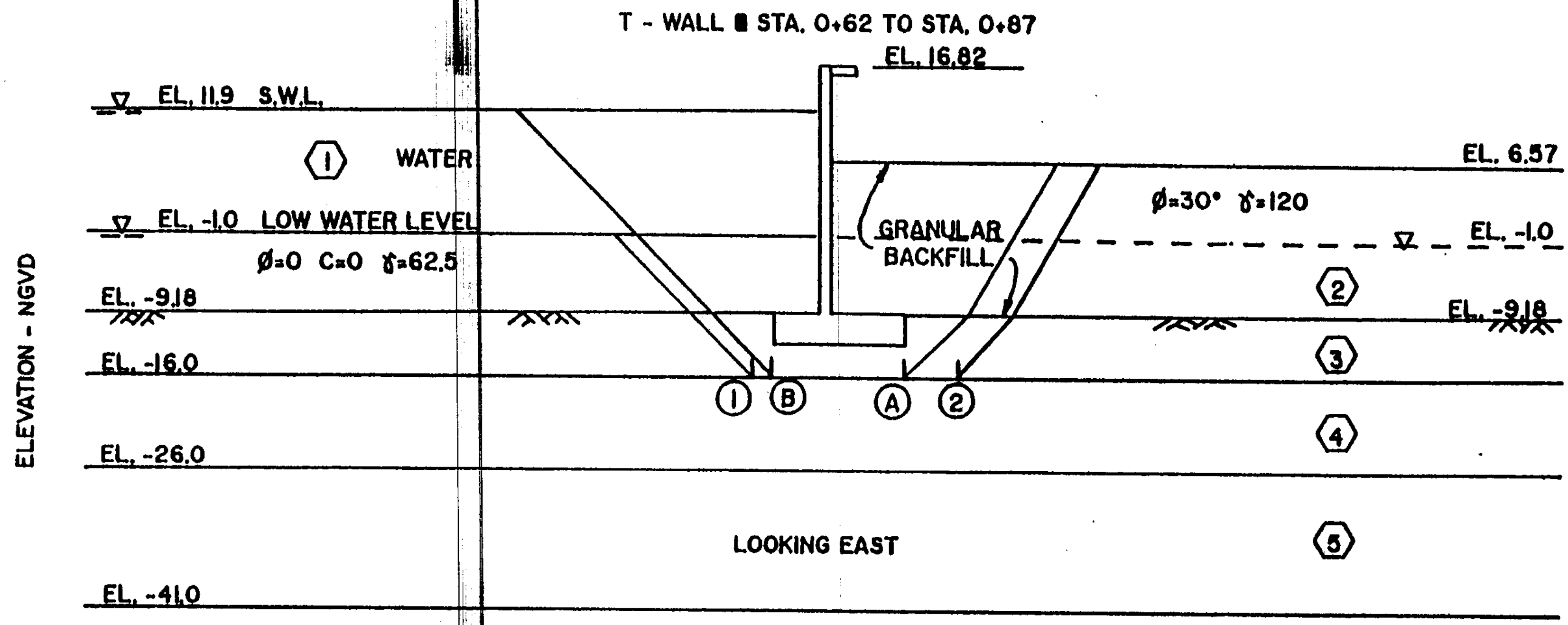
EUSTIS ENGINEERING COMPANY, INC.

FIGURE 6



SCALE : 1" = 20'

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

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

* PASSIVE DRIVING FORCES > ACTIVE DRIVING FORCES

STABILITY ANALYSES

LONDON AVENUE OUTFALL CANAL
FRONTAL PROTECTION AT
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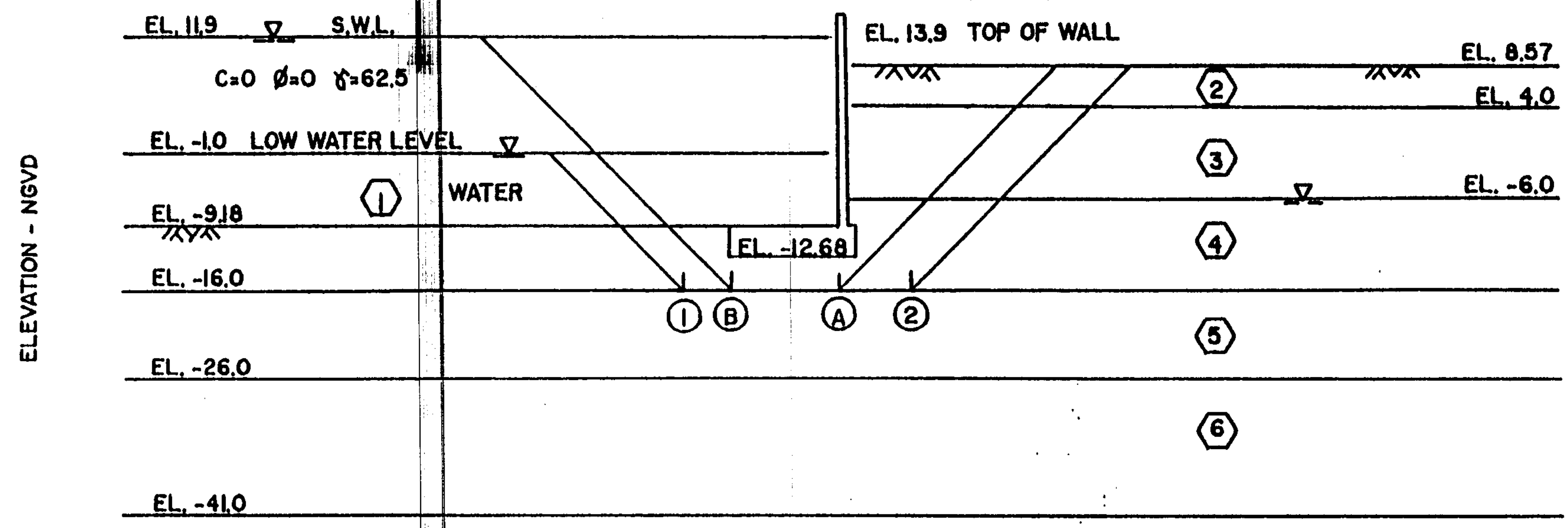
EUSTIS ENGINEERING COMPANY, INC.

FIGURE 7

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



LOOKING SOUTH
SCALE : 1" = 20'

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

⬡ STRATA NUMBER, SEE FIGURE 2 FOR SOIL DESIGN PARAMETERS

* PASSIVE DRIVING FORCES > ACTIVE DRIVING FORCES

STABILITY ANALYSES

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

FIGURE 8

EUSTIS ENGINEERING COMPANY, INC.

