2B

PROTECCION TECHNICA ECOLOGICA (PROTECO) INC.
PART B PERMIT APPLICATION
PRD 091018622
PENUELAS, PUERTO RICO

VOLUME 3
REVISION 3

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June 30, 1986

(1052B)

APPENDIX D-6.2

LANDFILL AND

SURFACE WATER MANAGEMENT FACILITIES

ADDENDUM TO GEOTECHNICAL EVALUATION

(0641B-8)

APPENDIX D.6-2

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ITEM NO.	DESCRIPTION
1	Bearing Capacity Analysis - Landfill I & II
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9	Leachate Head/Drain System Evaluation
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ITEM 1

BEARING CAPACITY ANALYSIS

LANDFILLS I & II

[1]

[1] Pages 13A and 13B were added to this item in June 1986, Revision 2 of this Application.

SUBJECT PROTECO - GEOTECHNICAL EVALUATION JOB NO. B-511E
BEARING CAPACITY / SETTLEMENT / STABILITY EVALUATION
GENERAL DISCUSSION

THE PROTECO LANDFILLS (I & II) ARE UNDERLAIN by THE JUANA DIAZ FORMATION WHICH HAS BEEN IDENTIFIED by THE U.S.G.S AS AN UNCONSOLIDATED, PARTIALLY LITHIFIED ROCK DEPOSIT. ON-SITE ASSESSMENT & BOREHOLE DATA CONFIRM THAT THE CLAY-LIKE SUBSTRATA ARE ESSENTIALLY SOFT CLAYSTONE. FROM A SOIL MECHANICS STANDPOINT, THIS SUBSTRATA IS CLASSIFIED AS A HARD SILTY CLAY THAT IS OVER-CONSOLIDATED WITH RESPECT TO COMPRESSION CHARACTERISTICS.

STRENGTH DATA IS INFERRED FROM LABORATORY TESTING ON RE-COMPACTED SITE SOIL AND EXISTING DIKE FILL, FIELD PENETROMETER DATA, STANDARD PENETRATION TESTING. AND FIELD OBSERVATIONS.

BY BMM DATE 1/2/86	FRED C. HART ASSOCIATES, INC.	SHEET 2 OF 3 PAGE
CHK'D RCM DATE 1/29/86		

SUBJECT PROTECO - GEOTECHNICAL EVALUATION JOB NO. B-511E

BEARING CAPACITY ANALYSIS - REVISED LANDFILLS I & II

PROBLEM STATEMENT

DEMONSTRATE THAT THE REVISED LANDFILL AREAS (I & II) HAVE ADEQUATE BEARING CAPACITY TO SUPPORT THE PROPOSED LANDFILL LOADINGS.

STRENGTH DATA

A) LABORATORY TESTS {3 UNCONSOLIDATED, UNDRAINED TESTS}

1) $\phi_u = 3.2$ $C_u = 1150$ PSF (SAMPLE RE-COMPACTED TO MODIFIED PROCTOR DENSITY)

2) $\phi_u = 8.4°$ $C = 1020$ PSF {LANDFILL DIKE BORING IF-2A DEPTH SAMPLE 2.5 → 5'}

3) $\phi_u = 20°$ $C = 320$ PSF {LANDFILL DIKE BORING OL-2 DEPTH 8 → 10.5'}

B) FIELD PENETROMETER DATA

DATA PRESENTED ON LOGS FOR BORINGS I.M. 2-1, IM 1-1, IM 1-C

Q_u (AVG) FOR 19 TESTS = 3.9 TSF {SEE SHEET 2}

C) STANDARD PENETRATION TESTS (N-VALUE)

REPRESENTATIVE DATA ON BORINGS TB-1, TB-2, 11W-83, 12W-83

1) TB-1 (0 → 72' DEPTH) $N_{AVG} = 45$

2) TB-2 (0 → 45' DEPTH) $N_{AVG} = 28$
$D_{pt} > 45'$ $N > 100$

3) 11W-83 (0 → 65' DEPTH) $N_{AVG} = 40$
$D_p > 65'$ $N > 100$

4) 12W-83 (0 - 55' DEPTH) $N_{AVG} = 51$
$D_p > 55$ $N > 100$

SEE SHEET 4

RANGE OF N IS ≈ 25 → 50 AVG N ≈ 40 (INCREASES WITH DEPTH)

SUBJECT PROTECO - GEOTECHNICAL EVALUATION JOB NO. B-511E
BEARING CAPACITY ANALYSIS - REVISED LANDFILL

DETERMINE AVERAGE UNCONFINED COMPRESSIVE STRENGTHS OF CLAY SOILS AT BORINGS I.M2-1, I.M.1-1, AND I.M 1-C (See APPENDICES VOLUME III FOR RCRA PART B PERMIT APPLICATION)

BORING IM 2-1 (DEPTH 62.5') WL@36'

$$\sum_1^7 Q_u = 4.5 + 3.9 + 5.4 + 5.1 + 3.3 + 4.8 + 3.8$$
$$= 31.3 \text{ TSF}$$
$$Q_u \text{ (AVG)} = 4.5 \text{ TSF}$$

BORING I.M. 1-1 (DEPTH = 66') WL@54'

$$\sum_1^5 Q_u = 4.3 + 3.9 + 3.3 + 2.0 + 3.7 = 17.2$$
$$Q_u \text{(AVG)} = 3.4 \text{ TSF}$$

BORING I.M. 1-C (DEPTH 65') WL>65'

$$\sum_1^7 Q_u = 2.8 + 3.3 + 3.8 + 4.7 + 3.9 + 4.7 + 3.8$$
$$= 26.0$$
$$Q_u \text{(AVG)} = 3.7 \text{ TSF}$$

AVG. ALL DATA:

$$\sum_1^{19} Q_u = 74.5 \qquad Q_u \text{(AVG)} = 3.9 \text{ TSF}$$

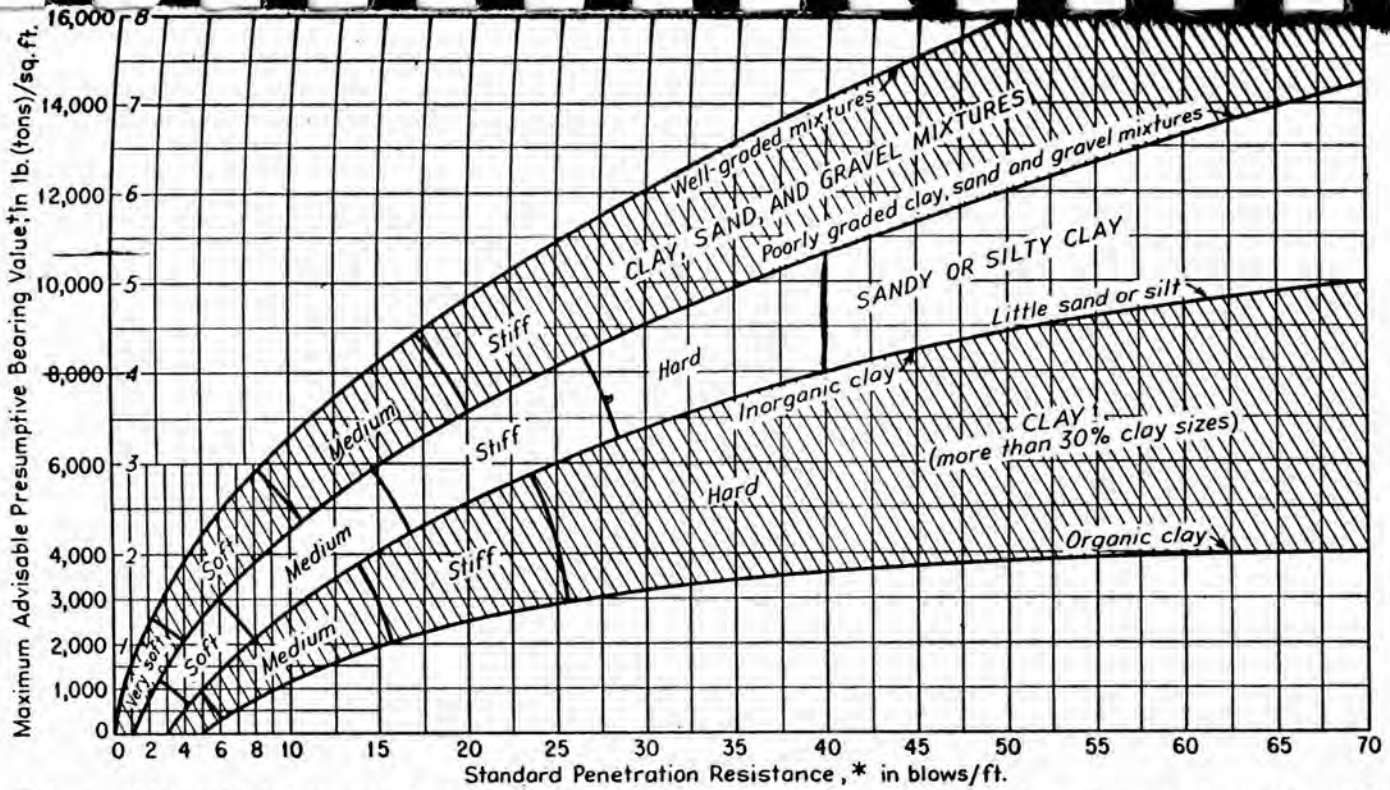
SUBJECT PROTECO - GEOTECHNICAL EVALUATION JOB NO. B-511E
BEARING CAPACITY ANALYSIS - REVISED LANDFILLS I & II

C) STANDARD PENETRATION TESTS (CONT'D)

FROM "BASIC SOIL ENGINEERING" P. 297, B.K HOUGH (FIRST ED.)
FOR HARD SANDY OR SILTY CLAY N = 40, THE MAXIMUM ADMISSABLE PRESUMPTIVE BEARING CAPACITY RANGE IS 4 → 5+ TSF. NOTE THAT IF N-VALUES THAT EXCEEDED 100 WERE USED IN AVERAGING N, RANGE COULD APPROACH 5 → 7 TSF WITH DEPTH (LIMIT OF FIG 10-13)

D) VISUAL ASSESSMENT VIA NAVFAC DM-7 (1971) TABLE 11-1.
a) FOR HARD SILTY CLAY, RANGE OR ALLOWABLE BEARING CAPACITY IS 3 → 6 TSF
b) FOR SOFT ARGILLACEOUS ROCK, RANGE IS 8 → 12 TSF

CONCLUSION: A PRESUMPTIVE BEARING CAPACITY OF 6 TSF APPEARS REASONABLE FOR THE CLAY-LIKE SUBSTRATA.



