ADDITIONAL GEOTECHNICAL ANALYSES

LONDON AVENUE OUTFALL CANAL

PROPOSED I–WALLS AND T–WALLS

MIRABEAU AVENUE TO LEON C. SIMON BOULEVARD

NEW ORLEANS, LOUISIANA

FOR
BURK–KLEINPETER, INC.
ENGINEERS, PLANNERS & ENVIRONMENTAL SCIENTISTS
NEW ORLEANS, LOUISIANA

19 MAY 1993

EUSTIS ENGINEERING
GEOTECHNICAL ENGINEERS
3011 28th Street • Metairie, Louisiana 70002 • 504-834-0157

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

19 May 1993

Burk–Kleinpeter, Inc.
Engineers, Architects,
Planners, Environmental Scientists
4176 Canal Street
Post Office Box 19087
New Orleans, Louisiana 70179

Attention Mr. Bill Giardina

Gentlemen:

Additional Geotechnical Analyses
London Avenue Outfall Canal
Proposed I–Walls and T–Walls
Mirabeau Avenue to Leon C. Simon Boulevard
New Orleans, Louisiana

1. This letter report contains the results of additional geotechnical analyses for proposed I–walls and T–walls at the subject site. The analyses were based on data developed by Eustis Engineering Company, Inc. (Eustis Engineering) and the U.S. Army Corps of Engineers (USACE). The data and other design criteria are contained in Design Memorandum No. 19A, "General Design, London Avenue Outfall Canal, Orleans Parish." Cross–sections and other information necessary for performance of the analyses were furnished to us in a preliminary set of drawings prepared by Burk–Kleinpeter, Inc.

I-Walls

2. Soil Reaches. The project was divided into two (2) soil reaches for purposes of the analyses. Reach limits and stratigraphy were based on information shown on Plates 38 and 60 of the aforementioned Design Memorandum as well as discussions with the representatives of the USACE.

3. Buried beach ridge deposits underlie artificial levee fill, swamp/marsh deposits, and deltaic plain deposits. Floodwall stability is greatly affected by the level of the buried beach surface. Soil parameters, stratification, and the extent of the two soil reaches are shown on Figure 1.

4. We should note that Boring 27 was interpreted to be several feet higher than indicated on Plate 38. Plate 38 indicates Boring 27 was drilled at the levee toe. Near surface soils are levee fill materials and this boring was apparently drilled at the levee centerline. We understand the USACE has also made this interpretation.

5. Design Cross-Sections. Due to variations in the finished levee grade on the protected and flood sides of the new I-wall, the project was also divided into seven (7) sub-reaches based on the cross-section. The furnished minimum design configuration of the ground surface on the protected side includes an 8-ft crown width and a side slope no steeper than 1 vertical on 3 horizontal. Furnished cross-sections indicate that placement of fill should not be required to obtain the minimum design configuration. On the flood side, a composite ground surface configuration was developed from furnished cross-sections.

6. Variations in the finished levee grade and offset distances for the seven reaches are tabulated on Figure 2. A typical cross-section used for the analyses is shown graphically on Figure 3.

7. Design Criteria. The still water level (SWL) is el 11.9. For cantilever I–walls, the freeboard is 2 feet (top of sheets at el 13.9). The furnished tailwater level is el 0.0. For "Q"–case analyses, a factor of safety of 1.5 is applied to the soil shear strength with water to SWL, and a factor of safety of 1.0 is applied with water to SWL plus a 2–ft freeboard. If the resulting penetration to head ratio is less than 3, the penetration is increased to satisfy the ratio of 3 or to that required using the "S"–case with a factor of safety of 1.5, whichever results in the least penetration. The SWL is used for the "S"–case analysis and to compute head for penetration to head ratio. Finally, the penetration is checked for creep distance using Lane's Weighted Creep Ratio (LWCR).

8. Sheetpile Analysis. The required sheetpile penetration below the levee crown was determined by computer analysis using the Method of Planes. Using the factored shear strengths, net lateral soil and water pressure diagrams were determined for movement toward each side of the sheetpile. The summations of horizontal forces and moments about the bottom of the sheetpiles were equated to zero for various tip penetrations. The results of these analyses along with the seepage and ratio computations are tabulated on Figure 2.

9. In every case, the ratio of penetration to head governs the design sheetpile tip elevation. The net pressure diagram to be used for determination of the maximum bending moment and deflection is shown on Figure 3. This diagram corresponds to the "Q"–case with SWL plus a 2–ft freeboard and a factor of safety of 1.0. This loading case at Station 115+00 to Station 119+16.17 on the east bank results in worst case loading conditions. The computer printouts for the maximum loading in each reach is included in the Appendix of this report.

10. Slope Stability. It is understood that slope stability analyses of the west bank previously performed by the USACE remain valid; therefore, additional slope stability analyses of the west bank are not required. Slope stability analyses of the east bank were performed by computer analysis utilizing the LMVD Method of Planes. The results of the

analyses using the critical cross-section for each of the two soil reaches are shown on Figure 4, along with the location of the critical active and passive wedges.

11. Where Reach I soil conditions exist, the minimum factor of safety is 1.33 without considering shear resistance of the sheetpiles. Where Reach 2 soil conditions are assumed, the minimum factor of safety is 1.24 without resistance from the sheetpiles and 2.92 with sheetpile resistance included. Considering that the existing sheetpiles extend to el −28 (12 feet below the potential slip plane failure elevation and 10 feet below the computed design tip elevation), we believe there is an adequate factor of safety against a potential slope stability failure throughout soil Reach 2.

T-Wall

12. In accordance with your instructions, we are including information regarding construction and design of a T-wall between Stations 115+00 and 126+65.00 on the east bank.

13. Cofferdam. Construction of a T-wall will require installation of a cantilever sheetpile cofferdam in order to maintain the current flood protection to el 10.5. In order to provide a factor of safety of at least 1.3 against a potential slope stability failure, the sheetpile cofferdam must not be located closer than 15 feet from the centerline of the proposed T-wall. A distance greater than 15 feet may be necessary to avoid interference between the cofferdam and batter piles supporting the T-wall depending on the degree of batter. The cofferdam sheetpiles should penetrate to el −38.5 to satisfy the "Q"-case analysis for a factor of safety of 1.5. Results of the slope stability analysis, critical active and passive wedges, and the net lateral pressure diagram are shown on Figure 5.

14. Sheetpile Cutoff. Based on slope stability analyses, the sheetpile cutoff below the T-wall will be subjected to a lateral load of 7071 plf. Assuming a triangular distribution of the load and a hinge at el 0.0, the force imposed on the T-wall from the sheetpile cutoff

is 2558 plf. Because the resistance developed by the sheetpile cutoff depends on its embedment into the sand stratum, Eustis Engineering recommends a minimum sheetpile penetration to el −28. The design cross-section along with the computations that determine the lateral forces on the T-wall is shown on Figure 6.

15. Allowable Pile Load Capacities. Ultimate compression and tension pile capacity versus tip elevation curves for vertical 14-in. square precast concrete and 12x53 steel H piles is shown on Figure 7. The axial capacity and horizontal component of batter piles can be determined from geometry in accordance with Figure 8. Computations were made for "Q"-case and "S"-case shear strengths, and the "Q"-case governs. Support from the clay stratum above the potential slope stability slip plane was disregarded. For planning purposes, a factor of safety of 2 may be applied to the values on Figure 7, assuming a pile load test will be performed to verify the design load. If a pile load test will not be performed, a factor of safety of 3 must be used to determine the estimated design load.

16. Group Efficiency. Since all piles will derive their supporting capacity through skin friction, consideration must be given to the effect of group action when piles are used in groups or rows. The capacity of a group or row of piles should be evaluated on the basis of group perimeter shear by the formula shown on Figure 9. The center to center spacing between piles within a group or row should be determined by the formula shown on Figure 9, but should not be less than three times the side dimension of the pile.

17. Subgrade Modulus. The modulus of horizontal subgrade reaction versus depth is plotted on Figure 10. These data were developed based on methods and criteria currently used by USACE.

18. Settlement. All piles supporting the T-wall should be embedded to the same tip elevation. Assuming the use of two or three-pile groups with a spacing of at least 7 feet between groups, settlement of the T-wall should be small and should not exceed 0.25 to

0.5 of an inch due to consolidation of the subsoils. This estimate does not include settlement due to elastic compression of the pile.

Other Considerations

19. Test Piles and Pile Load Tests. At least three test piles of the type selected for construction should be installed to develop more definitive information regarding anticipated driving resistance, requirements for jetting, effects of vibrations, and to verify the estimated pile load capacities. All test piles should be installed using the same equipment and techniques that will be used to drive the job piles. After all test piles have been installed, two piles should be selected for performance of a load test to failure in accordance with ASTM D 1143 for compression and for tension. The pile showing the least resistance to penetration should be selected for load testing. At least one pile should be loaded in compression and one pile loaded in tension. The loading procedure should not begin earlier than 21 days after all reaction piles are installed.

20. Pile Driving. Piles may be installed using a single acting air hammer delivering 24,000 ft-lbs of energy per blow. We recommend that the weight of ram be one-half to two-thirds of the weight of the pile driven and the ram drop be limited to no more than 3 feet. After a hammer selection is made, a driveability study can be made to evaluate its efficiency as well as tentative driving criteria and potential for damage to the pile. All piles should be driven to the embedment shown on Figure 7 unless modified by the test pile program.