APPENDIX I

LEGEND AND NOTES FOR LOG OF BORING AND TEST RESULTS

PP	Pocket penetrometer resistance in tons per square foot
TV	Torvane shear strength in tons per square foot
SPT	Standard Penetration Test. Number of blows of a 140-lb. hammer dropped 30 inches required to drive 2-in O.D., 1.4-in. I.D. sampler a distance of one foot into the soil, after first seating it 6 inches
SPLR	Type of Sampling ■ Shelby ⊠ SPT ⧄ Auger ☐ No Sample
SYMBOL	Clay Silt Sand Humus — Predominant type shown heavy; Modifying type shown light
DENSITY	Unit weight in pounds per cubic foot
USC	Unified Soil Classification

TYPE

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

ϕ Angle of internal friction in degrees

c Cohesion in pounds per square foot

Other laboratory test results reported on separate figure

Ground Water Measurements ▼ Initial ▽ Final

GENERAL NOTES

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

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

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

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

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

Scale In Feet	PP	SPT	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density Dry	Density Wet	Shear Tests Type	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					
	0.30		Very loose gray clayey silt w/clay layers	ML	8	18-19	43	77	110	OB	--	210					
20		19	Medium dense gray fine sand	SP	9	21-22											
25		7	Medium stiff gray clay w/silt layers	CH	10	24-25											
		5			11	27-28	59										
30																	
35	0.20		Soft gray clay w/silt lenses	CH	12	33-34	68	60	100	UC	--	380	95	28	67		
40	0.20		Soft dark gray clay	CH	13	38-39	69	59	100	UC	--	495					
45			Soft gray sandy clay w/clayey sand layers & shell fragments	CL	14	43-44	33	89	119	OB	--	340					
			w/shell fragments		15	48-49											
50			Medium dense gray silty sand	SM													

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

LONDON AVENUE OUTFALL CANAL, FRONTAL PROTECTION AT PUMPING STATION NO. 3 (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	S P L R	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density		Shear Tests			Atterberg Limits			% Passing No. 200 Sieve	Other Tests
										Dry	Wet	Type	Ø	C	LL	PL	PI		
		25			Medium dense gray silty sand	SM	16	51-52											PD
55		13					17	54-55											
60		6			Medium stiff gray clay w/silt layers	CH	18	59-60											
					Soft gray clay w/shell fragments & silty sand layers	CH	19	63-64	52	73	111	OB	--	485	85	26	59		
65					Medium stiff gray & tan silty clay w/fine sand layers	CL													
							20	68-69	27										
70																			
	2.20				Stiff greenish-gray & tan silty clay	CL	21	73-74	28	95	121	OB	--	1665					
75					Stiff greenish-gray & tan clay	CH													
	2.80						22	78-79	39	82	114	UC	--	1655					
80																			
	1.25						23	83-84	36	88	119	UC	--	1325	57	21	36		
85																			
	2.20						24	87-88	33	89	118	UC	--	1205					
90		40			Compact tan & gray clayey silt	ML	25	89-90										88.5	
95		19			Medium dense greenish-gray fine sand	SP	26	94-95											
100		10			Stiff gray clay	CH	27	99-100	50										

EUSTIS ENGINEERING COMPANY, INC.

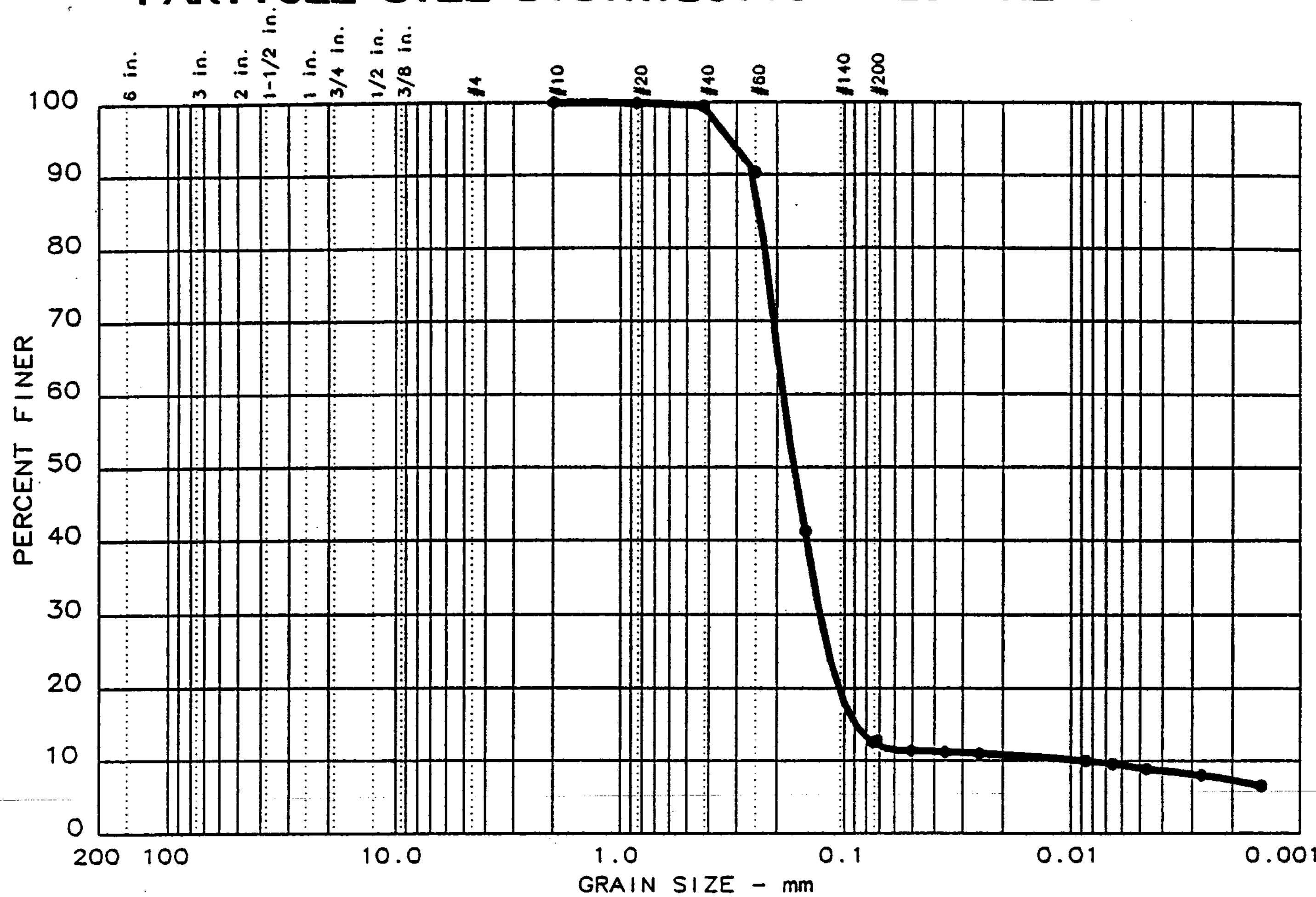
LOG OF BORING AND TEST RESULTS

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

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

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



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

SIEVE inches size	PERCENT FINER ●		

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

	COEFFICIENTS		
C_c	10.16		
C_u	21.5		

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

Sample information:

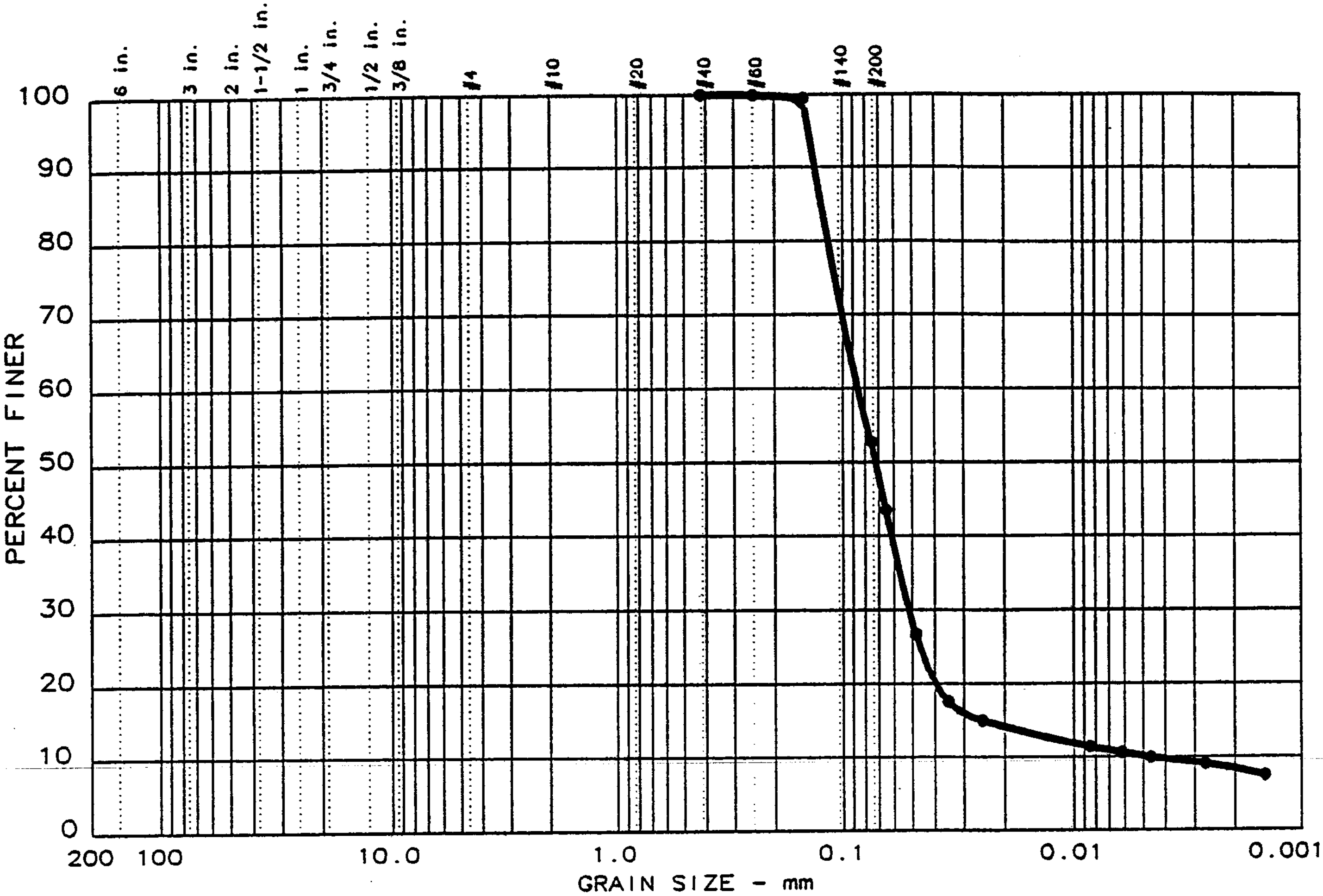
● Boring 1 Sample 16
Gray Silty Sand
w/tr shell frag & om

Remarks:
Sample depth 51'-52'

Eustis Engineering Company, Inc.

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



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

SIEVE inches size	PERCENT FINER ●		

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

	COEFFICIENTS		
C_c	6.54		
C_u	18.0		

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

Sample information:

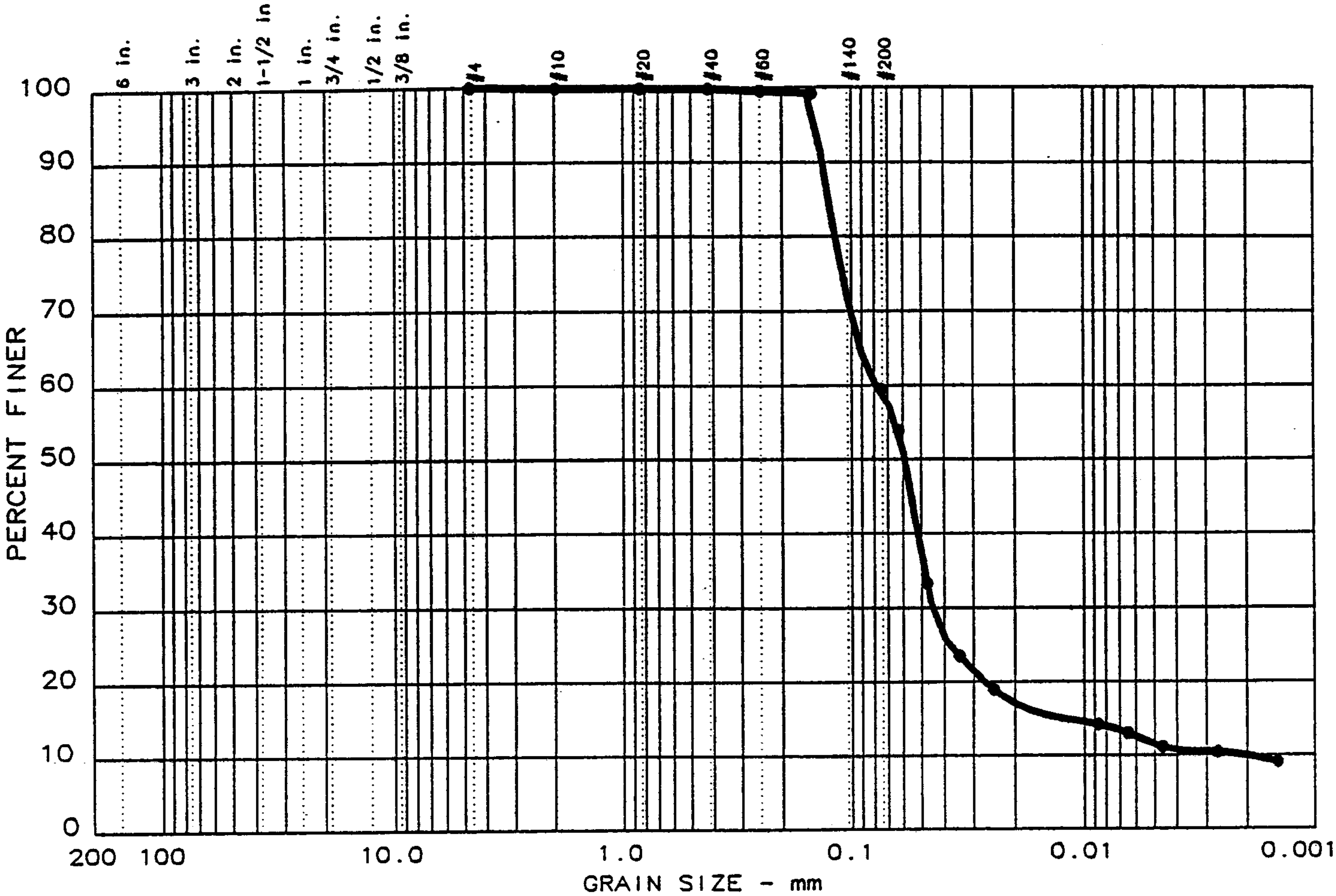
● Boring 1, Sample 25
Gray Sandy Silt

Remarks:
Sample depth 104'-105'

Eustis Engineering Company, Inc.