* Number of blows of 140-lb. pin-guided weight falling 30 in. per blow required to drive a split-barrel sample spoon with a 2-in. outside diameter 12 in.

† Higher values may be used for precompressed (or compacted) clays of low sensitivity than for normally loaded or extra-sensitive clays.

FIG. 10-13. Presumptive bearing values, clays and mixed soils.

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TABLE 11-1
Nominal Values of Allowable Bearing Pressures for Spread Foundations

Type of bearing material	Consistency in place	Allowable bearing pressure tons per sq ft	
		Ordinary range	Recommended value for use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks).	Hard, sound rock	60 to 100	80
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks).	Medium hard sound rock	30 to 40	35
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities.	Medium hard sound rock	15 to 25	20
Weathered or broken bed rock of any kind except highly argillaceous rock (shale).	Soft rock	8 to 12	10
Compaction shale or other highly argillaceous rock in sound condition.....	Soft rock	8 to 12	10
Well graded mixture of fine and coarse grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC).	Very compact	8 to 12	10
Gravel, gravel-sand mixtures, boulder-gravel mixtures (GW, GP, SW, SP)...	Very compact	7 to 10	8
	Medium to compact	5 to 7	6
	Loose	3 to 6	4
Coarse to medium sand, sand with little gravel (SW, SP).....	Very compact	4 to 6	4
	Medium to compact	3 to 4	3
	Loose	2 to 3	2
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)..	Very compact	3 to 5	3
	Medium to compact	2 to 4	2.5
	Loose	1 to 2	1.5
Fine sand, silty or clayey medium to fine sand (SP, SM, SC).....	Very compact	3 to 4	3
	Medium to compact	2 to 3	2
	Loose	1 to 2	1.5
Homogeneous inorganic clay, sandy or silty clay (CL, CH).....	Very stiff to hard	3 to 6	4
	Medium to stiff	1 to 3	2
	Soft	.5 to 1	.5
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)...	Very stiff to hard	2 to 4	3
	Medium to stiff	1 to 3	1.5
	Soft	.5 to 1	.5

Notes:
1. Variations of allowable bearing pressure for size, depth and arrangement of footings are given in Table 11-2.
2. Compacted fill, placed with control of moisture, density, and lift thickness, has allowable bearing pressure of equivalent natural soil.
3. Allowable bearing pressure on compressible fine grained soils is generally limited by considerations of overall settlement of structure, Table 6-1.
4. Allowable bearing pressure on organic soils or uncompacted fills is determined by investigation of individual case.

compressibility of subsoils is constant with depth, analyze consolidation settlement of the entire foundation.

5. PROPORTIONING INDIVIDUAL FOOTINGS. Where significant compression will not occur in strata below a depth equal to the distance between footings, proportion size of individual footings to give equal settlement; use formulas for immediate settlement in Figures 11-8 and 11-9. Where significant consolidation settlements may occur below this depth, select footing size on the basis of the safety factor against ultimate failure as a first trial. Analyze overall consolidation settlements for the combined effect of these individual footings. In this case, settlements are controlled by the combined stresses of all foundation units and may be little affected by alteration of individual footing areas.

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 Bowles, E.

Correlation between unconfined compressive strength and penetration number of cohesive coils has also been published, as in Table 3-3. The indicated values of unconfined compressive strength correlated to penetration number in Table 3-3 should be used cautiously, however. Experiences in different areas indicate that it is not unusual to get penetration numbers of 6 to 10 on soils where the unconfined compressive strength may be from 6 to 12 ksf, which is considerably different from data shown in the table, where q_u is approximately one-fourth of the penetration number. A valid correlation between the penetration number and the shear strength

Table 3-3 Empirical values for q_u† and consistency of cohesive soils based on the standard penetration number

Consistency	*Very soft*	*Soft*	*Medium*	*Stiff*	*Very stiff*	*Hard*
q_u, ksf	0	0.5	1.0	2.0	4.0	8.0
N, standard penetration resistance	0	2	4	8	16	32
$\gamma_{(sat)}$, pcf		100–120	110–130	120–140		

† These values should be used as a guide only. Local cohesive samples should be tested, and the relationship between N and the unconfined compressive strength q_u established as $q_u = KN$.

of cohesive (ϕ-c) soils can be made only on a local basis, and then the validity may be suspect, unless large enough quantities of tests are made to allow a statistical analysis.

In general, to correlate penetration number to the shear strength of a cohesive (ϕ-c) soil involves evaluating the following expression for the constant of proportionality:

$$q_u = KN \tag{3-4}$$

where K = proportionality constant
N = penetration number

The penetration test applied to gravel or gravelly soils and silty sands yields results which require careful interpretation. In loose gravel the voids formed when the gravel is displaced by the driving shoe of the split spoon may yield low penetration numbers. On the other hand, if the spoon pushes a large piece of rock, the number may be too high.

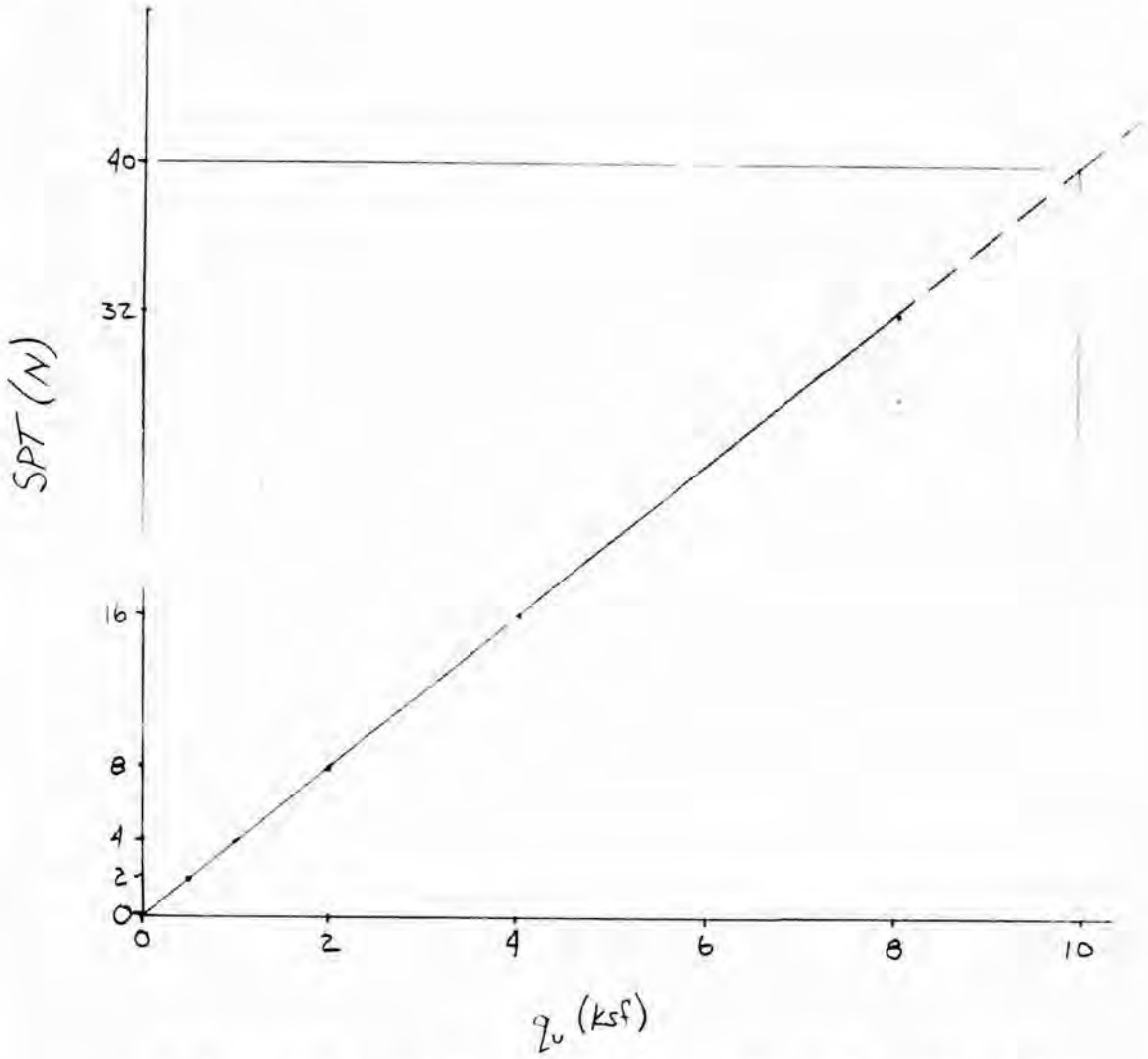
In silty sands, Meyerhof [9] suggests that the bearing capacity be reduced 50 percent [using Eqs. (2-20) and (2-21)]. For gravel the bearing capacity computed by these methods should be increased by a factor of 2.0. Terzaghi and Peck [10] have recommended that if the soil is a very fine, or silty, saturated sand and if the measured penetration number N is greater than 15, an adjusted design value of N' be used, as

$$N' = 15 + \tfrac{1}{2}(N - 15) \tag{3-5}$$

This was based on the assumption that the critical void ratio occurs at approximately