21. Jetting. Jetting will be required in order to minimize vibrations to adjacent structures when precast concrete piles penetrate through the sand strata. Jetting may be accomplished through PVC tubes cast into the pile using water pumped from the adjacent canal. The water pressure should be varied to prevent the blow count from falling below 8 to 12 blows per foot while driving through the sand stratum. This criteria is important to minimize the possibility of damage to concrete piles due to development of tension waves.

Jetting operations should be performed under the supervision of an experienced individual knowledgeable in jetting/pile installation techniques. Jetting should not be permitted for installation of steel H piles.

22. Vibrations. Pile driving operations will cause vibrations which may affect nearby structures, roadways, residences, and underground utilities. All adjacent facilities should be carefully inspected by a registered structural engineer prior to pile driving operations to evaluate the potential effects of vibrations. This inspection should include photographs and videotapes of all existing damage to these facilities. Vibrations transmitted to adjacent facilities should be monitored using a seismograph to record their magnitude. A peak particle velocity of 0.25 of an inch per second as measured by the seismograph is generally regarded as a vibration level uncomfortable to human perception. A peak particle velocity of 0.5 of an inch per second or greater measured at a structure may induce vibratory damage to the structure. Additionally, a peak particle velocity of 0.25 of an inch per second may densify near surface cohesionless soils. Such densification would result in potential settlement of surface founded structures or structures supported on piles driven into the sand. Therefore, if sustained peak particle velocity levels in excess of 0.25 of an inch per second are measured at adjacent structures of concern, pile driving operations should be terminated and pile installation procedures revised.

23. Excavations. Excavations required to degrade the levee crown to the finished grade should begin at the highest elevation and proceed down toward the toe of the slope. Spoil material should not be stockpiled and instead should be immediately removed from the site.

Additional Geotechnical Services

24. In order to provide continuity between the investigation, design and construction phases, Eustis Engineering may be retained to provide additional geotechnical services which may include consultation during design and construction, vibration

measurements, logging of test piles and job piles, concrete testing and inspection, and any other soil and material testing services which may provide quality control during construction and conformance to design specifications.

25. If any construction problems arise, Eustis Engineering should be notified so that appropriate action can be undertaken. Eustis Engineering should be retained to monitor the geotechnical related work performed by the contractor. This permits the geotechnical engineer to be on hand and to evaluate unanticipated conditions, to conduct additional testing if required, and to recommend alternative solutions to problems when necessary. This is recommended to avoid major construction cost overruns or disputes on the project.

Yours very truly,

EUSTIS ENGINEERING COMPANY, INC.

LOUIS J. NAPOLITANO

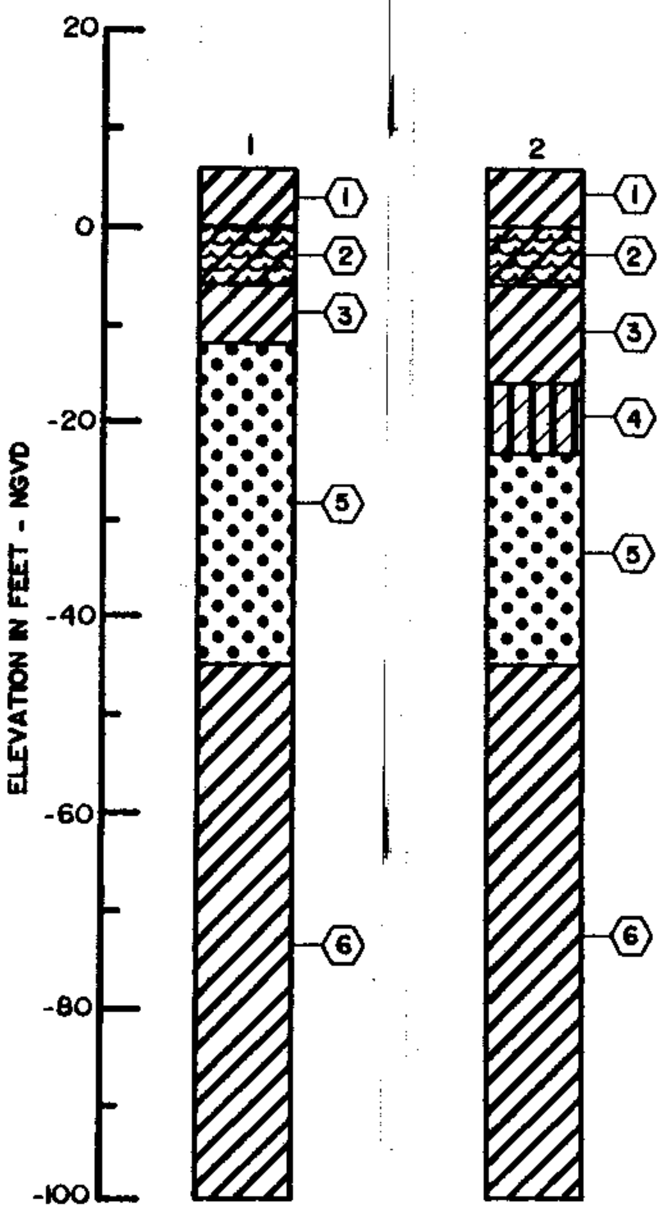
STATE OF LOUISIANA
LOUIS J. NAPOLITANO
REG. NO. 08493
REGISTERED PROFESSIONAL ENGINEER IN CIVIL ENGINEERING

LJN:bh

Figures 1 through 10 and Appendix

EE 12423

SOIL REACHES



SOIL PARAMETERS

ST. NO.	SAT. WEIGHT PCF	"Q"		"S"
		C-PSF	φ-DEG	φ-DEG.
1	109	700	0	23
2	96	400	0	23
3	102	320	0	23
4	117	200	15	30
5	122	0	30	30
6* T	108	830	0	23
B	115	1380	0	23

T = TOP OF STRATUM; B = BOTTOM OF STRATUM

* 108 ABOVE -70; 115 BELOW -70

REACH 1: STA. 70+00 TO STA. 85+50
STA. 95+00 TO STA. 115+00

REACH 2: STA. 85+50 TO STA. 95+00
STA. 115+00 TO STA. 127+00

TOP OF SHEETS: EL. 13.9

STILL WATER LEVEL: EL. 11.9

TAIL WATER LEVEL: EL. 0.0

SOIL REACHES & PARAMETERS

LONDON AVENUE OUTFALL CANAL
PROPOSED I-WALLS AND T-WALLS
NEW ORLEANS, LOUISIANA

EUSTIS ENGINEERING CO., INC.

FIGURE 1

LONDON AVENUE OUTFALL CANAL
PROPOSED I-WALLS AND T-WALLS
MIRABEAU AVENUE TO LEON C. SIMON BOULEVARD
NEW ORLEANS, LOUISIANA