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



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

SIEVE inches size	PERCENT FINER ●		

	GRAIN SIZE		
D_{60}	0.08		
D_{30}	0.04		
D_{10}	0.00		

	COEFFICIENTS		
C_c	13.49		
C_u	40.7		

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

Sample information:

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

Remarks:
Sample depth 109'-110'

Eustis Engineering Company, Inc.

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

APPENDIX II

13

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

DATE: 08-SEP-1994 TIME: 7.30.41

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¤ INPUT DATA ¤
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I.--HEADING:
'LONDON AVE CANAL 13065
'TEMPORARY COFFERDAM

II.--CONTROL
ANCHORED 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 = 6.00 (FT)
ELEVATION AT ANCHOR = 2.00 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

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

IV.B--LEFTSIDE

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

V.--SOIL LAYER DATA

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

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
95.00	95.00	18.00	.0	.00	.0	-12.00	.00	DEF	DEF
110.00	110.00	23.00	.0	.00	.0	-16.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-26.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-31.00	.00	DEF	DEF

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

V.B.-- LEFTSIDE LAYER DATA

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

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
95.00	95.00	18.00	.0	.00	.0	-12.00	.00	DEF	DEF
110.00	110.00	23.00	.0	.00	.0	-16.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-26.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-31.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-36.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-41.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-60.00	.00	DEF	DEF
110.00	110.00	23.00	.0	.00	.0	-65.00	.00	DEF	DEF
119.00	119.00	25.00	.0	.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 2.00 (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: 08-SEP-1994 TIME: 7.31.28

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¤ SUMMARY OF RESULTS FOR ¤
¤ ANCHORED WALL DESIGN ¤

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I.--HEADING

'LONDON AVE CANAL 13065
'TEMPORARY COFFERDAM

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.

METHOD	:	FREE EARTH	EQUIV. BEAM	FIXED EARTH
WALL BOTTOM ELEV. (FT)	:	-33.41	-45.36	-42.26
PENETRATION (FT)	:	23.41	35.36	32.26
MAX. BEND. MOMENT (LB-FT)	:	-51433.	42345.	-43239.
AT ELEVATION (FT)	:	-11.66	-35.33	-10.79
MAX. SCALED DEFL. (LB-IN3)	:	9.9082E+09	3.5138E+09	7.4559E+09
AT ELEVATION (FT)	:	-14.00	-10.00	-13.00
ANCHOR FORCE (LB)	:	5704.	4540.	5084.

(NOTE: PENETRATION FOR EQUIVALENT BEAM METHOD DOES NOT INCLUDE INCREASE PRESCRIBED BY DRAFT EM 1110-2-2906.)

(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: 08-SEP-1994 TIME: 7.31.28

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¤ COMPLETE RESULTS FOR ¤
¤ ANCHORED WALL DESIGN ¤
¤ BY FREE EARTH METHOD ¤
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I.--HEADING

'LONDON AVE CANAL 13065
'TEMPORARY COFFERDAM

II.--RESULTS (ANCHOR FORCE = 5704. (LB))

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
6.00	0.	0.	-3.9236E+09	.00
5.00	0.	0.	-2.9427E+09	.00
4.00	0.	0.	-1.9618E+09	.00
3.00	0.	0.	-9.8089E+08	.00
2.00	0.	0.	0.0000E+00	.00
2.00	0.	-5704.	0.0000E+00	.00
1.00	-5693.	-5673.	9.7925E+08	62.50
.00	-11324.	-5579.	1.9487E+09	125.00
-1.00	-16830.	-5423.	2.8985E+09	187.50
-2.00	-22148.	-5204.	3.8194E+09	250.00
-3.00	-27217.	-4923.	4.7019E+09	312.50
-4.00	-31973.	-4579.	5.5375E+09	375.00
-5.00	-36353.	-4173.	6.3179E+09	437.50
-6.00	-40297.	-3704.	7.0356E+09	500.00
-7.00	-43740.	-3173.	7.6837E+09	562.50
-8.00	-46621.	-2579.	8.2562E+09	625.00
-9.00	-48877.	-1923.	8.7484E+09	687.50
-10.00	-50445.	-1204.	9.1561E+09	750.00
-11.00	-51279.	-468.	9.4768E+09	721.19
-12.00	-51393.	234.	9.7090E+09	682.95
-13.00	-50828.	884.	9.8524E+09	617.66
-14.00	-49644.	1474.	9.9082E+09	561.78
-15.00	-47899.	2008.	9.8782E+09	505.90
-16.00	-45650.	2478.	9.7655E+09	433.98
-17.00	-42971.	2866.	9.5740E+09	343.07
-18.00	-39945.	3172.	9.3083E+09	268.19
-19.00	-36652.	3403.	8.9737E+09	193.31
-20.00	-33165.	3558.	8.5757E+09	118.44
-21.00	-29560.	3639.	8.1204E+09	43.56
-21.58	-27438.	3652.	7.8316E+09	.00
-21.79	-26674.	3651.	7.7238E+09	-15.66
-22.00	-25911.	3646.	7.6140E+09	-31.31
-23.00	-22294.	3577.	7.0629E+09	-106.19
-24.00	-18782.	3433.	6.4732E+09	-181.06
-25.00	-15452.	3215.	5.8511E+09	-255.94
-26.00	-12369.	2947.	5.2022E+09	-278.61
-27.00	-9560.	2672.	4.5319E+09	-271.69
-28.00	-7031.	2378.	3.8450E+09	-316.98
-29.00	-4819.	2038.	3.1459E+09	-362.27
-30.00	-2969.	1653.	2.4385E+09	-407.56
-31.00	-1527.	1223.	1.7259E+09	-452.85
-32.00	-538.	748.	1.0105E+09	-498.14
-33.00	-47.	227.	2.9419E+08	-543.43
-33.41	0.	0.	0.0000E+00	-562.03