SUBJECT PROTECO - GEOTECHNICAL EVALUATION JOB NO. B-511E
BEARING CAPACITY ANALYSIS - REVISED LANDFILLS 1 & 2

E) plotting attached data from Table 3.3 Bowles Foundation Analysis and Design p.126



extrapolation of the curve shows that for $N_{as} \sim 40$ an ult. bearing capacity of approx. 10 ksf or 5 tsf

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SUBJECT PROTECO - GEOTECHNICAL EVALUATION JOB NO. B5"E
BEARING CAPACITY ANALYSIS - REVISED LANDFILL

DETERMINE AVERAGE N-VALUES FOR BORINGS TB-1, TB-2, 11W-83, 12W-83 AS PER SHEET 3

(a) TB-1

$$\sum_{1}^{17} N = 11 + 14 + 18 + 35 + 29 + 25 + 13 + 21 + 44 + 46 + 43 + 50 + 74 + 79 + 96 + 90 + 82 = 770$$

Ava N = 770/17 = 45

(b) TB-2

$$\sum_{1}^{12} N = 8 + 11 + 16 + 16 + 19 + 24 + 23 + 28 + 41 + 45 + 38 + 61 = 330$$

Ava N = 330/12 = 28

(c) 11W-83

$$\sum N_1^7 = 26 + 32 + 30 + 42 + 39 + 52 + 57 = 278$$

Ava N = 278/7 = 40

(d) 12W-83

$$\sum N_1^5 = 58 + 60 + 45 + 36 + 55 = 254$$

Ava N = 254/5 = 51

Range Ava 'N' say 25 → 50

Ava 'N' is 1632/41 ≅ 40

SUMMARY OF STRENGTH EVALUATION / PARAMETERS

1) SUBSTRATA IS ROCK (CLAYSTONE) THAT INCREASES IN STRENGTH WITH DEPTH

2) BASED ON PENETROMETER READINGS, AUGERACK N-VALUES AND VISUAL ASSESSMENT, A PRESUMPTIVE BEARING VALUE OF 6 TSF APPEARS REASONABLE.

3) THE FOUNDATION ADEQUACY CAN BE MORE READILY ASSESSED AS A SLOPE STABILITY PROBLEM EXTENDING THROUGH THE LANDFILL MATERIAL AND INTO THE UNDERLYING SUBSTRATA.

SUBJECT PROTECO - GEOTECHNICAL EVALUATION JOB NO. B-511E
BEARING CAPACITY ANALYSIS - REVISED LANDFILL

ASSESS BEARING CAPACITY FOR VARIOUS STAGES OF LANDFILL CONSTRUCTION

REFERENCE DRAWINGS:

LANDFILL I : B511E-L19, L13, L23, L10

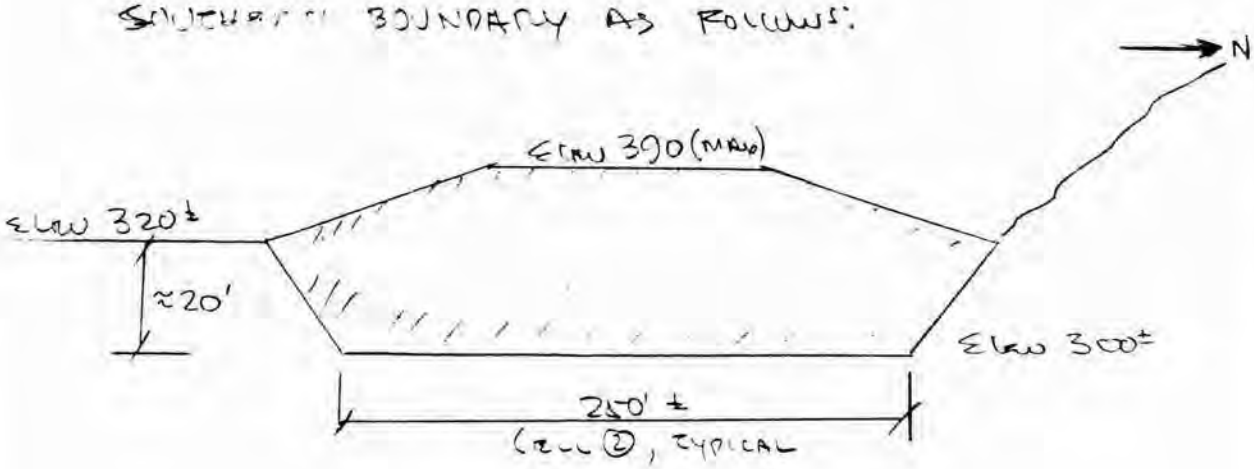
LANDFILL II : B511E-L20, , L27

LANDFILL MATERIAL WILL CONSIST OF BOTH STABILIZED & DIRECT DISPOSAL WASTE. ASSUME AVERAGE UNIT WEIGHT = 100 PCF

LANDFILL I

FROM SECTION C-C ON DRAWING B511E-L23, THE AVERAGE DEPTH OF LANDFILL IS APPROXIMATELY 90 FEET, AND THE AVERAGE DEPTH OF EXCAVATION BELOW EXISTING GRADE TO THE PROPOSED BASE OF LANDFILL IS 35 FEET

REVIEW OF DWGS. L10 & L13 INDICATE THAT THE LAST CONFINEMENT WITHIN LANDFILL I OCCURS ALONG ITS SOUTHERN BOUNDARY AS FOLLOWS:

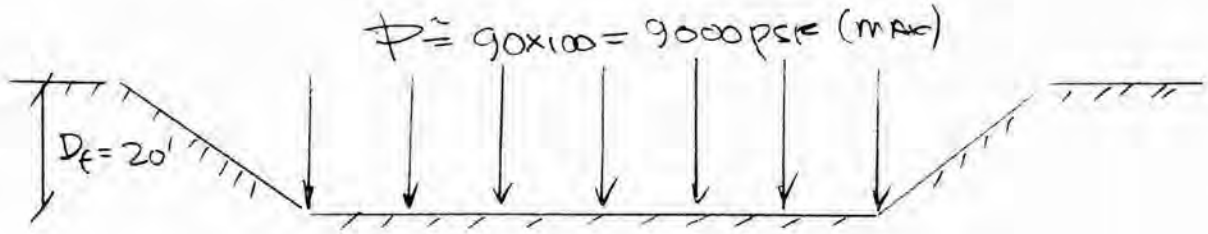


- NTS -

DETERMINE BEARING CAPACITY OF SUBSTRATA BASED ON THIS CONDITION WHICH IS MOST CRITICAL SECTION.

SUBJECT PROTECO - GEOTECHNICAL EVALUATION JOB NO. B-511E
BEARING CAPACITY ANALYSIS - REVISED LANDFILL

LOAD CONDITION



a) ASSUMING LAB PARAMETERS APPLY,

$\phi_u = 10°$, $C_u = 830$ psf } AVERAGE OF LAB TESTS

where $N_c = 9.6$, $N_q = 2.7$, $N_\gamma = 1.2$

then

$$q_{ult} = cN_c + D_f(\gamma)N_q + \tfrac{1}{2}\gamma B N_\gamma$$

$$= 830(9.6) + 20(120)(2.7) + \tfrac{1}{2}(120)(250)(1.2)$$

$$= 7968 + 6480 + 18{,}000 = 32{,}448$$

$q_{ult}/3 = 10{,}816$ psf > 9000 psf ∴ OK FOR EXTREME CONDITION WHICH ASSUMES FOUNDATION CONSISTS OF RE-COMPACTED SITE MATERIAL UNDER INSTANTANEOUS LOADING CONDITION.

b) BASED ON PENETROMETER VALUE OF 3.9 TSF ; AND ASSUMING $\phi = 0°$

$$q_{ult} = \left(\frac{3.9\ \text{TSF}}{2}\right) \cdot 5.7 + \frac{20(120)(1.0)}{2000}$$

$$= (11.1 + 1.2)\ \text{TSF} = 12.3\ \text{TSF}$$

$q_{allow} = 12.3/3 = 4.1$ TSF < 4.5 TSF BUT ADEQUATE FOR CONDITIONS ASSUMED

COMPUTED FACTOR OF SAFETY $= 12.3/4.5 = 2.7$

SINCE $\phi = 0$, CONSERVATIVE.