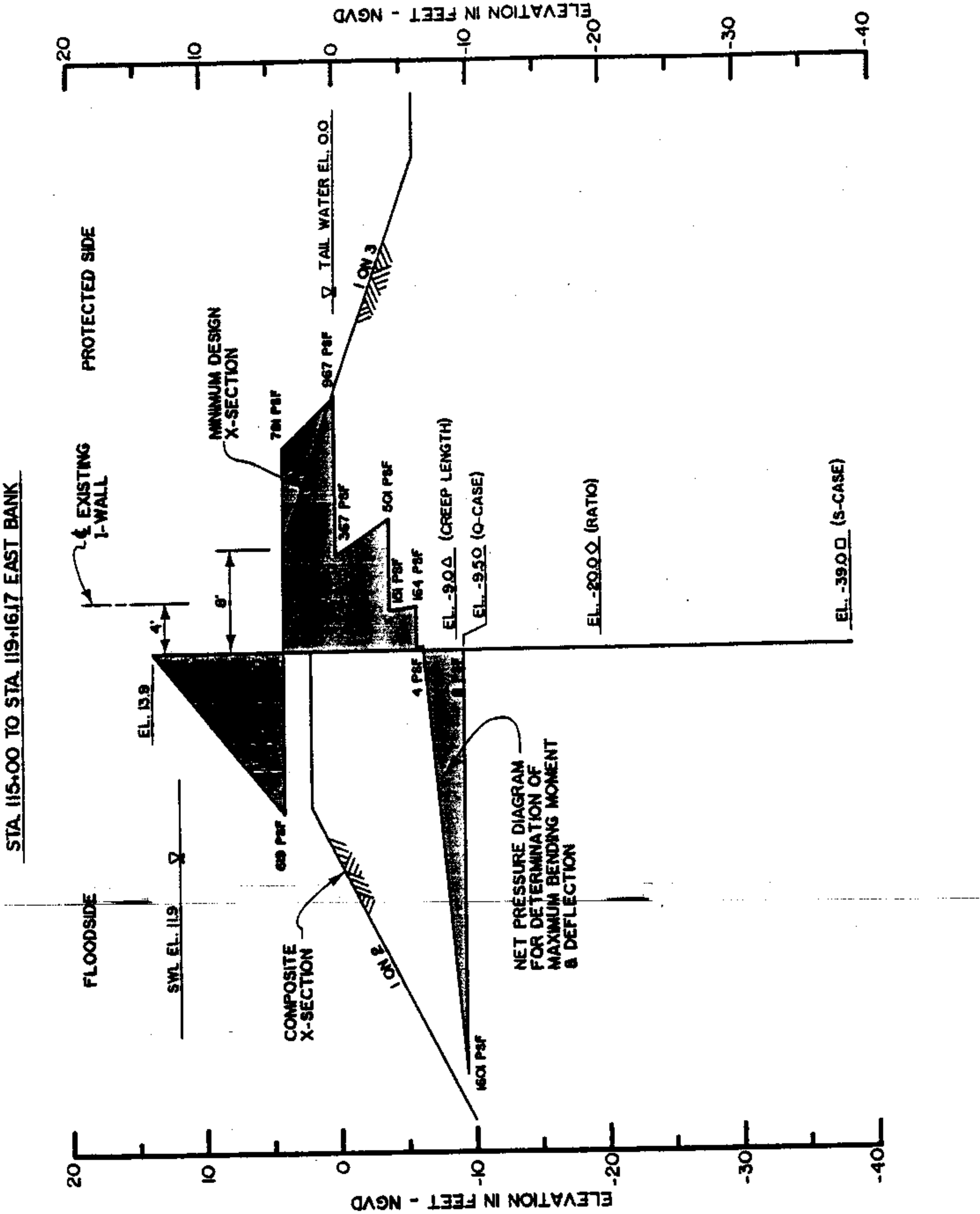
SUMMARY OF I-WALL ANALYSES

BANK	STATIONS	FINISHED GRADE⁺		OFFSET IN FEET	SOIL REACH	TIP ELEVATION IN FEET, NGVD			
		P.S.	F.S.			○	□	◇	△
East	70+24.93 to 84+36.60	5.0	3.0	+4	1	-4.0	-28.0	-16.0	-6.5
	85+54.63 to 95+00	4.5	2.5	Existing Sheetpile	2	-6.2	-36.0	-18.0	-8.0
	95+00 to 99+18.63				1		-32.0		
	102+62.50 to 115+00	4.0	2.0	-4	1	-9.5	-35.0	-20.0	-9.0
	115+00 to 119+16.17				2		-39.0		
	120+40.00 to 126+65.00	5.5	5.5	+4	2	-2.7	-36.0	-14.0	-4.5
West	70+64.00 to 84+90.00	5.0	3.0	+1	1	-4.0	-28.0	-16.0	-6.5
	86+10.00 to 95+00	5.5	3.5	+1	2	-2.6	-36.0	-14.0	-5.5
	95+00 to 101+12.19				1		-26.0		
	101+12.19 to 115+00	5.0	3.0	+1	1	-4.0	-28.0	-16.0	-6.5
	115+00 to 119+65.00				2		-38.0		

⁺ Levee Crown. (PS) = Protected Side; (FS) = Flood Side.
* From existing floodwall. (+) = toward protected side; (–) = toward flood side.
○ Q-Case: Factor of Safety = 1.5 with SWL or Factor of Safety = 1.0 with SWL + 2
□ S-Case: Factor of Safety = 1.5 with SWL
◇ Ratio of Penetration to Head = 3 (Controls design elevation in all cases)
△ Creep Length using Lane's Weighted Creep Ratio

EUSTIS ENGINEERING COMPANY, INC.

FIGURE 2



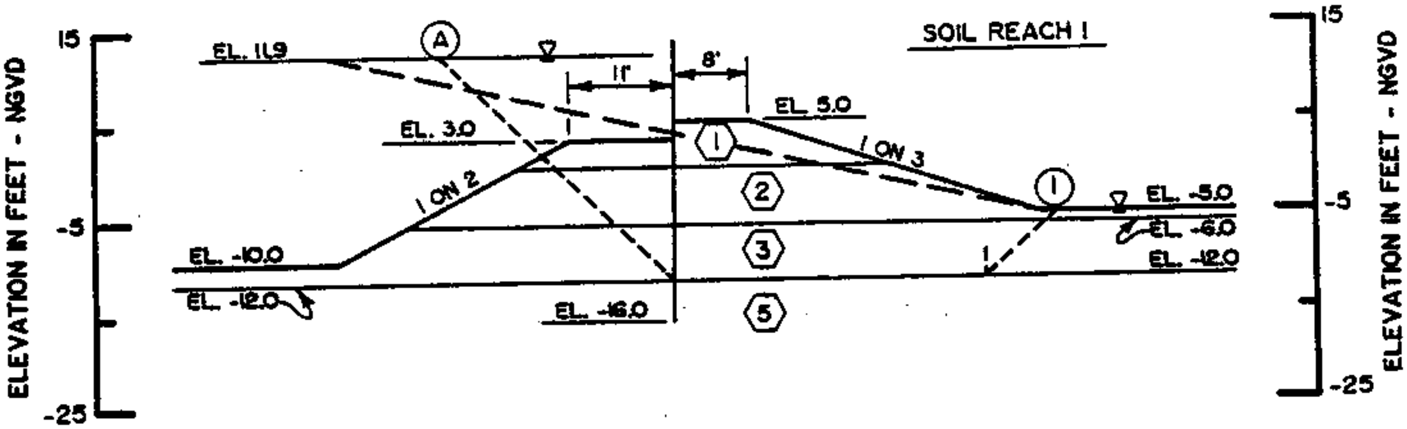
TYPICAL X-SECTION

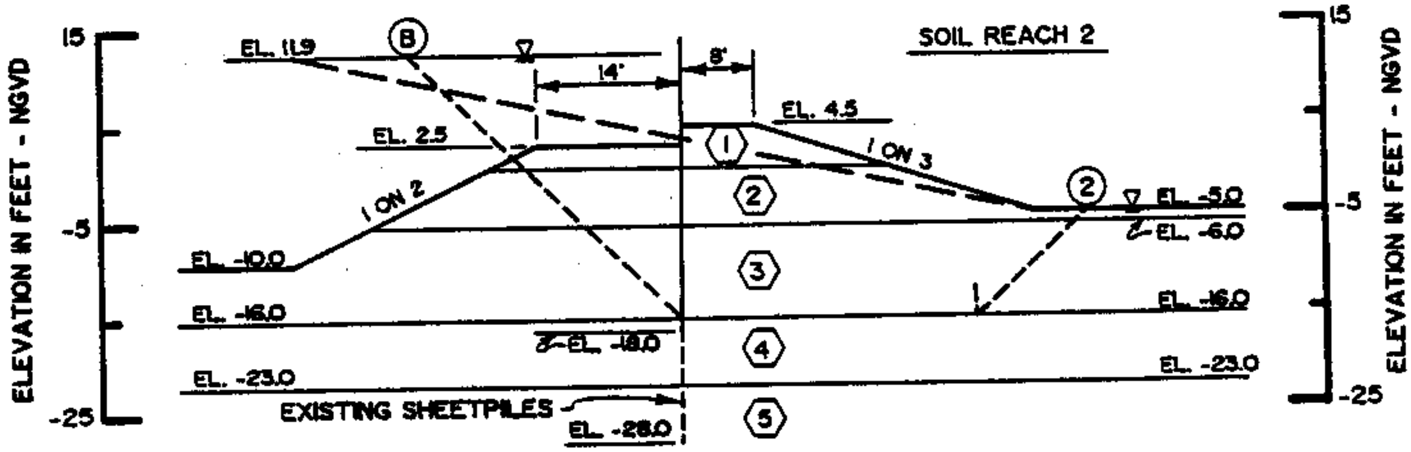
LONDON AVENUE OUTFALL CANAL
PROPOSED I-WALLS AND T-WALLS
NEW ORLEANS, LOUISIANA

FIGURE 3

EUSTIS ENGINEERING CO., INC.

EAST BANK





NO.	EL.	DRIVING FORCE			RESISTING FORCE				RESISTING SHEAR FORCE IN SHEETPILE *	FACTOR OF SAFETY	
		D_A	D_P	ΣD	R_A	R_B	R_P	ΣR		W/O SHEETPILE	W/ SHEETPILE
A-1	-12	22112	2860	19252	10975	19952	4640	25567	———	1.33	———
B-2	-16	30783	6648	24135	12818	9920	7200	29938	40500	1.24	2.92

SCALE: 1"=20'