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

III.--SOIL PRESSURES

ELEVATION (FT)	< LEFTSIDE PRESSURE (PSF)> PASSIVE	ACTIVE	<RIGHTSIDE PRESSURE (PSF)> ACTIVE	PASSIVE
6.00	0.	0.	0.	0.
5.00	0.	0.	0.	0.
4.00	0.	0.	0.	0.
3.00	0.	0.	0.	0.
2.00	0.	0.	0.	0.
1.00	0.	0.	0.	0.
.00	0.	0.	0.	0.
-1.00	0.	0.	0.	0.
-2.00	0.	0.	0.	0.
-3.00	0.	0.	0.	0.
-4.00	0.	0.	0.	0.
-5.00	0.	0.	0.	0.
-6.00	0.	0.	0.	0.
-7.00	0.	0.	0.	0.
-8.00	0.	0.	0.	0.
-9.00	0.	0.	0.	0.
-10.00	0.	0.	0.	0.
-11.00	50.	21.	21.	50.
-12.00+	100.	42.	42.	100.
-12.00-	114.	37.	37.	114.
-13.00	197.	64.	64.	197.
-14.00	280.	92.	92.	280.
-15.00	363.	119.	119.	363.
-16.00+	446.	146.	146.	446.
-16.00-	470.	138.	138.	470.
-17.00	576.	169.	169.	576.
-18.00	682.	201.	201.	682.
-19.00	788.	232.	232.	788.
-20.00	895.	263.	263.	895.
-21.00	1001.	294.	294.	1001.
-21.58	1062.	312.	312.	1062.
-21.79	1084.	319.	319.	1084.
-22.00	1107.	325.	325.	1107.
-23.00	1213.	356.	356.	1213.
-24.00	1319.	388.	388.	1319.
-25.00	1425.	419.	419.	1425.
-26.00+	1531.	450.	450.	1531.
-26.00-	1451.	475.	475.	1451.
-27.00	1518.	497.	497.	1518.
-28.00	1586.	519.	519.	1586.
-29.00	1653.	541.	541.	1653.
-30.00	1720.	563.	563.	1720.
-31.00	1788.	585.	585.	1788.
-32.00	1855.	607.	607.	1855.
-33.00	1922.	629.	629.	1922.
-34.00	1990.	651.	651.	1990.

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

DATE: 08-SEP-1994 TIME: 7.34.42

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¤ INPUT DATA ¤
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I.--HEADING:
'LONDON AVE CANAL 13065
'TEMPORARY COFFERDAM

II.--CONTROL
ANCHORED 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 = 6.00 (FT)
ELEVATION AT ANCHOR = 2.00 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

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

IV.B-- LEFTSIDE

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

V.--SOIL LAYER DATA

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

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
95.00	95.00	18.00	.0	.00	.0	-12.00	.00	DEF	DEF
110.00	110.00	23.00	.0	.00	.0	-16.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-26.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-31.00	.00	DEF	DEF

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

V.B.-- LEFTSIDE LAYER DATA

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

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
95.00	95.00	18.00	.0	.00	.0	-12.00	.00	DEF	DEF
110.00	110.00	23.00	.0	.00	.0	-16.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-26.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-31.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-36.00	.00	DEF	DEF
101.00	101.00	23.00	.0	.00	.0	-41.00	.00	DEF	DEF
120.00	120.00	25.00	.0	.00	.0	-60.00	.00	DEF	DEF
110.00	110.00	23.00	.0	.00	.0	-65.00	.00	DEF	DEF
119.00	119.00	25.00	.0	.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 2.00 (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: 08-SEP-1994 TIME: 7.34.54

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¤ SUMMARY OF RESULTS FOR ¤
¤ ANCHORED WALL DESIGN ¤

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I.--HEADING

'LONDON AVE CANAL 13065
'TEMPORARY COFFERDAM

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.

METHOD	:	FREE EARTH	EQUIV. BEAM	FIXED EARTH
WALL BOTTOM ELEV. (FT)	:	-25.59	-33.72	-32.29
PENETRATION (FT)	:	15.59	23.72	22.29
MAX. BEND. MOMENT (LB-FT)	:	-35619.	-25496.	-28562.
AT ELEVATION (FT)	:	-9.96	-8.70	-9.11
MAX. SCALED DEFL. (LB-IN3)	:	4.3340E+09	1.7546E+09	3.1568E+09
AT ELEVATION (FT)	:	-11.00	-8.00	-10.00
ANCHOR FORCE (LB)	:	4468.	3575.	3857.

(NOTE: PENETRATION FOR EQUIVALENT BEAM METHOD DOES NOT INCLUDE INCREASE PRESCRIBED BY DRAFT EM 1110-2-2906.)

(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: 08-SEP-1994 TIME: 7.34.54

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¤ COMPLETE RESULTS FOR ¤
¤ ANCHORED WALL DESIGN ¤
¤ BY FREE EARTH METHOD ¤
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I.--HEADING

'LONDON AVE CANAL 13065
'TEMPORARY COFFERDAM

II.--RESULTS (ANCHOR FORCE = 4468. (LB))

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
6.00	0.	0.	-2.1006E+09	.00
5.00	0.	0.	-1.5755E+09	.00
4.00	0.	0.	-1.0503E+09	.00
3.00	0.	0.	-5.2515E+08	.00
2.00	0.	0.	0.0000E+00	.00
2.00	0.	-4468.	0.0000E+00	.00
1.00	-4458.	-4437.	5.2387E+08	62.50
.00	-8853.	-4343.	1.0400E+09	125.00
-1.00	-13123.	-4187.	1.5409E+09	187.50
-2.00	-17206.	-3968.	2.0192E+09	250.00
-3.00	-21039.	-3687.	2.4677E+09	312.50
-4.00	-24559.	-3343.	2.8800E+09	375.00
-5.00	-27705.	-2937.	3.2498E+09	437.50
-6.00	-30412.	-2468.	3.5719E+09	500.00
-7.00	-32620.	-1937.	3.8414E+09	562.50
-8.00	-34265.	-1343.	4.0547E+09	625.00
-9.00	-35286.	-687.	4.2089E+09	687.50
-10.00	-35619.	32.	4.3021E+09	750.00
-11.00	-35219.	760.	4.3340E+09	705.59
-12.00	-34117.	1435.	4.3051E+09	645.64
-13.00	-32376.	2029.	4.2173E+09	542.49
-14.00	-30090.	2528.	4.0736E+09	454.88
-15.00	-27349.	2939.	3.8780E+09	367.26
-16.00	-24246.	3249.	3.6353E+09	252.42
-17.00	-20895.	3428.	3.3506E+09	106.86
-17.90	-17770.	3477.	3.0624E+09	.00
-17.95	-17601.	3477.	3.0462E+09	-5.74
-18.00	-17433.	3476.	3.0299E+09	-11.48
-19.00	-13982.	3406.	2.6790E+09	-129.82
-20.00	-10661.	3217.	2.3040E+09	-248.16
-21.00	-7588.	2909.	1.9105E+09	-366.49
-22.00	-4882.	2484.	1.5038E+09	-484.83
-23.00	-2661.	1940.	1.0886E+09	-603.17
-24.00	-1043.	1277.	6.6877E+08	-721.51
-25.00	-146.	497.	2.4701E+08	-839.85
-25.59	0.	0.	0.0000E+00	-857.24

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

III.--SOIL PRESSURES

ELEVATION (FT)	< LEFTSIDE PRESSURE (PSF) >		<RIGHTSIDE PRESSURE (PSF) >	
	PASSIVE	ACTIVE	ACTIVE	PASSIVE
6.00	0.	0.	0.	0.
5.00	0.	0.	0.	0.
4.00	0.	0.	0.	0.