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SUBJECT PROTECO – GEOTECHNICAL EVALUATION JOB NO. B-511E
BEARING CAPACITY EVALUATION – REVISED LANDFILL

c)

FROM PRESUMPTIVE BEARING EVALUATION, THE PROPOSED LANDFILL LOADS (3000 FT – 4 TSF), IS LESS THAN 6 TSF

LANDFILL I

REVIEW OF THE REFERENCED DRAWINGS INDICATES THAT LOADING CONDITIONS IN LANDFILL II ARE LESS SEVERE THAN LANDFILL I. AND REQUIRES NO FURTHER EVALUATION.

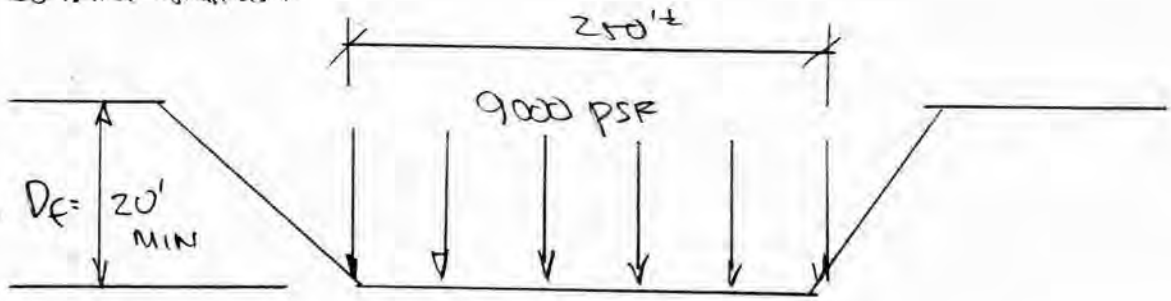
CONCLUSION: THE SITE SUBSTRATA HAS SUFFICIENT STRENGTH TO SUSTAIN THE PROPOSED LANDFILL LOADS.

SUBJECT PROTECO - GEOTECHNICAL EVALUATION JOB NO. B-511E
BEARING CAPACITY ANALYSIS - REVISED LANDFILL

CHECK BEARING CAPACITY ASSUMING THAT SITE IS UNDERLAIN ENTIRELY BY COMPACTED FILL WITH EFFECTIVE STRENGTH PARAMETERS OF

$\phi' = 22^\circ \quad C' = 325$ { AVG. OF MH & CH PARAMETERS FROM TABLE 9-1 NAVFAC DM-7

LOADING CONDITION



for $\phi' = 22^\circ \quad C' = 325$ $\quad N_c = 17$, $N_q = 8$, $N_\gamma = 7$ } FIG 3-15, P 128 FOUNDATION ENGR HANDBOOK

$$q_{ult} = cN_c + D_f(\gamma)(N_q) + \tfrac{1}{2}\gamma B N_\gamma$$

$$= 17(325) + 20(120)(8) + \tfrac{1}{2}(120)(250)(7)$$

$$= 5525 + 19{,}200 + 105{,}000$$

$$q_{ult} = 129{,}725 \text{ PSF}$$

$$q_{allow} = q_{ult}/3 = 43{,}241 \approx 43 \text{ KSF} \gg 9 \text{ KSF}$$

∴ ADEQUATE IF CONSERVATIVELY ASSUME ALL SUBSTRATA IS RECOMPACTED SITE MATERIAL!

TABLE 9-1
Typical Properties of Compacted Materials

Group symbol	Soil type	Range of maximum dry unit weight, pcf	Range of optimum moisture, percent	Typical value of compression: At 1.4 tsf (20 psi)	Typical value of compression: At 3.6 tsf (50 psi)	Typical strength characteristics: Cohesion (as compacted) psf	Typical strength characteristics: Cohesion (saturated) psf	Typical strength characteristics: ϕ (Effective stress envelope) degrees	Typical strength characteristics: Tan ϕ	Typical coefficient of permeability ft/min.	Range of CBR values	Range of subgrade modulus k lb/cu in.
				percent of original height								
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix.	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	.....	.....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	.....	.....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	5×10^{-5}	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	2×10^{-6}	.....	
SC	Clayey sands, poorly graded sand-clay mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	5×10^{-7}	5 - 20	100 - 300
ML	Inorganic silts and clayey silts .	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	10^{-5}	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	5×10^{-7}	.....	
CL	Inorganic clays of low to med. plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	10^{-7}	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	5×10^{-7}	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	10^{-7}	15 or less	50 - 150
OH	Organic clays and silty clays . . .	65 - 100	45 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	25 - 100

AVG = 325 22°

Notes:
1. All properties are for condition of "standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown.
(....) indicates insufficient data available for an estimate.

REF NAVFAC DM-7

Sheet 13 B
GF 13

ITEM 2

SETTLEMENT ANALYSIS

LANDFILLS I & II

1 Pages 5 and 6 of this item were revised in June 1986, Revision 2 of this Application.

BY BMM DATE 1/24/86	FRED C. HART ASSOCIATES, INC.	SHEET 1 OF 6 PAGE
CHK'D RCM DATE 2/12/86		

SUBJECT PROTECO - GEOTECHNICAL EVALUATION JOB NO. B511E

SETTLEMENT ANALYSIS

PROBLEM STATEMENT

ASSESS THE POTENTIAL FOR SETTLEMENT OF THE SUBSTRATA & DETERMINE ITS EFFECT ON THE THE PROPOSED LINER SYSTEM

ASSUMPTIONS & AVAILABLE DATA

1) ASSUME THAT SETTLEMENT WILL BE PRODUCED ONLY FROM THE RELATIVELY SOFT (HARD CLAY) STRATA THAT OVERLIES THE HARD LIMESTONE ENCOUNTERED AT VARYING DEPTHS WITHIN THE SITE. CONTOURS OF THE TOP OF LIMESTONE ARE PRESENTED ON UN-NUMBERED WORKSHEET

2) REVIEW OF DWGS B511-L6 AND B511E-L10 INDICATE THAT THE "COMPRESSIBLE STRATA UNDER LANDFILL I VARIES FROM NEAR 0 TO A MAXIMUM OF 50 FEET THICK IN A N→S DIRECTION BELOW THE BASE OF THE LANDFILL (SEE BELOW)

3) REVIEW OF DWGS B511-L7 AND B511E-L16 INDICATES THAT THE "COMPRESSIBLE" STRATA UNDER LANDFILL II VARIES FROM 50 TO ABOUT 200 FEET THICK IN A NE → SW DIRECTION BELOW THE BASE OF THE LANDFILL.

4) GROUNDWATER LEVELS ON DWGS B-511E-L5, L6, & L7 INDICATE THAT GROUNDWATER IS ABOUT 10 FEET BELOW PROPOSED GRADE (BASE) IN BOTH LANDFILLS. FOR ANALYSIS, CAN ASSUME AT BASE (WORST CASE).

5) USE PREVIOUSLY DEVELOPED CONSOLIDATION PARAMETERS BASED ON SOIL CLASSIFICATION TESTS

ie $C_{cr} = 0.025$ $\gamma_{SAT} = 125$ PCF [illegible]

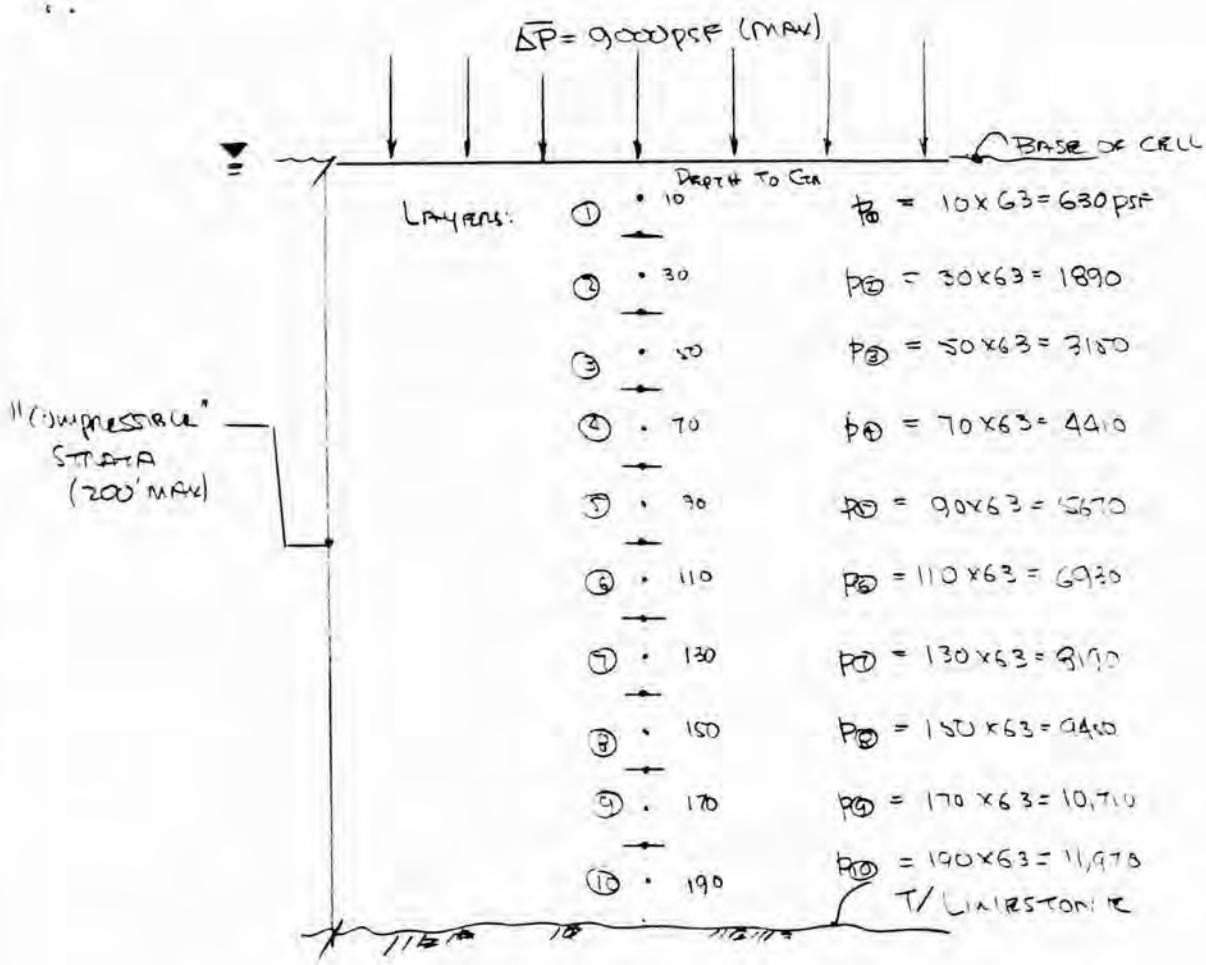
$P_c >>$ PROPOSED STRESS INCREASES + [illegible]