* BASED ON PZ27

SLOPE STABILITY ANALYSES

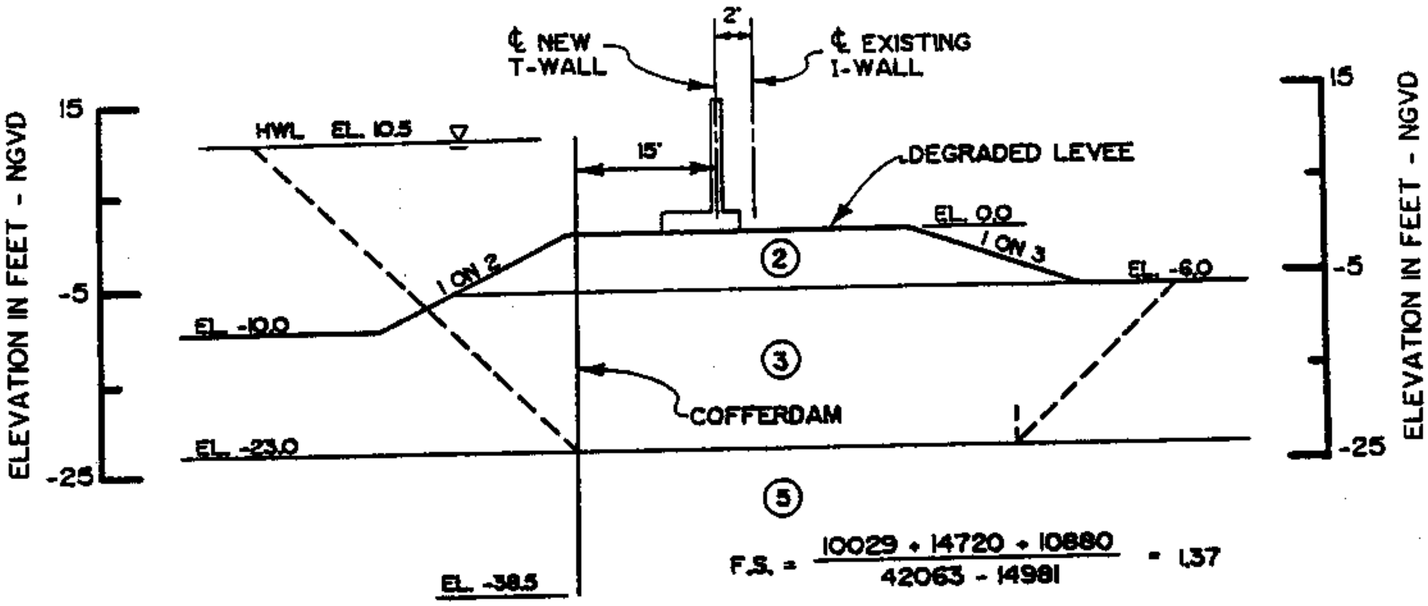
LONDON AVENUE OUTFALL CANAL

PROPOSED I-WALLS AND T-WALLS

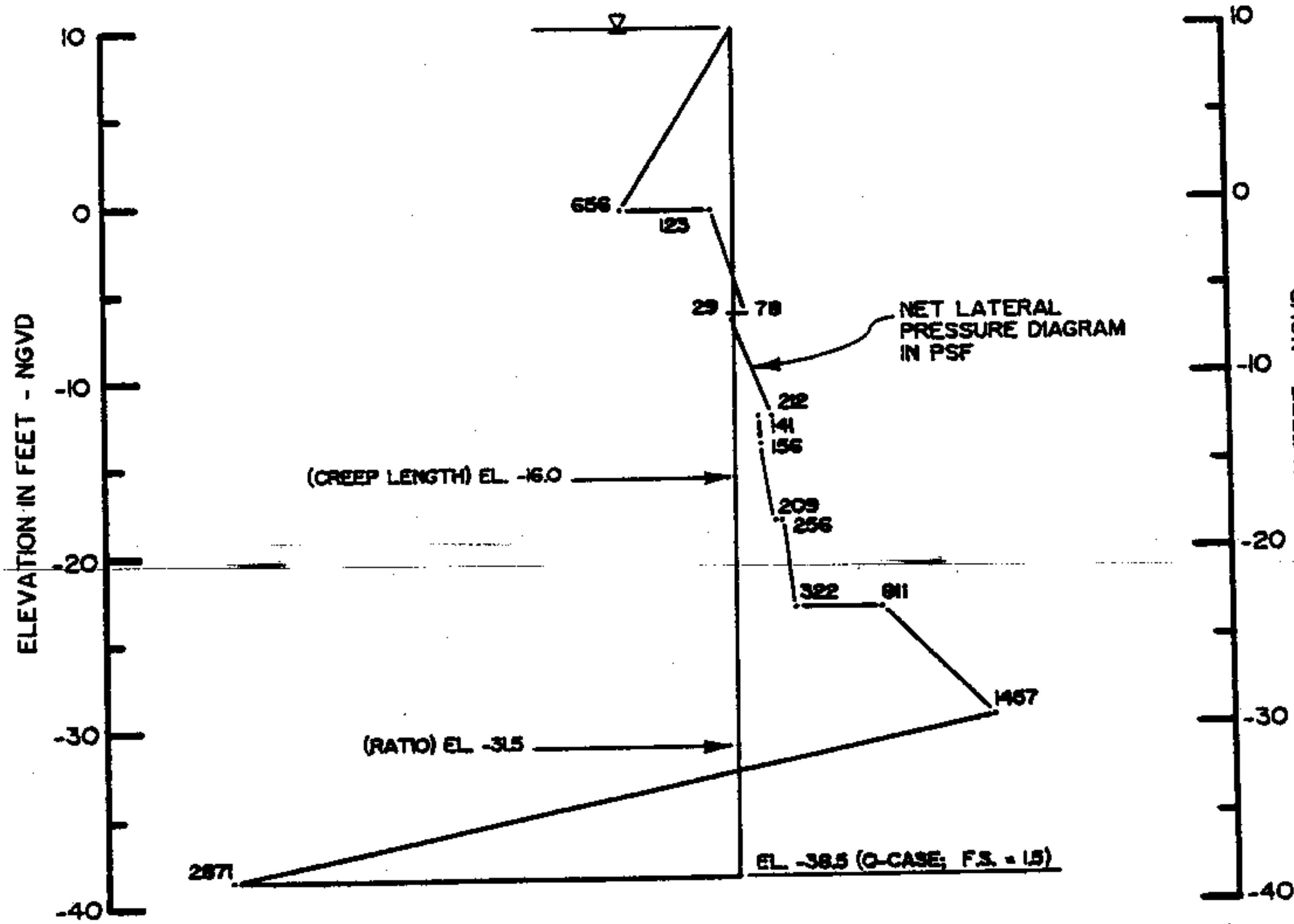
NEW ORLEANS, LOUISIANA

FIGURE 4

EUSTIS ENGINEERING CO., INC.



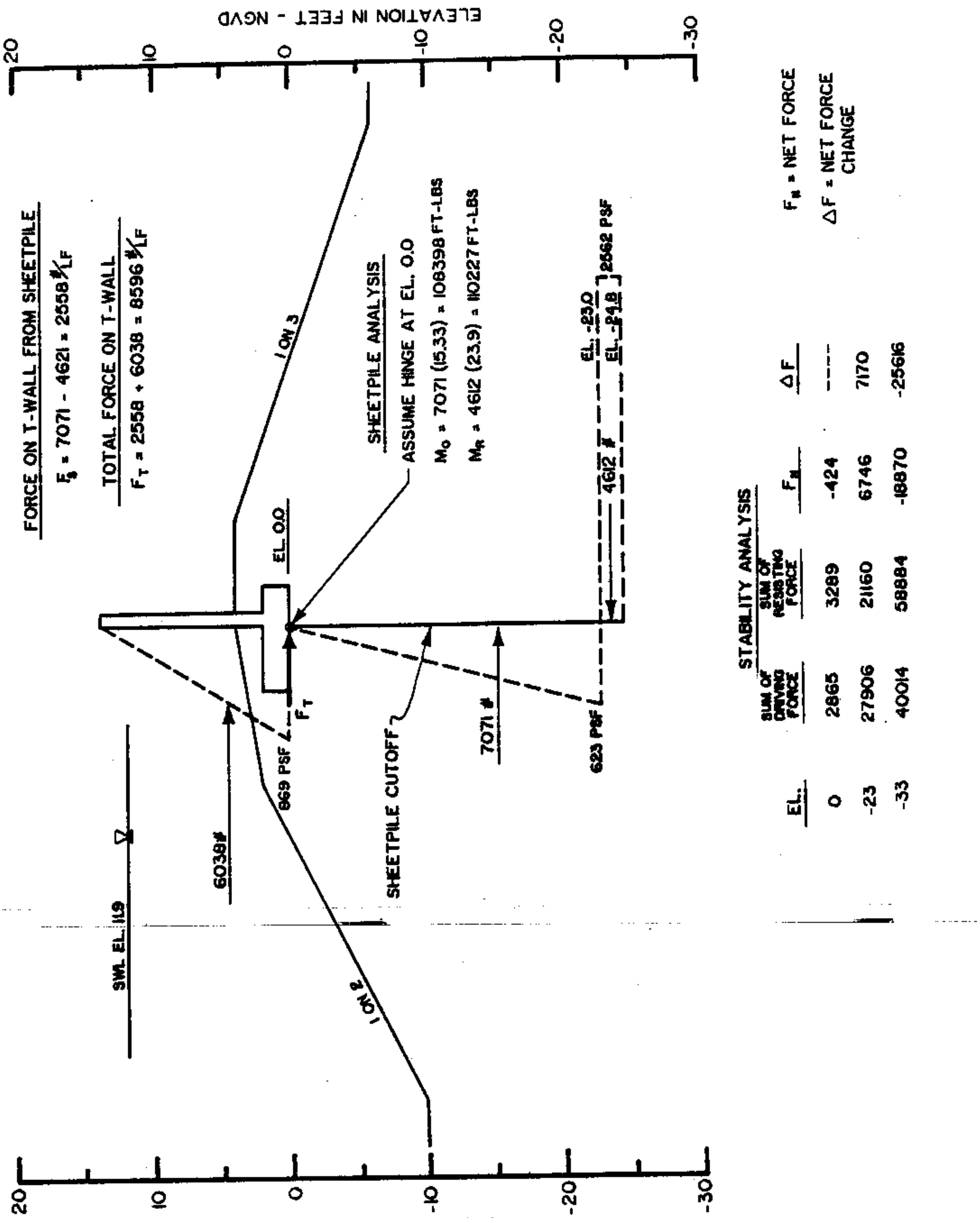
$$F.S. = \frac{10029 + 14720 + 10880}{42063 - 14981} = 1.37$$