3.00	0.	0.	0.	0.
2.00	0.	0.	0.	0.
1.00	0.	0.	0.	0.
.00	0.	0.	0.	0.
-1.00	0.	0.	0.	0.
-2.00	0.	0.	0.	0.
-3.00	0.	0.	0.	0.
-4.00	0.	0.	0.	0.
-5.00	0.	0.	0.	0.
-6.00	0.	0.	0.	0.
-7.00	0.	0.	0.	0.
-8.00	0.	0.	0.	0.
-9.00	0.	0.	0.	0.
-10.00	0.	0.	0.	0.
-11.00	62.	17.	17.	62.
-12.00+	123.	34.	34.	123.
-12.00-	148.	28.	28.	148.
-13.00	257.	49.	49.	257.
-14.00	365.	70.	70.	365.
-15.00	474.	91.	91.	474.
-16.00+	582.	112.	112.	582.
-16.00-	628.	103.	103.	628.
-17.00	770.	127.	127.	770.
-17.90	898.	148.	148.	898.
-17.95	905.	149.	149.	905.
-18.00	912.	150.	150.	912.
-19.00	1053.	174.	174.	1053.
-20.00	1195.	197.	197.	1195.
-21.00	1337.	220.	220.	1337.
-22.00	1478.	244.	244.	1478.
-23.00	1620.	267.	267.	1620.
-24.00	1762.	290.	290.	1762.
-25.00	1903.	314.	314.	1903.
-26.00+	2045.	337.	337.	2045.
-26.00-	1895.	364.	364.	1895.

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

DATE: 08-SEP-1994 TIME: 7.42.18

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¤ INPUT DATA ¤
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I.--HEADING:
'LONDON AVE CANAL 13065
'TEMPORARY COFFERDAM

II.--CONTROL
ANCHORED 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 = 6.00 (FT)
ELEVATION AT ANCHOR = 2.00 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

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

IV.B-- LEFTSIDE

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

V.--SOIL LAYER DATA

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

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

101.00	101.00	.00	525.0	.00	.0	-36.00	.00	DEF	DEF
101.00	101.00	.00	575.0	.00	.0	-41.00	.00	DEF	DEF
120.00	120.00	15.00	300.0	.00	.0	-60.00	.00	DEF	DEF
110.00	110.00	.00	750.0	.00	.0	-65.00	.00	DEF	DEF
119.00	119.00	.00	1650.0	.00	.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA

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

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

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 2.00 (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: 08-SEP-1994 TIME: 7.42.48

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¤ SUMMARY OF RESULTS FOR ¤
¤ ANCHORED WALL DESIGN ¤

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I.--HEADING

'LONDON AVE CANAL 13065
'TEMPORARY COFFERDAM

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.

METHOD		FREE EARTH	EQUIV. BEAM	FIXED EARTH
WALL BOTTOM ELEV. (FT)	:	-29.25	-45.29	-36.95
PENETRATION (FT)	:	19.25	35.29	26.95
MAX. BEND. MOMENT (LB-FT)	:	-31493.	62172.	-26410.
AT ELEVATION (FT)	:	-9.48	-32.51	-8.82
MAX. SCALED DEFL. (LB-IN3)	:	4.7944E+09	2.7086E+10	3.5430E+09
AT ELEVATION (FT)	:	-12.00	-45.29	-11.00
ANCHOR FORCE (LB)	:	4116.	2230.	3660.

(NOTE: PENETRATION FOR EQUIVALENT BEAM METHOD DOES NOT INCLUDE INCREASE PRESCRIBED BY DRAFT EM 1110-2-2906.)

(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: 08-SEP-1994 TIME: 7.42.48

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¤ COMPLETE RESULTS FOR ¤
¤ ANCHORED WALL DESIGN ¤
¤ BY FREE EARTH METHOD ¤
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I.--HEADING

'LONDON AVE CANAL 13065
'TEMPORARY COFFERDAM

II.--RESULTS (ANCHOR FORCE = 4116. (LB))

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
6.00	0.	0.	-2.1610E+09	.00
5.00	0.	0.	-1.6207E+09	.00
4.00	0.	0.	-1.0805E+09	.00
3.00	0.	0.	-5.4024E+08	.00
2.00	0.	0.	0.0000E+00	.00
2.00	0.	-4116.	0.0000E+00	.00
1.00	-4106.	-4085.	5.3906E+08	62.50
.00	-8149.	-3991.	1.0710E+09	125.00
-1.00	-12067.	-3835.	1.5889E+09	187.50
-2.00	-15798.	-3616.	2.0860E+09	250.00
-3.00	-19279.	-3335.	2.5558E+09	312.50
-4.00	-22447.	-2991.	2.9924E+09	375.00
-5.00	-25240.	-2585.	3.3902E+09	437.50
-6.00	-27596.	-2116.	3.7445E+09	500.00
-7.00	-29451.	-1585.	4.0511E+09	562.50
-8.00	-30745.	-991.	4.3070E+09	625.00
-9.00	-31413.	-335.	4.5098E+09	687.50
-10.00	-31393.	384.	4.6584E+09	750.00
-10.00	-31393.	384.	4.6584E+09	616.67
-11.00	-30707.	984.	4.7529E+09	584.17
-12.00	-29480.	1419.	4.7944E+09	285.00
-12.91	-28115.	1548.	4.7879E+09	.00
-12.95	-28043.	1548.	4.7865E+09	-14.58
-13.00	-27971.	1547.	4.7849E+09	-29.17
-14.00	-26447.	1494.	4.7272E+09	-76.67
-15.00	-24999.	1393.	4.6237E+09	-124.17
-16.00	-23627.	1393.	4.4771E+09	123.14
-17.00	-22135.	1626.	4.2896E+09	343.07
-18.00	-20350.	1932.	4.0639E+09	268.19
-19.00	-18297.	2162.	3.8031E+09	193.31
-20.00	-16050.	2318.	3.5107E+09	118.44
-21.00	-13685.	2399.	3.1906E+09	43.56
-22.00	-11277.	2405.	2.8468E+09	-31.31
-23.00	-8899.	2337.	2.4836E+09	-106.19
-24.00	-6628.	2193.	2.1049E+09	-181.06
-25.00	-4538.	1975.	1.7148E+09	-255.94
-26.00	-2720.	1635.	1.3168E+09	-423.74
-27.00	-1312.	1165.	9.1406E+08	-516.67
-28.00	-406.	648.	5.0896E+08	-516.67
-29.00	-17.	131.	1.0308E+08	-516.67
-29.25	0.	0.	0.0000E+00	-516.67