SUBJECT PROTECO – GEOTECHNICAL EVALUATION JOB NO. B511E
SETTLEMENT ANALYSIS

CALCULATIONS.

A CONSERVATIVE ESTIMATE OF SETTLEMENTS (& DIFFERENTIALS) CAN BE MADE BY ASSUMING MAXIMUM LOADING CONDITIONS AND NO STRESS ATTENUATION WITH DEPTH DUE TO THE LARGE AREA OF LOADING. MAXIMUM FOUNDATION LOADING OCCURS IN LANDFILL I ≈ 9000 PSF.

∴



CALCULATE SETTLEMENT FOR EACH LAYER BY:

$$\Delta H = \frac{C_{cr}}{1+e_0}(H)\ \log\left(\frac{p_0 + \Delta\bar{P}}{p_0}\right)$$

$\Delta\bar{P} = 9000$ psf
$C_{cr} = 0.025$
$e_0 = 0.70$
$H = 20(2) = 40$

$$\therefore \frac{C_{cr}}{1+e_0}(H) = 3.5$$

BY TBMM DATE 1/24/86	FRED C. HART ASSOCIATES, INC.	SHEET 3 OF 6 PAGE
CHK'D RCM DATE 2/12/86		

SUBJECT PROTECO – GEOTECHNICAL EVALUATION JOB NO. B-511E
SETTLEMENT ANALYSIS

CALCULATE SETTLEMENTS FOR VARIOUS LAYERS:

$\Sigma_{\Delta H}$

LAYER ① $\Delta H_1 = \underbrace{\frac{0.025}{1+.7}(20\times12)}_{3.5} \log\left(\frac{9000+630}{630}\right)$

$= 3.5 \times 1.18 = 4.13$ inches — 4.1"

② $\Delta H_2 = 3.5 \log\left(\frac{9000+1890}{1890}\right)$

$= 3.5 \times 0.76 = 2.7''$ — 6.8"

③ $\Delta H_3 = 3.5 \log\left(\frac{9000+3150}{3150}\right)$ — 8.9"

$= 3.5 \times 0.58 = 2.05''$

④ $\Delta H_4 = 3.5 \times \log\left(\frac{9000+4410}{4410}\right)$ — 10.6"

$3.5 \times 0.48 = 1.7''$

⑤ $\overline{\Delta H_5} = 3.5 \log\left(\frac{9000+5670}{5670}\right)$

$= 3.5 \times 0.41 = 1.4$ — 12.0"

⑥ $\overline{\Delta H_6} = 3.5 \log\left(\frac{9000+6930}{6930}\right)$

$3.5 \times 0.36 = 1.3$ — 13.3"

⑦ $\Delta H_7 = 3.5 \log\left(\frac{9000+8190}{8190}\right)$

$3.5 \times 0.32 = 1.1$ inch. — 14.4

⑧ $\overline{\Delta H_8} = 3.5 \log\left(\frac{9000+9450}{9450}\right)$

$= 3.5 \times 0.29 = 1$ inch — 15.4

SUBJECT PROTECO - GEOTECHNICAL EVALUATION JOB NO. B-5115
SETTLEMENT ANALYSIS

(9) $\Delta H_q = 3.5 \log\left(\frac{9000 + 10{,}710}{10{,}710}\right)$

$= 3.5 \times 0.26 = 0.9''$ 16.3 (19)

(10) $\overline{\Delta H_{10}} = 3.5 \log\left(\frac{9000 + 11{,}970}{11{,}970}\right)$

$= 3.5 \times 0.24 = 0.8''$ 17.1'' (20)

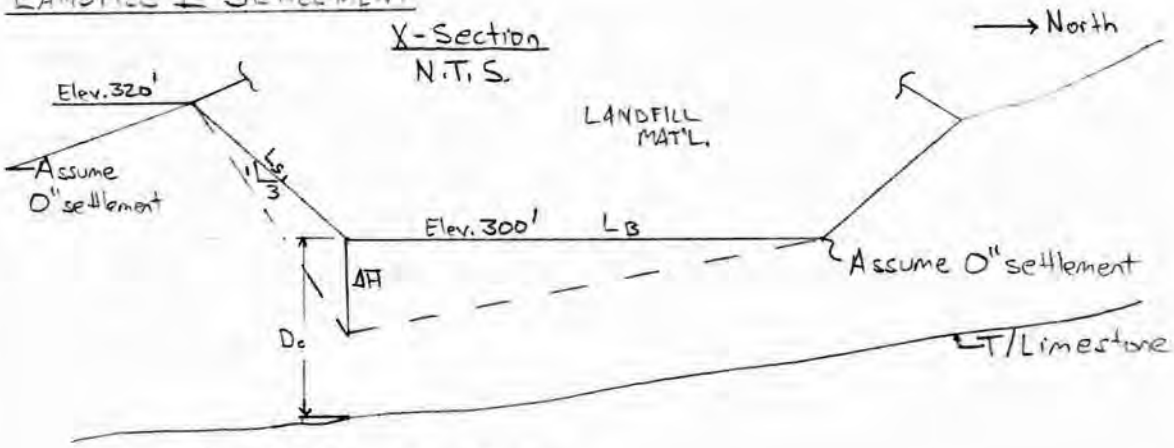
ASSUME PLASTIC RESPONSE NEGLIGIBLE RELATIVE TO CONSOLIDATION & ASSESS EFFECTS OR SETTLEMENT DIFFERENTIALS ON LANDFILLS I & II.

SUBJECT PROTECO JOB NO. 01511-00-85002-07

SETTLEMENT POTENTIAL AND LINER STRAIN

LANDFILL I SETTLEMENT

X-Section
N.T.S.



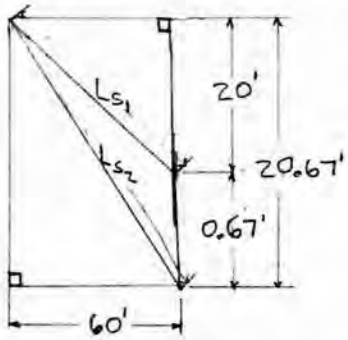
$\Delta H_{max} \cong 8" = .67'$ $D_c \approx 50$ FT.

1) Length of base $\approx 250' = L_B$

$\overline{\Delta L}$ of liner along base $= \left[(250)^2 + (8/12)^2)\right]^{1/2} - 250 \cong 250$ FT

$e = \Delta L / L \approx 0/250 \approx 0$ ∴ strain no problem along base

2) Assume liner fixed at top of south slope



$L_{s_1} = (20^2 + 60^2)^{1/2} = 63.25$ FT.

$L_{s_2} = (20.67^2 + 60^2)^{1/2} = 63.46$ FT.