EAST BANK
STA. 115+00 TO STA. 126+65.00
COFFERDAM ANALYSIS

LONDON AVENUE OUTFALL CANAL
PROPOSED I-WALLS AND T-WALLS
NEW ORLEANS, LOUISIANA

FIGURE 5



STABILITY ANALYSIS

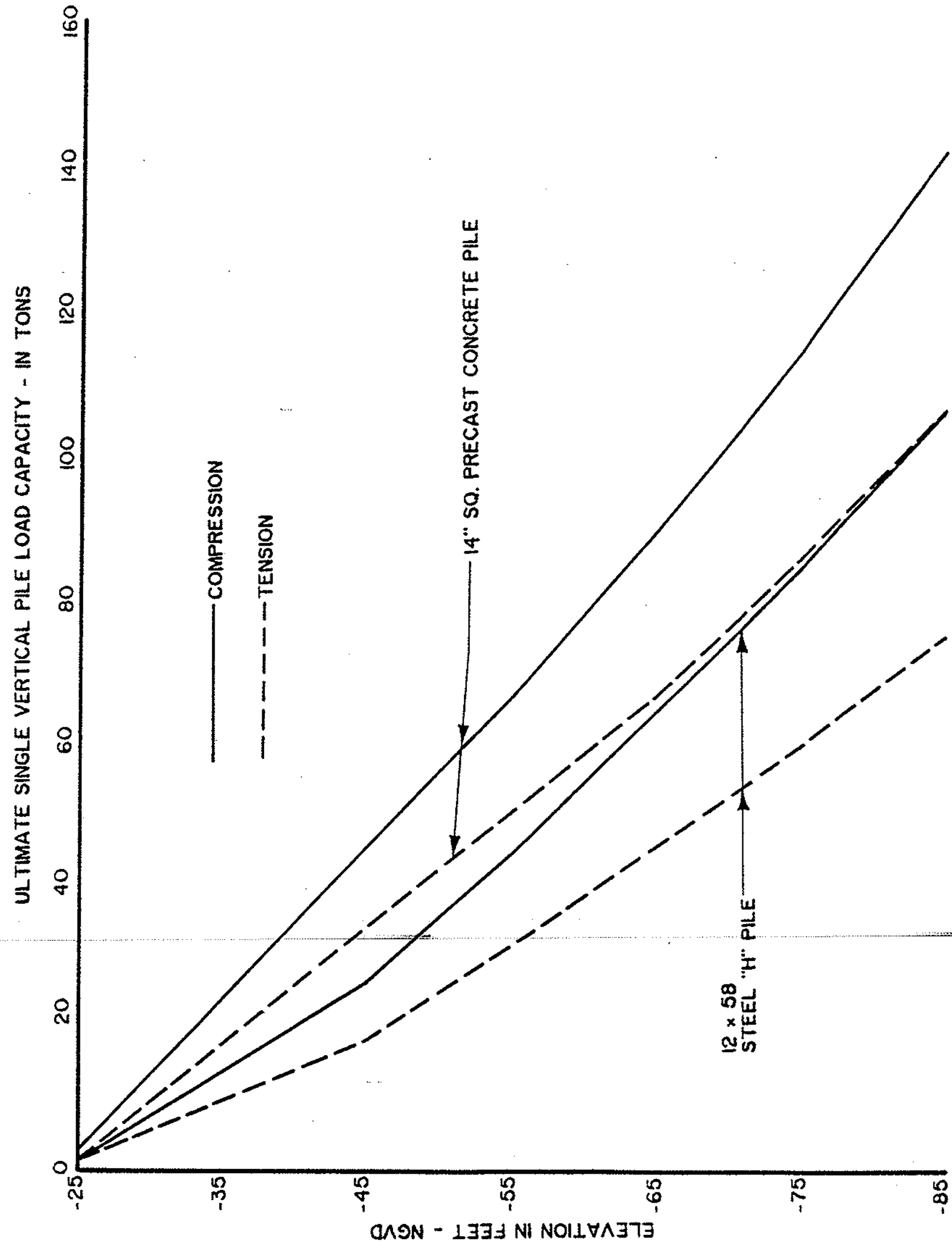
EL.	SUM OF DRIVING FORCE	SUM OF RESISTING FORCE	F_N	ΔF
0	2865	3289	-424	----
-23	27906	21160	6746	7170
-33	40014	58884	-18870	-25616

EAST BANK
STA. 115+00 TO STA. 126+65.00
T-WALL ANALYSIS

LONDON AVENUE OUTFALL CANAL
PROPOSED I-WALLS AND T-WALLS
NEW ORLEANS, LOUISIANA

FIGURE 6

EUSTIS ENGINEERING CO., INC.



NOTE: ① NO SUPPORT IN CLAY ABOVE SLIP PLANE

② "S"-CASE CHECKED AND DOES NOT GOVERN

EAST BANK
STA. 115+00 TO STA. 126+65.00
ULTIMATE PILE LOAD CAPACITIES

LONDON AVENUE OUTFALL CANAL
PROPOSED I-WALLS AND T-WALLS
NEW ORLEANS, LOUISIANA

EUSTIS ENGINEERING

FIGURE 7

AXIAL AND HORIZONTAL RESISTANCE OF BATTER PILES ESTIMATED FROM ALLOWABLE VERTICAL LOAD CAPACITY

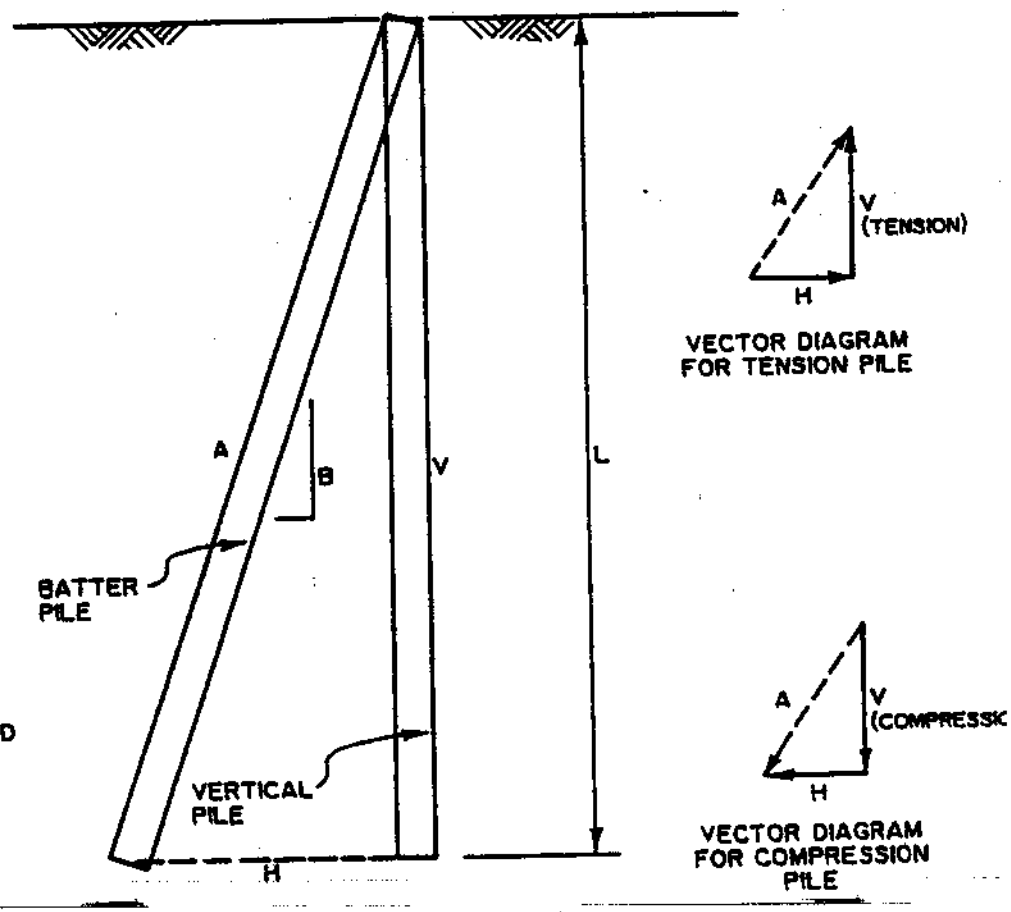
L = VERTICAL COMPONENT OF BATTER PILE EMBEDMENT LENGTH.

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

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

H = HORIZONTAL RESISTANCE OF BATTER PILE ESTIMATED AS FOLLOWS:

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



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

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

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

CAPACITY OF PILE GROUPS

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

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

In Which:

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

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

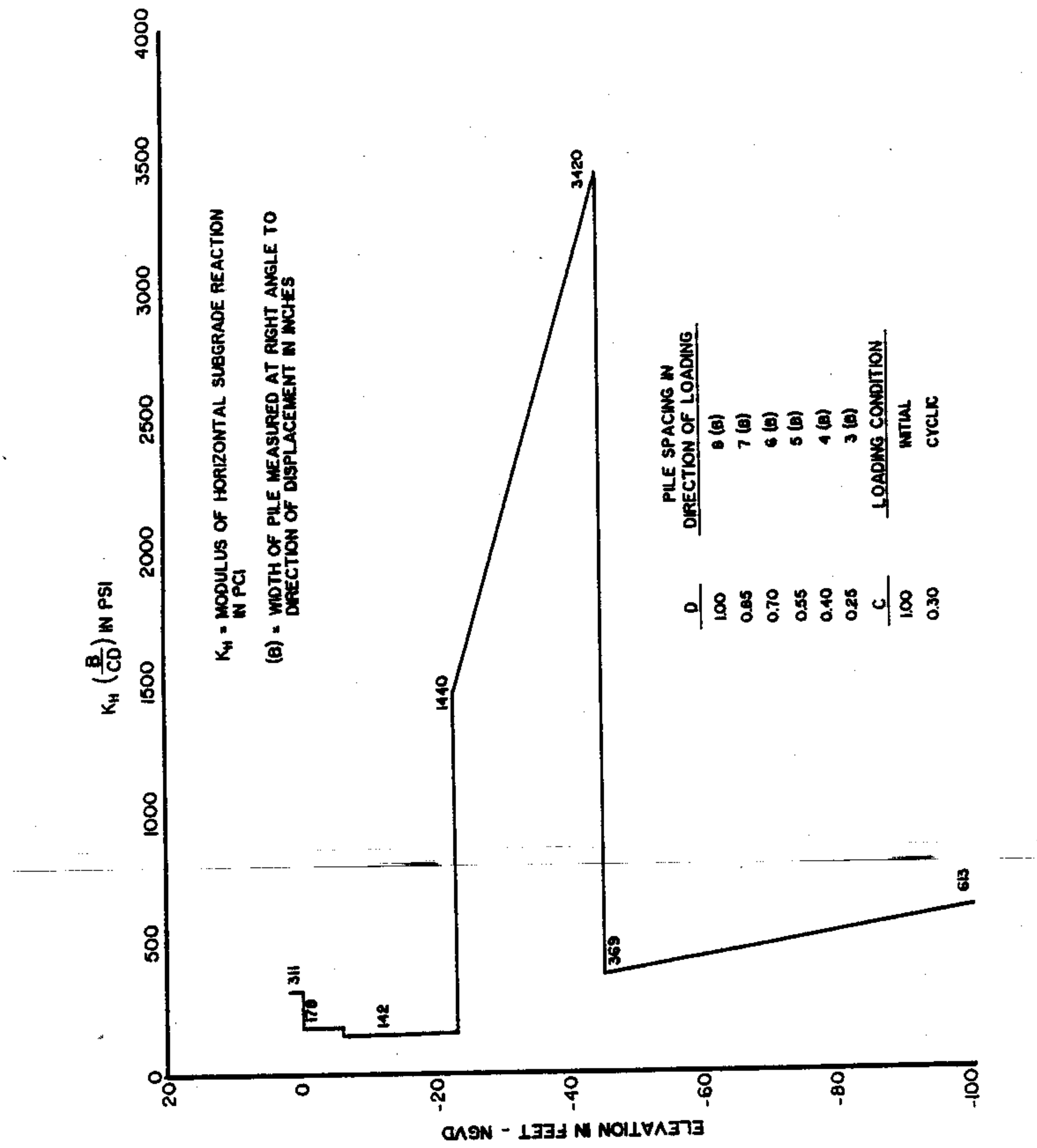
SPACING OF PILE GROUPS

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

In Which:

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

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



MODULUS OF HORIZONTAL
SUBGRADE REACTION

LONDON AVENUE OUTFALL CANAL
PROPOSED I-WALLS AND T-WALLS
NEW ORLEANS, LOUISIANA

APPENDIX

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

DATE: 19-MAY-1993 TIME: 9.44.13

INPUT DATA

I.--HEADING:
'LONDON TRIAL 2

II.--CONTROL
CANTILEVER WALL DESIGN

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

III.--WALL DATA
ELEVATION AT TOP OF WALL = 13.90 (FT)

IV.--SURFACE POINT DATA

STA 70+24.93 TO STA 84+36.60 EB
STA 70+64.00 TO STA 84+70.00 WB
STA 101+12.19 TO STA 115+00 WB
STA 115+00 TO STA 119+65.00 WB

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	3.00
11.00	3.00
37.00	-10.00

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	5.00
8.00	5.00
38.00	-5.00

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.
109.00	109.00	.00	700.0	.00	.0	.00	.00	DEF	DEF
96.00	96.00	.00	400.0	.00	.0	-6.00	.00	DEF	DEF

102.00	102.00	.00	320.0	.00	.0	-12.00	.00	DEF	DEF
122.00	122.00	30.00	.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.
109.00	109.00	.00	700.0	.00	.0	.00	.00	DEF	DEF
96.00	96.00	.00	400.0	.00	.0	-6.00	.00	DEF	DEF
102.00	102.00	.00	320.0	.00	.0	-12.00	.00	DEF	DEF
122.00	122.00	30.00	.0	.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 13.90 (FT)
LEFTSIDE ELEVATION = .00 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

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

DATE: 19-MAY-1993 TIME: 9.44.58

àäääääääääääääääääääääääääääää£
¤ SUMMARY OF RESULTS FOR ¤
¤ CANTILEVER WALL DESIGN ¤
àäääääääääääääääääääääääääääää¥

I.--HEADING

'LONDON TRIAL 2

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

```
WALL BOTTOM ELEV. (FT)     :        -4.06
     PENETRATION (FT)      :         9.06

MAX. BEND. MOMENT (LB-FT) :        10799.
     AT ELEVATION (FT)     :         2.27

MAX. SCALED DEFL. (LB-IN3):    1.5256E+09
     AT ELEVATION (FT)     :        13.90
```

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

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

DATE: 19-MAY-1993 TIME: 9.44.58

```
àëëëëëëëëëëëëëëëëëëëëëëëëëëëëë£
¤   COMPLETE RESULTS FOR    ¤
¤  CANTILEVER WALL DESIGN   ¤
àëëëëëëëëëëëëëëëëëëëëëëëëëëëëë¥
```

I.--HEADING

'LONDON TRIAL 2

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
13.90	0.	0.	1.5256E+09	.00
12.90	10.	31.	1.3891E+09	62.50
11.90	83.	125.	1.2526E+09	125.00
10.90	281.	281.	1.1162E+09	187.50
9.90	667.	500.	9.8039E+08	250.00
8.90	1302.	781.	8.4575E+08	312.50
7.90	2250.	1125.	7.1341E+08	375.00
6.90	3573.	1531.	5.8501E+08	437.50
5.90	5333.	2000.	4.6284E+08	500.00
5.00	7343.	2475.	3.6073E+08	556.25
5.00	7343.	2475.	3.6073E+08	-843.75
4.90	7587.	2391.	3.4996E+08	-848.40

3.90	9545.	1519.	2.5014E+08	-894.90
3.00	10545.	695.	1.7423E+08	-936.75
2.95	10578.	648.	1.7043E+08	-939.07
2.90	10609.	601.	1.6668E+08	-941.40
2.00	10763.	-265.	1.0711E+08	-983.25
1.90	10732.	-364.	1.0142E+08	-988.32
.90	9868.	-1369.	5.4563E+07	-1021.85
.00	8256.	-2176.	2.6926E+07	-771.41
-.10	8035.	-2251.	2.4614E+07	-722.83
-.28	7617.	-2378.	2.0790E+07	-685.62
-1.10	5502.	-2706.	8.4459E+06	-114.87
-2.10	2855.	-2472.	1.7692E+06	581.61
-3.10	790.	-1542.	1.1010E+05	1278.10
-4.06	1.	0.	0.0000E+00	1944.76

(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
13.90	0.	0.	0.	0.
12.90	0.	0.	0.	0.
11.90	0.	0.	0.	0.
10.90	0.	0.	0.	0.
9.90	0.	0.	0.	0.
8.90	0.	0.	0.	0.
7.90	0.	0.	0.	0.
6.90	0.	0.	0.	0.
5.90	0.	0.	0.	0.
5.00+	0.	0.	0.	0.
5.00-	1400.	0.	0.	0.
4.90	1411.	0.	0.	0.
3.90	1520.	0.	0.	0.
3.00+	1618.	0.	0.	0.
3.00-	1618.	0.	0.	1400.
2.95	1623.	0.	0.	1402.
2.90	1629.	0.	0.	1405.
2.00	1727.	0.	0.	1446.
1.90	1738.	0.	0.	1452.
.90	1834.	0.	0.	1485.
.00	1640.	0.	0.	1240.
-.10	1592.	0.	0.	1189.
-.28	1554.	0.	0.	1150.
-1.10	1385.	0.	0.	973.
-2.10	1373.	0.	0.	1010.
-3.10	1246.	0.	0.	1043.
-4.06	1127.	0.	0.	1077.
-5.10	1101.	2.	0.	1107.

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

DATE: 19-MAY-1993 TIME: 10.10.52

ÈÊÊÊÊÊÊÊÊÊÊÊÊÊÊÊÊ£
¤ INPUT DATA ¤
ÀÊÊÊÊÊÊÊÊÊÊÊÊÊÊÊÊ¥

I.--HEADING:
'LONDON TRIAL 3

II.--CONTROL
CANTILEVER WALL DESIGN

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

III.--WALL DATA
ELEVATION AT TOP OF WALL = 13.90 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	2.50
15.00	2.50
40.00	-10.00

STA. 85+54.63 TO STA 95+00 EB
STA. 95+00 TO STA 99+18.63 EB

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	4.50
8.00	4.50
36.00	-5.00

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)	<--BOTTOM--> SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	<-SAFETY-> <-FACTOR-> PASS.
109.00	109.00	.00	700.0	.00	.0	.00	.00	DEF	DEF

96.00	96.00	.00	400.0	.00	.0	-6.00	.00	DEF	DEF
102.00	102.00	.00	320.0	.00	.0	-16.00	.00	DEF	DEF
117.00	117.00	15.00	200.0	.00	.0	-23.00	.00	DEF	DEF
122.00	122.00	30.00	.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.
109.00	109.00	.00	700.0	.00	.0	.00	.00	DEF	DEF
96.00	96.00	.00	400.0	.00	.0	-6.00	.00	DEF	DEF
102.00	102.00	.00	320.0	.00	.0	-16.00	.00	DEF	DEF
117.00	117.00	15.00	200.0	.00	.0	-23.00	.00	DEF	DEF
122.00	122.00	30.00	.0	.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 13.90 (FT)
LEFTSIDE ELEVATION = .00 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