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

III.--SOIL PRESSURES

ELEVATION (FT)	< LEFTSIDE PRESSURE (PSF)> PASSIVE	ACTIVE	<RIGHTSIDE PRESSURE (PSF)> ACTIVE	PASSIVE
6.00	0.	0.	0.	0.
5.00	0.	0.	0.	0.
4.00	0.	0.	0.	0.
3.00	0.	0.	0.	0.
2.00	0.	0.	0.	0.
1.00	0.	0.	0.	0.
.00	0.	0.	0.	0.
-1.00	0.	0.	0.	0.
-2.00	0.	0.	0.	0.
-3.00	0.	0.	0.	0.
-4.00	0.	0.	0.	0.
-5.00	0.	0.	0.	0.
-6.00	0.	0.	0.	0.
-7.00	0.	0.	0.	0.
-8.00	0.	0.	0.	0.
-9.00	0.	0.	0.	0.
-10.00+	0.	0.	0.	0.
-10.00-	133.	0.	0.	133.
-11.00	166.	0.	0.	166.
-12.00+	198.	0.	0.	198.
-12.00-	732.	0.	0.	732.
-12.91	775.	0.	0.	775.
-12.95	777.	0.	0.	777.
-13.00	779.	0.	0.	779.
-14.00	827.	0.	0.	827.
-15.00	874.	0.	0.	874.
-16.00+	922.	0.	0.	922.
-16.00-	470.	138.	138.	470.
-17.00	576.	169.	169.	576.
-18.00	682.	201.	201.	682.
-19.00	788.	232.	232.	788.
-20.00	895.	263.	263.	895.
-21.00	1001.	294.	294.	1001.
-22.00	1107.	325.	325.	1107.
-23.00	1213.	356.	356.	1213.
-24.00	1319.	388.	388.	1319.
-25.00	1425.	419.	419.	1425.
-26.00+	1531.	450.	450.	1531.
-26.00-	1463.	197.	197.	1463.
-27.00	1502.	235.	235.	1502.
-28.00	1540.	274.	274.	1540.
-29.00	1579.	312.	312.	1579.
-30.00	1617.	351.	351.	1617.

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

DATE: 08-SEP-1994 TIME: 7.46.27

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¤ INPUT DATA ¤
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I.--HEADING:
'LONDON AVE CANAL 13065
'TEMPORARY COFFERDAM

II.--CONTROL
ANCHORED 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 = 6.00 (FT)
ELEVATION AT ANCHOR = 2.00 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

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

IV.B-- LEFTSIDE

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

V.--SOIL LAYER DATA

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

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

101.00	101.00	.00	525.0	.00	.0	-36.00	.00	DEF	DEF
101.00	101.00	.00	575.0	.00	.0	-41.00	.00	DEF	DEF
120.00	120.00	15.00	300.0	.00	.0	-60.00	.00	DEF	DEF
110.00	110.00	.00	750.0	.00	.0	-65.00	.00	DEF	DEF
119.00	119.00	.00	1650.0	.00	.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA

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

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

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 2.00 (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: 08-SEP-1994 TIME: 7.46.39

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¤ SUMMARY OF RESULTS FOR ¤
¤ ANCHORED WALL DESIGN ¤

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I.--HEADING

'LONDON AVE CANAL 13065
'TEMPORARY COFFERDAM

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.

METHOD	:	FREE EARTH	EQUIV. BEAM	FIXED EARTH
WALL BOTTOM ELEV. (FT)	:	-22.36	-30.68	-27.53
PENETRATION (FT)	:	12.36	20.68	17.53
MAX. BEND. MOMENT (LB-FT)	:	-18929.	22294.	-15799.
AT ELEVATION (FT)	:	-7.69	-23.81	-7.12
MAX. SCALED DEFL. (LB-IN3)	:	1.6572E+09	2.0987E+09	1.2129E+09
AT ELEVATION (FT)	:	-9.00	-30.68	-8.00
ANCHOR FORCE (LB)	:	2932.	2044.	2599.

(NOTE: PENETRATION FOR EQUIVALENT BEAM METHOD DOES NOT INCLUDE INCREASE PRESCRIBED BY DRAFT EM 1110-2-2906.)

(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: 08-SEP-1994 TIME: 7.46.39

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¤ COMPLETE RESULTS FOR ¤
¤ ANCHORED WALL DESIGN ¤
¤ BY FREE EARTH METHOD ¤
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I.--HEADING

'LONDON AVE CANAL 13065
'TEMPORARY COFFERDAM

II.--RESULTS (ANCHOR FORCE = 2932. (LB))

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
6.00	0.	0.	-9.5855E+08	.00
5.00	0.	0.	-7.1891E+08	.00
4.00	0.	0.	-4.7928E+08	.00
3.00	0.	0.	-2.3964E+08	.00
2.00	0.	0.	0.0000E+00	.00
2.00	0.	-2931.	0.0000E+00	.00
1.00	-2921.	-2900.	2.3879E+08	62.50
.00	-5780.	-2806.	4.7255E+08	125.00
-1.00	-8513.	-2650.	6.9634E+08	187.50
-2.00	-11059.	-2431.	9.0544E+08	250.00
-3.00	-13355.	-2150.	1.0955E+09	312.50
-4.00	-15339.	-1806.	1.2625E+09	375.00
-5.00	-16948.	-1400.	1.4030E+09	437.50
-6.00	-18119.	-931.	1.5143E+09	500.00
-7.00	-18790.	-400.	1.5944E+09	562.50
-8.00	-18898.	194.	1.6421E+09	625.00
-9.00	-18382.	850.	1.6572E+09	687.50
-10.00	-17178.	1569.	1.6407E+09	750.00
-10.00	-17178.	1569.	1.6407E+09	550.00
-11.00	-15340.	2102.	1.5946E+09	517.50
-12.00	-13051.	2404.	1.5220E+09	85.00
-12.19	-12593.	2412.	1.5056E+09	.00
-12.59	-11622.	2375.	1.4679E+09	-181.25
-13.00	-10679.	2265.	1.4269E+09	-362.50
-14.00	-8604.	1879.	1.3133E+09	-410.00
-15.00	-6938.	1445.	1.1848E+09	-457.50
-16.00	-5669.	1146.	1.0442E+09	-139.90
-17.00	-4552.	1130.	8.9379E+08	106.86
-18.00	-3389.	1177.	7.3554E+08	-11.48
-19.00	-2237.	1107.	5.7143E+08	-129.82
-20.00	-1215.	918.	4.0343E+08	-248.16
-21.00	-441.	610.	2.3330E+08	-366.49
-22.00	-34.	185.	6.2347E+07	-484.83
-22.36	0.	0.	0.0000E+00	-527.96

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

III.--SOIL PRESSURES

ELEVATION (FT)	< LEFTSIDE PRESSURE (PSF)> PASSIVE	ACTIVE	<RIGHTSIDE PRESSURE (PSF)> ACTIVE	PASSIVE
6.00	0.	0.	0.	0.
5.00	0.	0.	0.	0.
4.00	0.	0.	0.	0.
3.00	0.	0.	0.	0.
2.00	0.	0.	0.	0.

1.00	0.	0.	0.	0.
.00	0.	0.	0.	0.
-1.00	0.	0.	0.	0.
-2.00	0.	0.	0.	0.
-3.00	0.	0.	0.	0.
-4.00	0.	0.	0.	0.
-5.00	0.	0.	0.	0.
-6.00	0.	0.	0.	0.
-7.00	0.	0.	0.	0.
-8.00	0.	0.	0.	0.
-9.00	0.	0.	0.	0.
-10.00+	0.	0.	0.	0.
-10.00-	200.	0.	0.	200.
-11.00	233.	0.	0.	233.
-12.00+	265.	0.	0.	265.
-12.00-	1065.	0.	0.	1065.
-12.19	1074.	0.	0.	1074.
-12.59	1093.	0.	0.	1093.
-13.00	1113.	0.	0.	1113.
-14.00	1160.	0.	0.	1160.
-15.00	1208.	0.	0.	1208.
-16.00+	1255.	0.	0.	1255.
-16.00-	628.	103.	103.	628.
-17.00	770.	127.	127.	770.
-18.00	912.	150.	150.	912.
-19.00	1053.	174.	174.	1053.
-20.00	1195.	197.	197.	1195.
-21.00	1337.	220.	220.	1337.
-22.00	1478.	244.	244.	1478.
-23.00	1620.	267.	267.	1620.