$\Delta L = 63.46 - 63.25 = 0.21' = 2.5"$

$e = \Delta L / L = 0.21/63.25 = 0.0033$

$e = 0.33\%$ NEGLIGIBLE << 13%

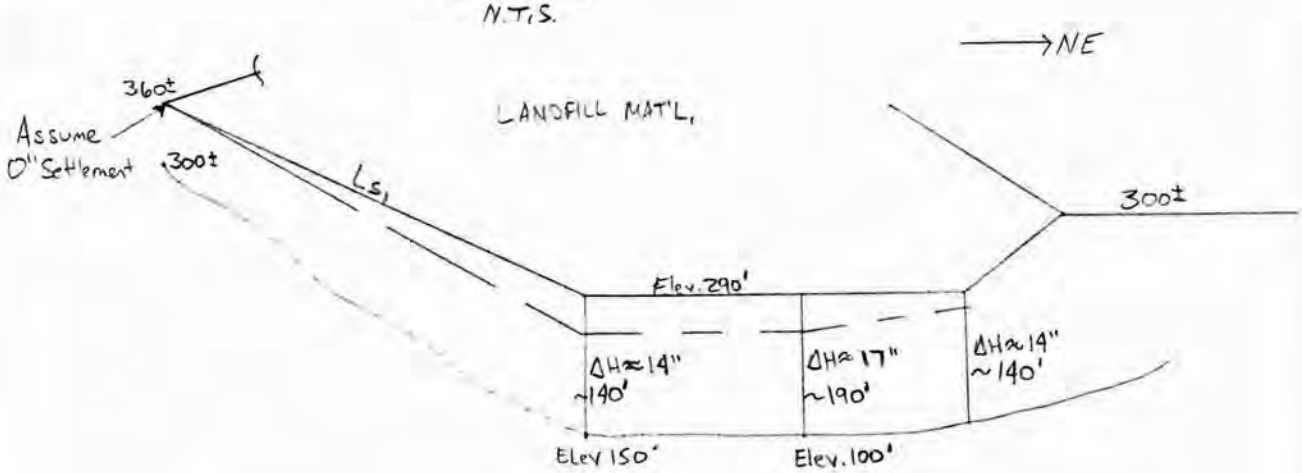
∴ differential settlement is no problem considering liner elongation, and differential settlement is in the design direction for leachate flow

SUBJECT PROTECO JOB NO. 01511-00-85002-07

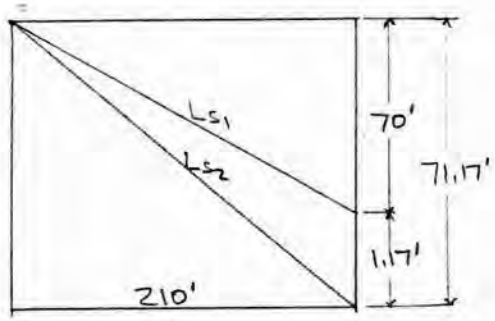
SETTLEMENT POTENTIAL AND LINER STRAIN

LANDFILL II SETTLEMENT

X-Section
N.T.S.



1)



$L_{s1} = (210^2 + 70^2)^{1/2} = 221.36'$

$L_{s2} = (210^2 + 71.17^2)^{1/2} = 221.73'$

$\Delta L = 0.37' = 4.5''$

$e = \Delta L / L = 0.37 / 221.36' = 0.0017$

$e = 0.17\%$ NEGLIGIBLE $\ll 13\%$

2) general trend of settlement will be adverse to flow, however, settlement should be insignificant considering that piping capacity is much greater than design requirements

ITEM 3

STRESSES IN LINER FROM

LANDFILL SETTLEMENT

BY RCM DATE 2/5/86	FRED C. HART ASSOCIATES, INC.	SHEET 1 OF 2 PAGE
CHK'D BMM DATE 2/10/86		

SUBJECT PROTECO GEOTECHNICAL EVALUATION JOB NO. B511

PROBLEM STATEMENT

DETERMINE LATERAL STRESSES DUE TO SETTLEMENT INDUCED STRAIN.

ASSUMPTIONS AND AVAILABLE DATA

LANDFILL 1 LINER STRAIN ≈ 0.08 %
LANDFILL 2 LINER STRAIN ≈ 0.30 %
} SEE P. 405 FROM BMM WORK 1/24/86

80 MIL HDPE : MODULUS OF ELASTICITY = 110,000 PSI (GUNDLE)

SOLUTION

MAXIMUM TENSILE STRESS

LANDFILL 1: $\sigma_T = 110{,}000(.0008) = 88$ PSI

LANDFILL 2: $\sigma_T = 110{,}000(.0030) = 330$ PSI

80 MIL HDPE TENSILE STRENGTH AT YIELD = 190 P/IN. WIDTH (GUNDLE)

80 MIL HDPE TENSILE STRENGTH AT BREAK = 320 P/IN. WIDTH (GUNDLE)

$80 \text{ MIL}\left(\frac{.0001 \text{ IN}}{\text{MIL}}\right)(1 \text{ IN}) = 0.08 \text{ IN}^2/\text{IN. WIDTH}$

LANDFILL 1: σ_T / IN. WIDTH = 88 PSI (.08 IN²/IN. WIDTH)
= 7.0 P/IN. WIDTH LL 190 OK

LANDFILL 2: σ_T / IN. WIDTH = 330 PSI (.08 IN²/IN. WIDTH)
= 26.4 P/IN. WIDTH LL 190 OK

CONCLUSION

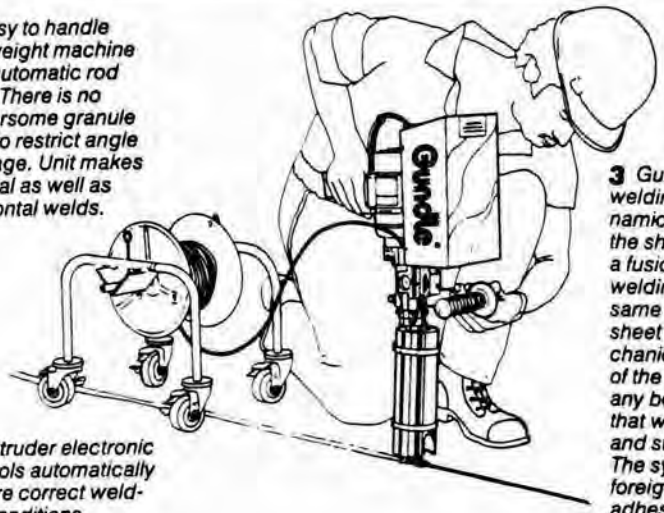
TENSILE STRESSES IN HDPE LINERS UNDERLYING LANDFILLS 1 AND 2 ARE LESS THAN THE TENSILE STRESS AT YIELD AS QUOTED BY THE MANUFACTURER (GUNDLE). THE TENSILE ARE THEREFORE ACCEPTABLE.

Sheet 2 of 2

Gundle Fusion Welding: the process which consistently gives you seams strong and as durable as the sheet.

Until now, the most failure-prone part of flexible membrane liners has been the seam. Various bonding methods have been used, but none could produce a seam that would withstand as much physical stress and weathering as the sheet itself. Now Gundle has the answer: Gundle Fusion Welding . . . a patented, field-applied process. Its effectiveness has been proven repeatedly in rigorous laboratory tests and in successful field applications worldwide.

For more detailed information request a copy of our Gundline® Flexible Membrane Lining Systems brochure.



1 *Easy to handle lightweight machine with automatic rod feed. There is no bothersome granule feed to restrict angle of usage. Unit makes vertical as well as horizontal welds.*

2 *Extruder electronic controls automatically assure correct welding conditions.*

3 *Gundle's unique welding method dynamically integrates the sheets. It creates a fusion weld using welding rod of the same material as the sheet itself. This mechanical integration of the sheets eliminates any boundary layers that would be weak and subject to failure. The system uses no foreign solvents or adhesives which can degenerate over time.*

SPECIFICATIONS

PROPERTY	TEST METHOD	GAUGE (NOMINAL)					
		20 Mil	30 Mil	40 Mil	60 Mil	80 Mil	100 Mil
Density (g/cc) (Minimum)	ASTM D1505	0.94	0.94	0.94	0.94	0.94	0.94
Minimum Tensile Properties (Each direction)	ASTM D638 Type IV Dumb-bell at 2 ipm.						
1. Tensile Strength at Break (Pounds/inch width)		80	120	160	240	320	400
2. Tensile Strength at Yield (Pounds/inch width)		50	70	95	140	190	240
3. Elongation at Break (Percent)		700	700	700	700	700	700
4. Elongation at Yield (Percent)		13	13	13	13	13	13
5. Modulus of Elasticity (Pounds/square inch)	ASTM D882	110,000	110,000	110,000	110,000	110,000	110,000
Tear Resistance Initiation (Pounds minimum)	ASTM D1004 Die C	15	22	30	45	60	75
Low Temperature	ASTM D746 Procedure B	−40°F	−40°F	−40°F	−40°F	−40°F	−40°F
Dimensional Stability (Each direction, percent change maximum)	ASTM D1204 212°F 1 hr.	±2	±2	±2	±2	±2	±2
Volatile Loss (Maximum %)	ASTM D1203 Method A	0.1	0.1	0.1	0.1	0.1	0.1
Environmental Stress Crack (Minimum hours)	ASTM D1693	750	750	750	750	750	750

ITEM 4

STABILITY ANALYSIS OF LANDFILLS

1

1 The calculations contained in this item were revised in June 1986, Revision 2 of this Application

SUBJECT PROTECO JOB NO. 01511-00-85002-07
SLOPE STABILITY ANALYSIS LANDFILL 1

PROBLEM STATEMENT

Determine the stability of landfill 1.

REFERENCES

1) Siegel, Ronald A., STABL User Manual, JHRP-75-9 Project No. C-36-36K File No. 6-14-11, Purdue University, and the Indiana State Highway Commission, June 26, 1978

2) FRED C. HART drawing numbers B511E-L13 and B511E-L13.