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

DATE: 19-MAY-1993 TIME: 10.11.28

àëëëëëëëëëëëëëëëëëëëëëëëëëëëëë£
¤ SUMMARY OF RESULTS FOR ¤
¤ CANTILEVER WALL DESIGN ¤
àëëëëëëëëëëëëëëëëëëëëëëëëëëëëë¥

I.--HEADING

'LONDON TRIAL 3

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

```
WALL BOTTOM ELEV. (FT)     :        -6.23
      PENETRATION (FT)     :        10.73

MAX. BEND. MOMENT (LB-FT)  :       13077.
      AT ELEVATION (FT)    :         1.38

MAX. SCALED DEFL. (LB-IN3) :   2.3388E+09
      AT ELEVATION (FT)    :        13.90
```

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

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

DATE: 19-MAY-1993 TIME: 10.11.28

COMPLETE RESULTS FOR
CANTILEVER WALL DESIGN

I.--HEADING

'LONDON TRIAL 3

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
13.90	0.	0.	2.3388E+09	.00
12.90	10.	31.	2.1485E+09	62.50
11.90	83.	125.	1.9582E+09	125.00
10.90	281.	281.	1.7681E+09	187.50
9.90	667.	500.	1.5785E+09	250.00
8.90	1302.	781.	1.3900E+09	312.50
7.90	2250.	1125.	1.2039E+09	375.00
6.90	3573.	1531.	1.0217E+09	437.50
5.90	5333.	2000.	8.4576E+08	500.00

4.90	7594.	2531.	6.7910E+08	562.50
4.50	8652.	2761.	6.1588E+08	587.50
4.50	8652.	2761.	6.1588E+08	-812.50
3.90	10161.	2265.	5.2562E+08	-840.40
2.90	11998.	1402.	3.8959E+08	-886.90
2.50	12487.	1043.	3.4082E+08	-905.50
2.20	12759.	770.	3.0650E+08	-919.45
1.90	12949.	492.	2.7416E+08	-933.40
1.50	13070.	114.	2.3418E+08	-954.17
.90	12966.	-464.	1.8097E+08	-972.60
.00	12189.	-1224.	1.1618E+08	-716.40
-.10	12063.	-1293.	1.1006E+08	-667.65
-1.10	10471.	-1855.	5.9883E+07	-456.59
-2.10	8381.	-2331.	2.7736E+07	-494.80
-2.55	7284.	-2546.	1.8224E+07	-463.51
-3.10	5830.	-2706.	1.0001E+07	-117.28
-4.10	3170.	-2509.	2.3237E+06	511.88
-5.10	1021.	-1683.	1.9717E+05	1141.04
-6.00	46.	-401.	3.3708E+02	1707.28
-6.10	14.	-227.	3.2242E+01	1770.20
-6.23	0.	0.	0.0000E+00	1849.09

(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
13.90	0.	0.	0.	0.
12.90	0.	0.	0.	0.
11.90	0.	0.	0.	0.
10.90	0.	0.	0.	0.
9.90	0.	0.	0.	0.
8.90	0.	0.	0.	0.
7.90	0.	0.	0.	0.
6.90	0.	0.	0.	0.
5.90	0.	0.	0.	0.
4.90	0.	0.	0.	0.
4.50+	0.	0.	0.	0.
4.50-	1400.	0.	0.	0.
3.90	1465.	0.	0.	0.
2.90	1574.	0.	0.	0.
2.50+	1618.	0.	0.	0.
2.50-	1618.	0.	0.	1400.
2.20	1651.	0.	0.	1414.
1.90	1683.	0.	0.	1428.
1.50	1729.	0.	0.	1449.
.90	1785.	0.	0.	1467.
.00	1585.	0.	0.	1216.
-.10	1536.	0.	0.	1165.
-1.10	1325.	0.	0.	950.
-2.10	1364.	0.	0.	987.
-2.55	1332.	0.	0.	1002.
-3.10	1294.	0.	0.	1020.
-4.10	1124.	0.	0.	1054.
-5.10	1050.	1.	0.	1084.

-6.00	989.	29.	0.	1038.
-6.10	975.	36.	0.	1027.
-6.23	920.	95.	0.	1000.
-8.10	933.	134.	0.	1040.

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

DATE: 13-MAY-1993 TIME: 8.55.43

INPUT DATA

I.--HEADING:
'LONDON AVENUE CANAL
'JOB # 12423
'TRIAL 1

II.--CONTROL
CANTILEVER WALL DESIGN

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

III.--WALL DATA
ELEVATION AT TOP OF WALL = 13.90 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	2.00
12.00	2.00
36.00	-10.00

STA 102+62.50 TO STA 115+00 EB
STA 115+00 TO STA 119+16.17 EB

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	4.00
8.00	4.00
40.00	-6.00

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.

109.00	109.00	.00	700.0	.00	.0	.00	.00	DEF	DEF
96.00	96.00	.00	400.0	.00	.0	-6.00	.00	DEF	DEF
102.00	102.00	.00	320.0	.00	.0	-16.00	.00	DEF	DEF
117.00	117.00	15.00	200.0	.00	.0	-23.00	.00	DEF	DEF
122.50	122.50	30.00	.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.
109.00	109.00	.00	700.0	.00	.0	.00	.00	DEF	DEF
96.00	96.00	.00	400.0	.00	.0	-6.00	.00	DEF	DEF
102.00	102.00	.00	320.0	.00	.0	-12.00	.00	DEF	DEF
122.50	122.50	30.00	.0	.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 13.90 (FT)
LEFTSIDE ELEVATION = .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: 13-MAY-1993 TIME: 9.02.04

SUMMARY OF RESULTS FOR
CANTILEVER WALL DESIGN

I.--HEADING

'LONDON AVENUE CANAL

'JOB # 12423
'TRIAL 1

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

WALL BOTTOM ELEV. (FT) : -9.39
PENETRATION (FT) : 13.39

MAX. BEND. MOMENT (LB-FT) : 15713.
AT ELEVATION (FT) : .42

MAX. SCALED DEFL. (LB-IN3): 3.8375E+09
AT ELEVATION (FT) : 13.90

(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: 13-MAY-1993 TIME: 9.02.04

COMPLETE RESULTS FOR
CANTILEVER WALL DESIGN

I.--HEADING

'LONDON AVENUE CANAL
'JOB # 12423
'TRIAL 1

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
13.90	0.	0.	3.8375E+09	.00
12.90	10.	31.	3.5596E+09	62.50
11.90	83.	125.	3.2818E+09	125.00
10.90	281.	281.	3.0042E+09	187.50
9.90	667.	500.	2.7271E+09	250.00

8.90	1302.	781.	2.4511E+09	312.50
7.90	2250.	1125.	2.1775E+09	375.00
6.90	3573.	1531.	1.9078E+09	437.50
5.90	5333.	2000.	1.6443E+09	500.00
4.90	7594.	2531.	1.3901E+09	562.50
4.00	10107.	3063.	1.1725E+09	618.75
4.00	10107.	3063.	1.1725E+09	-781.25
3.90	10410.	2984.	1.1491E+09	-785.90
2.90	12993.	2175.	9.2610E+08	-832.40
2.00	14608.	1407.	7.4436E+08	-874.25
1.95	14678.	1364.	7.3484E+08	-876.57
1.90	14745.	1320.	7.2539E+08	-878.90
1.00	15570.	507.	5.6638E+08	-926.84
.90	15616.	414.	5.5004E+08	-929.16
.00	15649.	-301.	4.1517E+08	-660.98
-.10	15615.	-365.	4.0154E+08	-611.92
-1.10	14980.	-871.	2.7993E+08	-400.98
-2.10	13901.	-1294.	1.8415E+08	-444.00
-3.10	12387.	-1732.	1.1233E+08	-431.35
-4.10	10460.	-2103.	6.1850E+07	-311.43
-5.10	8221.	-2356.	2.9400E+07	-193.81
-6.00	6034.	-2489.	1.2433E+07	-103.17
-6.10	5785.	-2499.	1.1126E+07	-90.35
-6.27	5370.	-2513.	9.1813E+06	-81.53
-7.10	3300.	-2384.	2.8388E+06	391.20
-8.10	1206.	-1709.	3.0919E+05	957.66
-9.10	69.	-469.	8.6242E+02	1524.13
-9.39	0.	0.	0.0000E+00	1689.31

(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
13.90	0.	0.	0.	0.
12.90	0.	0.	0.	0.
11.90	0.	0.	0.	0.
10.90	0.	0.	0.	0.
9.90	0.	0.	0.	0.
8.90	0.	0.	0.	0.
7.90	0.	0.	0.	0.
6.90	0.	0.	0.	0.
5.90	0.	0.	0.	0.
4.90	0.	0.	0.	0.
4.00+	0.	0.	0.	0.
4.00-	1400.	0.	0.	0.
3.90	1411.	0.	0.	0.
2.90	1520.	0.	0.	0.
2.00+	1618.	0.	0.	0.
2.00-	1618.	0.	0.	1400.
1.95	1623.	0.	0.	1402.
1.90	1629.	0.	0.	1405.
1.00	1733.	0.	0.	1453.
.90	1742.	0.	0.	1455.
.00	1530.	0.	0.	1192.