APPENDIX III

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

I-WALL ANALYSES

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

Q-CASE

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

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

COMPUTED VALUE (CV) IS DEEPEST PENETRATION ABOVE.

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

3:1 PENETRATION TO HEAD RATIO

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

PENETRATION = 3 x 3.33 = 9.99 FEET

TIP EL. -1.42

2.5:1 PENETRATION TO HEAD RATIO NO

HEAD = $\boxed{13.90}$ - 8.57 = 5.33 FEET (USING SWL + 2 FEET FREEBOARD)

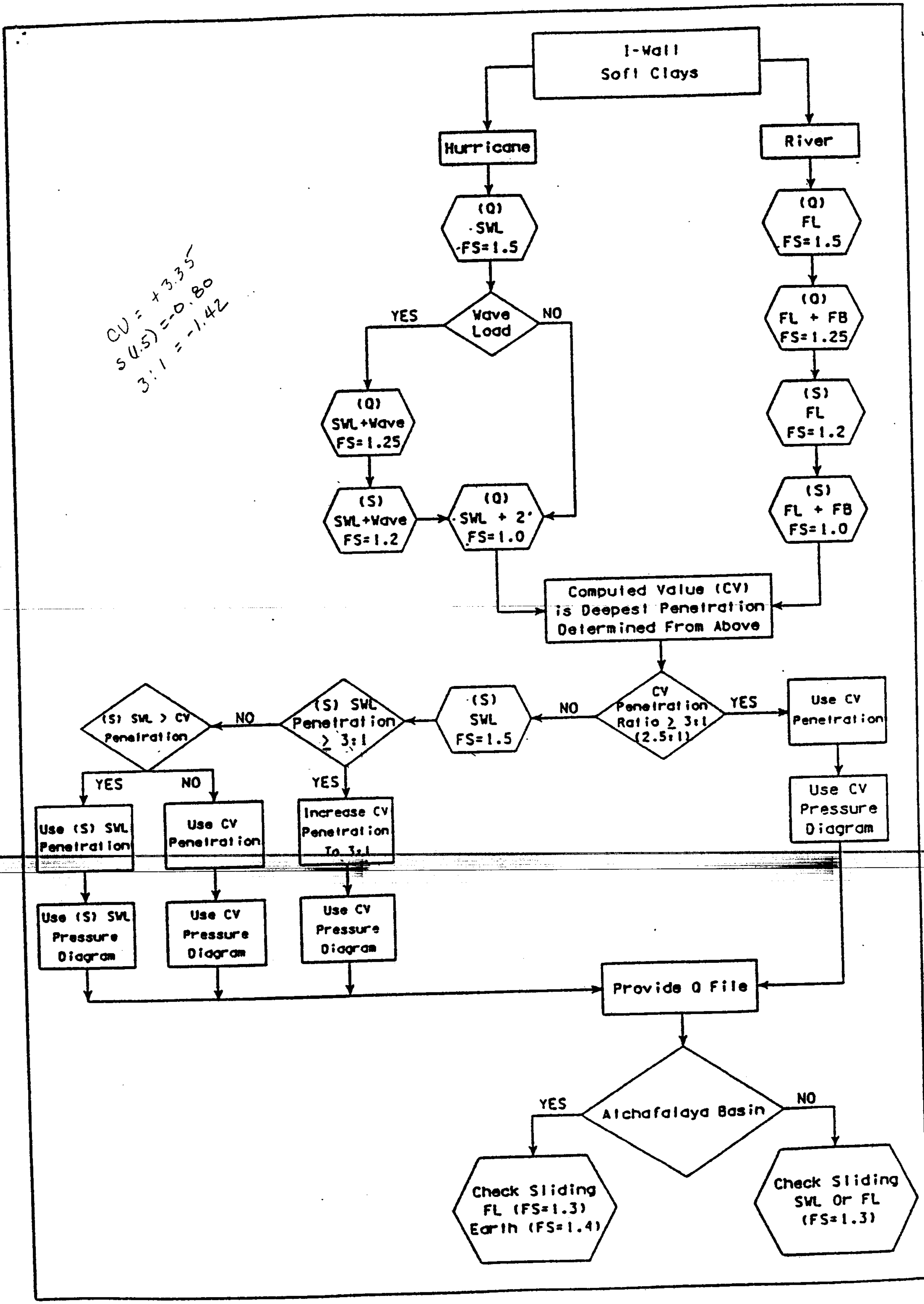
PENETRATION = 2.5 x 5.33 = 13.33 FEET

TIP EL. -4.76

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

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

∴ SINCE TIP EL. -0.80 LESS THAN PENETRATION FOR 3:1 RATIO AND GREATER THAN CV PENETRATION, USE TIP EL -0.80 AND PRESSURE DIAGRAM FOR S-CASE FOR DESIGN. NO


I-Wall
Soft Clays
Hurricane
River
(Q)
SWL
FS=1.5
(Q)
FL
FS=1.5
Wave
Load
YES
NO
(Q)
FL + FB
FS=1.25
(Q)
SWL+Wave
FS=1.25
(S)
FL
FS=1.2
(S)
SWL+Wave
FS=1.2
(Q)
SWL + 2'
FS=1.0
(S)
FL + FB
FS=1.0
Computed Value (CV)
is Deepest Penetration
Determined From Above
CV
Penetration
Ratio ≥ 3:1
(2.5:1)
YES
Use CV
Penetration
Use CV
Pressure
Diagram
NO
(S)
SWL
FS=1.5
(S) SWL
Penetration
≥ 3:1
NO
(S) SWL > CV
Penetration
YES
NO
YES
Use (S) SWL
Penetration
Use CV
Penetration
Increase CV
Penetration
To 3:1
Use (S) SWL
Pressure
Diagram
Use CV
Pressure
Diagram
Use CV
Pressure
Diagram
Provide Q File
Atchafalaya Basin
YES
NO
Check Sliding
FL (FS=1.3)
Earth (FS=1.4)
Check Sliding
SWL Or FL
(FS=1.3)
CV = +3.35
S(1.5) = -0.80
3:1 = -1.42

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

DATE: 26-JAN-1995   TIME: 18.47.25

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

II.--CONTROL
CANTILEVER WALL DESIGN

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

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

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

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

IV.B-- LEFTSIDE

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

V.--SOIL LAYER DATA

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

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

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

V.B.-- LEFTSIDE LAYER DATA

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

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

VI.--WATER DATA

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

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

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

DATE: 26-JAN-1995 TIME: 18.47.52

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

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

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

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

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

MAX. SCALED DEFL. (LB-IN3) : 2.4178E+08
AT ELEVATION (FT) : 14.40

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

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

DATE: 26-JAN-1995 TIME: 18.47.52

COMPLETE RESULTS FOR
CANTILEVER WALL DESIGN

I.--HEADING

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

II.--RESULTS

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

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

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

III.--SOIL PRESSURES

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