SOLUTION

- determine the most probable location for a critical potential failure surface in landfills 1 and 2

The location of the critical potential failure surface would most probably occur South of Landfill 1. Landfill 1 is considered more critical from a stability standpoint since the height of refuse placement is higher than in Landfill 2. The worst case for Landfill 1 is section C-C on drawing B511E-L13 in plan and in section on drawing B511E-L23. The worst case for section C-C is when Stage 2 has been completed and the refuse is at maximum height. Landfill 1 Section C-C Stage 2 is then the worst case for foundation stability encountered during the life of the structure.

- assign appropriate soil parameters for the slope stability analysis and determine boundaries for the different soil parameters

Soil zonation (boundaries) are as given on pages 6, 7, and 8. Soil strength parameters are conservative considering that the water table depth is below potential critical failure surfaces, unconsolidated undrained test results are conservative since the landfill will not be filled instantaneously but over a period of time, hence a value of $\phi = 22°$ is more appropriate than an unconsolidated undrained value of $\phi = 32°$ for recompacted clay.

Soil zonation boundaries for use in the STABL 3 computer analysis are as given on pages 6 & 7 and were determined by use of Section C-C on drawing BS11E-L23. A detail of the liner system is shown on page 8. For purposes of computer analysis and modeling, liner materials (HDPE liner and geofabric) were divided in order to provide an accurate model that would account for different strength parameters, i.e. HDPE liner with sand above and recompacted clay below requires 2 different strength parameters.
The assumed soil parameters are as follows:

Refuse: $\phi = 30°$, $c = 0$, $\gamma = 100$ pcf (Soil Type No. 1)

Assuming a value of zero for cohesion is conservative. A friction angle of 30° is used based on the type of refuse. Refuse is typically industrial waste, stabilized industrial waste, fly ash kiln dust, and limestone. The assumed value of 30° is therefore conservative. A friction angle of 26.5° was used in Cases 13 and 14. The value was obtained by using the equation F.S. = $\tan\phi'/\tan\beta$ (p.542, Perloff and Baron, Soil Mechanics). A factor of safety of 1.5 and a slope of 1V:3H was used in order to find ϕ. A unit weight of 100 pcf was used and is conservative. A unit of 50 pcf was used in Cases 12 and 14 in order to see what affect lower unit weight would have on the stability of the slope.

Protective cover material: $\phi = 30°$, $c = 0$, $\gamma = 120$ pcf (Soil Type No. 2)

The protective cover material is sand. A friction angle of 30° is typical for a well-graded sand. A unit weight of 120 pcf is also typical. The thickness of the sand layer is 1 foot.

Non-woven geotextile fabric: $\phi = 26°$, $c = 0$, $\gamma = 140$ pcf (Soil Type No. 3)

Since the geotextile fabric is sandwiched between two sand layers it is not divided. A friction angle of 26° is conservative considering sand with a $\phi = 30°$ and CZ600 geomembrane (see page 9). Unit weight is assumed.

HDPE / sand interface: $\phi = 18°$, $c = 0$, $\gamma = 140$ pcf (Soil Type Nos. 4 and 5)

The HDPE / sand interface friction angle of 18° was obtained from Martin, Koerner, and Whitty, International Conference on Geomembranes, page 193. See page 9. The ϕ angle assumes sand with a friction angle of 30° and HDPE geomembrane. Unit weight is assumed.

Recompacted clay and subgrade: $\phi = 22°$, $c = 325$ psf, $\gamma = 125$ pcf (Soil Type Nos. 6 and 7)

The typical subgrade material is an inorganic clay with high plasticity (CH). Typical Properties of Compacted Materials, Table 9-1, from NAVFAC was used in order to assign strength values. The average of the MH and CH soil type strength parameters were used in order to be conservative. See page 10. Note that the strength parameters assigned for Cases 1-3 for Soil Types 6 and 7 are not as given above, however, no potential critical failure circles passed through these soils.

- consider failures through the refuse, along weak planes in the liner system, and deep failures through the subgrade, also consider the affects of earthquake loading, change in refuse unit weight and friction angle

Case 1: Case 1 considers the slope with ϕ of the refuse equal to 30° under an imposed horizontal earthquake coefficient, a_H, of 0.15 (see pages 11 through 14. A critical failure surface searching method was used in the STABL3 program. A minimum factor of safety of 1.192 was found. Note that imposed limitations on surface generation contained the critical surface. The critical surface is shown in section on page 14.

Case 2: Case 2 considers the slope with ϕ of the refuse equal to 20°. The assumed friction angle is very conservative and was investigated in order to see what factor of safety would result. A critical failure surface searching method was used in the STABL3 program. A minimum factor of safety of 1.165 was found. Note that imposed limitations on surface generation contained the critical surface. The critical surface is shown in section on page 14.

Case 3: Case 3 is identical to Case 1 except that no earthquake load is imposed and the search limitations are not the same. A minimum factor of safety of 1.822 was found.

Cases 4-14 consider failures through the recompacted clay, subgrade, and along the weak zone in the liner system. Also considered are changes in the refuse unit weight and friction angle. Trial failure surfaces are as specified.

Case 4: Case 4 considers the slope with no earthquake loading, ϕ of refuse equal to 30°, ϕ of clay layers equal to 22°, c of clay layers equal to 325 psf. The failure circle was forced through the clay layers. The factor of safety was found to be 2.082. See pages 19 and 20.

Case 5: Case 5 is the same as Case 4 except that the failure circle was forced through the weak zone of the liner system where $\phi = 18°$. See pages 21 and 22. The factor of safety was found to be 1.892.

Case 6 & 7: Since the factor of safety decreased when a failure plane was forced along the bottom of the liner system, a second analysis was conducted allowing the failure plane to pass along the liner system on the interim berm. Case 6 is the static case with a factor of safety of 1.726. Case 7 is the dynamic case (earthquake loading) with a factor of safety of 1.129. See pages 23, 24, and 25.

Cases 8 & 9: Case 7 is the most critical case for cases 4 through 7. Case 7 was further investigated to see if moving the point where the failure surface propagated upward from the liner system was in the critical location. Inspection of Cases 8 and 9 show that Case 7 is more critical. See pages 26, 27, 28, and 29.

Cases 10 & 11: Cases 10 and 11 employ the same rationale as Cases 8 and 9 except the point where the failure surface terminates at the refuse surface is investigated. Inspection of Cases 10 and 11 show that Case 7 is more critical. See pages 30, 31, and 32.

Case 12: Case 12 considers the affect of a lower refuse unit weight on stability. Case 12 is the same as Case 7 except that the refuse unit weight is 50 pcf. The factor of safety was found to be 1.202. See pages 33 and 34.

Cases 13 & 14: Case 13 considers the slope with refuse friction angle of 26.5° and refuse unit weight of 100 pcf, all other parameters the same as in Case 7. The factor of safety was found to be 1.088. Case 14 is identical to Case 13 except that the refuse unit weight is 50 pcf. The factor of safety was found to be 1.141. See pages 35, 36, and 37.

CONCLUSION

1) The section considered is the most critical considering refuse height and staging.
2) The critical potential failure surface is a surface passing through the sand/HDPE liner interface.
3) The factor of safety increases with increases in refuse unit weight.