-.10	1481.	0.	0.	1140.
-1.10	1270.	0.	0.	927.
-2.10	1313.	0.	0.	963.
-3.10	1300.	0.	0.	997.
-4.10	1180.	0.	0.	1031.
-5.10	1063.	0.	0.	1060.
-6.00	972.	1.	0.	1015.
-6.10	959.	3.	0.	1004.
-6.27	950.	9.	0.	999.
-7.10	906.	40.	0.	977.
-8.10	920.	80.	0.	1026.
-9.10	934.	119.	0.	1001.
-9.39	945.	159.	0.	831.
-11.10	976.	200.	0.	681.

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

DATE: 19-MAY-1993 TIME: 10.34.54

INPUT DATA

I.--HEADING:
'LONDON TRIAL 4

II.--CONTROL
CANTILEVER WALL DESIGN

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

III.--WALL DATA
ELEVATION AT TOP OF WALL = 13.90 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	5.50
8.00	5.50
34.00	-3.00

STA 120+40.00 TO STA 126+65.00 EB

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	5.50
8.00	5.50
34.00	-3.00

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.
109.00	109.00	.00	700.0	.00	.0	.00	.00	DEF	DEF
96.00	96.00	.00	400.0	.00	.0	-6.00	.00	DEF	DEF

102.00	102.00	.00	320.0	.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.
109.00	109.00	.00	700.0	.00	.0	.00	.00	DEF	DEF
96.00	96.00	.00	400.0	.00	.0	-6.00	.00	DEF	DEF
102.00	102.00	.00	320.0	.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 13.90 (FT)
LEFTSIDE ELEVATION = .00 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

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

DATE: 19-MAY-1993 TIME: 10.35.21

SUMMARY OF RESULTS FOR
CANTILEVER WALL DESIGN

I.--HEADING

'LONDON TRIAL 4

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

```
WALL BOTTOM ELEV. (FT)    :        -2.65
     PENETRATION (FT)     :         8.15

MAX. BEND. MOMENT (LB-FT) :        8839.
     AT ELEVATION (FT)    :         3.13

MAX. SCALED DEFL. (LB-IN3):   1.0463E+09
     AT ELEVATION (FT)    :        13.90
```

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

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

DATE: 19-MAY-1993 TIME: 10.35.21

COMPLETE RESULTS FOR CANTILEVER WALL DESIGN

I.--HEADING

'LONDON TRIAL 4

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
13.90	0.	0.	1.0463E+09	.00
12.90	10.	31.	9.4499E+08	62.50
11.90	83.	125.	8.4371E+08	125.00
10.90	281.	281.	7.4260E+08	187.50
9.90	667.	500.	6.4200E+08	250.00
8.90	1302.	781.	5.4258E+08	312.50
7.90	2250.	1125.	4.4547E+08	375.00
6.90	3573.	1531.	3.5229E+08	437.50
5.90	5333.	2000.	2.6535E+08	500.00
5.50	6174.	2205.	2.3298E+08	525.00
5.50	6174.	2205.	2.3298E+08	-875.00
5.20	6796.	1940.	2.0980E+08	-888.95
4.90	7338.	1672.	1.8769E+08	-902.90
4.50	7934.	1307.	1.5998E+08	-921.50

3.90	8550.	745.	1.2257E+08	-949.40
2.90	8813.	-227.	7.2101E+07	-995.90
1.90	8080.	-1247.	3.6713E+07	-1044.73
1.57	7606.	-1597.	2.8105E+07	-1055.27
.90	6340.	-2150.	1.5139E+07	-601.10
.00	4245.	-2415.	5.1248E+06	11.88
-.10	4004.	-2410.	4.4339E+06	79.99
-1.10	1747.	-1990.	6.5839E+05	761.07
-2.10	251.	-888.	1.1073E+04	1442.15
-2.65	0.	0.	0.0000E+00	1813.67

(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	< LEFTSIDE PRESSURE (PSF)>		<RIGHTSIDE PRESSURE (PSF)>	
(FT)	PASSIVE	ACTIVE	ACTIVE	PASSIVE
13.90	0.	0.	0.	0.
12.90	0.	0.	0.	0.
11.90	0.	0.	0.	0.
10.90	0.	0.	0.	0.
9.90	0.	0.	0.	0.
8.90	0.	0.	0.	0.
7.90	0.	0.	0.	0.
6.90	0.	0.	0.	0.
5.90	0.	0.	0.	0.
5.50+	0.	0.	0.	0.
5.50-	1400.	0.	0.	1400.
5.20	1433.	0.	0.	1414.
4.90	1465.	0.	0.	1428.
4.50	1509.	0.	0.	1447.
3.90	1574.	0.	0.	1474.
2.90	1683.	0.	0.	1521.
1.90	1795.	0.	0.	1570.
1.57	1826.	0.	0.	1580.
.90	1889.	0.	0.	1601.
.00	1695.	0.	0.	1356.
-.10	1646.	0.	0.	1305.
-1.10	1436.	0.	0.	1092.
-2.10	1372.	0.	0.	1030.
-2.65	1208.	0.	0.	874.
-4.10	1140.	0.	0.	821.

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

DATE: 19-MAY-1993 TIME: 10.49.58

INPUT DATA

I.--HEADING:
'LONDON TRIAL 5

II.--CONTROL
CANTILEVER WALL DESIGN

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

III.--WALL DATA
ELEVATION AT TOP OF WALL = 13.90 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	3.50
10.00	3.50
40.00	-5.00

STA 86+10.00 TO STA 95+00 WB

STA 95+00 TO STA 101+12.19 WB

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	5.50
8.00	5.50
31.00	-2.00

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.
109.00	109.00	.00	700.0	.00	.0	.00	.00	DEF	DEF
96.00	96.00	.00	400.0	.00	.0	-6.00	.00	DEF	DEF

102.00	102.00	.00	400.0	.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.
109.00	109.00	.00	700.0	.00	.0	.00	.00	DEF	DEF
96.00	96.00	.00	400.0	.00	.0	-6.00	.00	DEF	DEF
102.00	102.00	.00	400.0	.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 13.90 (FT)
LEFTSIDE ELEVATION = .00 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

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

DATE: 19-MAY-1993 TIME: 10.50.25

àäääääääääääääääääääääääääääää£
¤ SUMMARY OF RESULTS FOR ¤
¤ CANTILEVER WALL DESIGN ¤
àäääääääääääääääääääääääääääää¥

I.--HEADING

'LONDON TRIAL 5

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

```
WALL BOTTOM ELEV. (FT)     :        -2.55
     PENETRATION (FT)      :         8.05

MAX. BEND. MOMENT (LB-FT) :         8839.
     AT ELEVATION (FT)     :         3.13

MAX. SCALED DEFL. (LB-IN3):    1.0401E+09
     AT ELEVATION (FT)     :        13.90
```

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

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

DATE: 19-MAY-1993 TIME: 10.50.25

COMPLETE RESULTS FOR CANTILEVER WALL DESIGN

I.--HEADING

'LONDON TRIAL 5

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
13.90	0.	0.	1.0401E+09	.00
12.90	10.	31.	9.3921E+08	62.50
11.90	83.	125.	8.3835E+08	125.00
10.90	281.	281.	7.3765E+08	187.50
9.90	667.	500.	6.3747E+08	250.00
8.90	1302.	781.	5.3847E+08	312.50
7.90	2250.	1125.	4.4177E+08	375.00
6.90	3573.	1531.	3.4901E+08	437.50
5.90	5333.	2000.	2.6249E+08	500.00
5.50	6174.	2205.	2.3028E+08	525.00
5.50	6174.	2205.	2.3028E+08	-875.00
4.90	7338.	1672.	1.8524E+08	-902.90
3.90	8550.	745.	1.2055E+08	-949.40
3.50	8772.	362.	9.8704E+07	-968.00

3.20	8837.	70.	8.3910E+07	-981.95
2.90	8813.	-227.	7.0489E+07	-995.90
2.50	8642.	-629.	5.4722E+07	-1014.50
1.90	8080.	-1247.	3.5518E+07	-1044.15
1.45	7415.	-1718.	2.4459E+07	-1058.60
.90	6328.	-2189.	1.4360E+07	-648.80
.00	4185.	-2472.	4.6991E+06	20.06
-.10	3938.	-2466.	4.0436E+06	94.38
-1.10	1643.	-2000.	5.4511E+05	837.56
-2.10	185.	-791.	5.5816E+03	1580.75
-2.55	0.	0.	0.0000E+00	1917.01

(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
13.90	0.	0.	0.	0.
12.90	0.	0.	0.	0.
11.90	0.	0.	0.	0.
10.90	0.	0.	0.	0.
9.90	0.	0.	0.	0.
8.90	0.	0.	0.	0.
7.90	0.	0.	0.	0.
6.90	0.	0.	0.	0.
5.90	0.	0.	0.	0.
5.50+	0.	0.	0.	0.
5.50-	1400.	0.	0.	0.
4.90	1465.	0.	0.	0.
3.90	1574.	0.	0.	0.
3.50+	1618.	0.	0.	0.
3.50-	1618.	0.	0.	1400.
3.20	1651.	0.	0.	1414.
2.90	1683.	0.	0.	1428.
2.50	1727.	0.	0.	1447.
1.90	1794.	0.	0.	1476.
1.45	1837.	0.	0.	1491.
.90	1889.	0.	0.	1508.
.00	1695.	0.	0.	1263.
-.10	1646.	0.	0.	1212.
-1.10	1436.	0.	0.	996.
-2.10	1372.	0.	0.	1033.
-2.55	1208.	0.	0.	1067.
-4.10	1140.	0.	0.	1100.