BOUNDARY COORDINATES

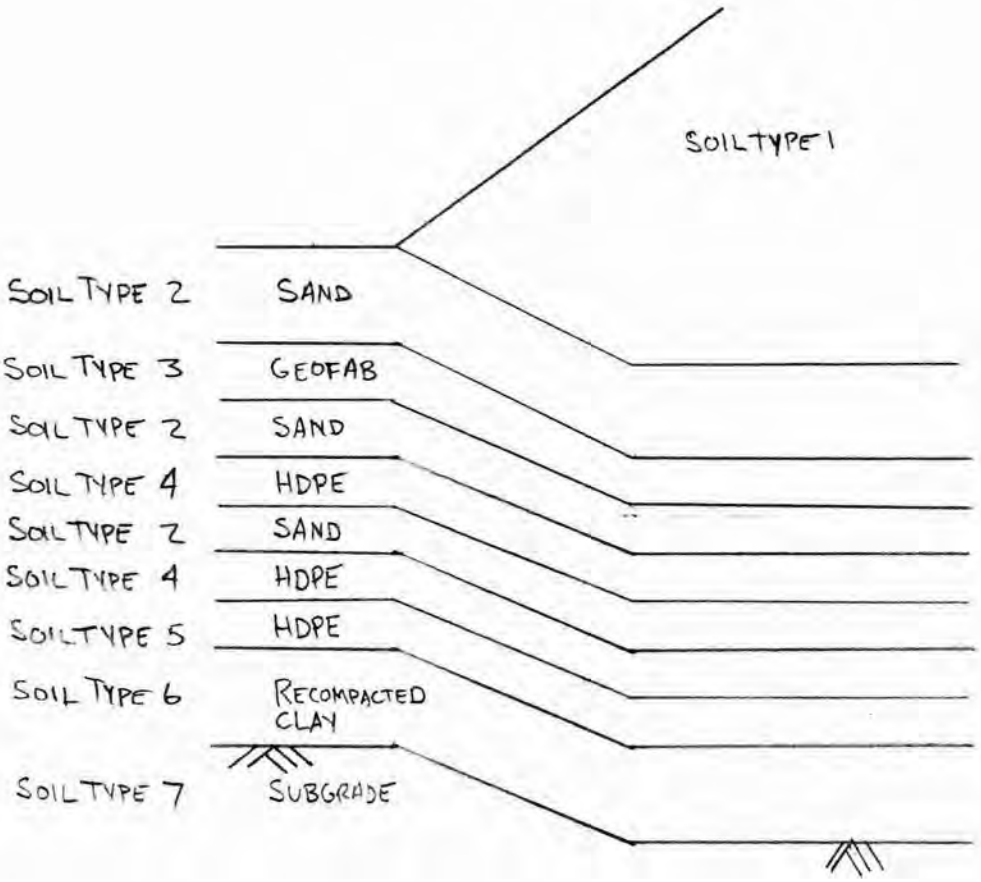
7 TOP BOUNDARIES
89 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	31.04	116.00	32.04	2
2	116.00	32.04	143.00	40.04	2
3	143.00	40.04	153.00	40.04	2
4	153.00	40.04	198.00	53.00	1
5	198.00	53.00	345.00	103.10	1
6	345.00	103.10	440.00	123.00	1
7	440.00	123.00	543.00	109.00	1
8	153.00	40.04	191.00	28.04	2
9	191.00	28.04	224.00	27.04	2
10	224.00	27.04	388.00	28.04	2
11	388.00	28.04	415.00	37.04	2
12	415.00	37.04	426.00	37.04	2
13	426.00	37.04	460.00	27.04	2
14	460.00	27.04	543.00	27.04	2
15	.00	30.04	116.00	31.04	3
16	116.00	31.04	143.00	39.04	3
17	143.00	39.04	153.00	39.04	3
18	153.00	39.04	191.00	27.04	3
19	191.00	27.04	224.00	26.04	3
20	224.00	26.04	388.00	27.04	3
21	388.00	27.04	415.00	36.04	3
22	415.00	36.04	426.00	36.04	3
23	426.00	36.04	460.00	26.04	3
24	460.00	26.04	543.00	26.04	3
25	.00	30.02	116.00	31.02	2
26	116.00	31.02	143.00	39.02	2
27	143.00	39.02	153.00	39.02	2
28	153.00	39.02	191.00	27.02	2
29	191.00	27.02	224.00	26.02	2
30	224.00	26.02	388.00	27.02	2
31	388.00	27.02	415.00	36.02	2
32	415.00	36.02	426.00	36.02	2
33	426.00	36.02	460.00	26.02	2
34	460.00	26.02	543.00	26.02	2
35	.00	28.52	116.00	29.52	4
36	116.00	29.52	143.00	37.52	4
37	143.00	37.52	153.00	37.52	4
38	153.00	37.52	191.00	25.52	4
39	191.00	25.52	224.00	24.52	4
40	224.00	24.52	388.00	25.52	4
41	388.00	25.52	415.00	34.52	4
42	415.00	34.52	426.00	34.52	4
43	426.00	34.52	460.00	24.52	4
44	460.00	24.52	543.00	24.52	4

[illegible]	[illegible]	34.52	426.00	34.52	4
43	426.00	34.52	460.00	24.52	4
44	460.00	24.52	543.00	24.52	4
45	.00	28.51	116.00	29.51	2
46	116.00	29.51	143.00	37.51	2
47	143.00	37.51	153.00	37.51	2
48	153.00	37.51	191.00	25.51	2
49	191.00	25.51	224.00	24.51	2
50	224.00	24.51	388.00	25.51	2
51	388.00	25.51	415.00	34.51	2
52	415.00	34.51	426.00	34.51	2
53	426.00	34.51	460.00	24.51	2
54	460.00	24.51	543.00	24.51	2
55	.00	27.04	116.00	28.04	4
56	116.00	28.04	143.00	36.04	4
57	143.00	36.04	153.00	36.04	4
58	153.00	36.04	191.00	25.04	4
59	191.00	25.04	224.00	24.04	4
60	224.00	24.04	388.00	25.04	4
61	388.00	25.04	415.00	34.04	4
62	415.00	34.04	426.00	34.04	4
63	426.00	34.04	460.00	24.04	4
64	460.00	24.04	543.00	24.04	4
65	.00	27.02	116.00	28.02	5
66	116.00	28.02	143.00	36.02	5
67	143.00	36.02	153.00	36.02	5
68	153.00	36.02	191.00	25.02	5
69	191.00	25.02	224.00	24.02	5
70	224.00	24.02	388.00	25.02	5
71	388.00	25.02	415.00	34.02	5
72	415.00	34.02	426.00	34.02	5
73	426.00	34.02	460.00	24.02	5
74	460.00	24.02	543.00	24.02	5
75	.00	27.00	116.00	28.00	6
76	116.00	28.00	143.00	36.00	6
77	143.00	36.00	153.00	36.00	6
78	153.00	36.00	191.00	25.00	6
79	191.00	25.00	224.00	24.00	6
80	224.00	24.00	388.00	25.00	6
81	388.00	25.00	415.00	34.00	6
82	415.00	34.00	426.00	34.00	6
83	426.00	34.00	460.00	24.00	6
84	460.00	24.00	543.00	24.00	6
85	.00	23.00	116.00	24.00	7
86	116.00	24.00	268.00	20.00	7
87	268.00	20.00	378.00	22.00	7
88	378.00	22.00	501.00	18.50	7
89	501.00	18.50	543.00	18.50	7

SUBJECT PROTECO JOB NO. 01511-00-85002-07

SLOPE STABILITY ANALYSIS LANDFILL 1



SOIL TYPE NOS. 2-6

NOT TO SCALE

linear and the data spread in a given locus of points was nominal.

After each shear failure, the direction of deformation was reversed, and the test repeated. The purpose of this exercise was to indicate residual friction angles where membrane tension is alternately increased and reduced as the level of a storage lagoon changes. Such reversals of strain direction may tend to align particles along the shear plane, and reduce slip resistance. However, the difference between initial and repeated shear strengths was negligible in all cases.(5)

MATERIALS TESTED AND RESULTS

Three granular soil types were used in these tests:

(1) Ottawa sand (SP) with d_{10} = 0.42 mm; CU = 1.9 and rounded particle shapes.
(2) Concrete sand (SP) with d_{10} = 0.20 mm; CU = 2.6 and angular particle shapes.
(3) Mica schist silty sand (SM) with d_{10} = 0.057 mm; CU = 5.1 and angular particle shapes.

Thus the three soil types selected give a contrast in particle shape, size and uniformity. They are limited however, to granular soils with essentially no plasticity.

Four types of geomembranes (using five separate surfaces) were used in these tests. They were all tested in their manufactured directions.

(1) High density polyethylene (HDPE) which was 20 mils thick and can be characterized as being stiff, hard and smooth as far as physical or frictional characteristics are concerned.
(2) Ethylene propylene diene monomer (EPDM) which was 30 mils thick and can be characterized as being flexible, soft and smooth.
(3) Polyvinyl chloride (PVC) which was 30 mils thick and characterized as being of medium stiffness and hardness and rough on one side while smooth on the other side. Both sides were used during these tests.
(4) Chlorosulfonated polyethylene (CSPE) which was reinforced with a fabric scrim and was 36 mils thick. It is characterized as being of medium stiffness and hardness, but was of wavy roughness due to the laminated 10 x 10 scrim reinforcement contained within it.

Four types of geotextiles were used in these tests which represented each of the general manufacturing classifications of these materials.(6) They were all tested in their manufactured directions.

(1) Woven monofilament polypropylene fabric (Carthage Mills Polyfilter X) which is characterized as being a thin, stiff fabric with a relatively high percent open area as far as physical or frictional characteristics are concerned.
(2) Woven split film (tape) polypropylene fabric (Mirafi 500 X) which is characterized as being a thin, flexible fabric with a low percent open area.
(3) Nonwoven heat set polypropylene fabric (duPont 3401) which is characterized as being a thin, flexible fabric with a relatively low open area.
(4) Nonwoven needled polypropylene fabric (Crown Zellerbach 600) which is characterized as being a compressible, thick, bulky, very flexible fabric with a relatively high open area.

These three soil types, four geomembranes types and four geotextile types were tested within their own categories and against one another in the manner described in the previous section. The results are given in Table 1 in two ways. The principal information (for design purposes) is given as angular values of friction angle; "ϕ" values for the soil by itself and "δ" values for the composite behavior. In parenthesis is given the relative amount (for comparison purposes) of mobilized soil strength that the geomembrane or geotextile gives, i.e.,

$$E = \frac{\tan \delta}{\tan \phi}$$

where

E = efficiency ratio
$\tan \delta$ = tangent of soil to material friction angle
$\tan \phi$ = tangent of soil friction angle, where
$\tau = c + \bar{\sigma}_n \tan \phi$
c = cohesion (zero for these granular soils)
$\bar{\sigma}_n$ = effective normal stress

Table 1 - Summary of Friction Angle and Efficiencies (in Parentheses) For Soils, Geomembranes and Geotextiles Testing in this Study

(a) Soil to Geomembrane Friction Angles

Geomembrane \ Soil	Concrete Sand (ϕ = 30°)	Ottawa Sand (ϕ = 28°)	Mica Schist (ϕ = 26°)
EPDM	24° (.80)	20° (.71)	24° (.92)
PVC (Rough)	27° (.90)	-	25° (.96)
PVC (Smooth)	25° (.83)	-	21° (.81)
CSPE	25° (.83)	21° (.75)	23° (.88)
HDPE	18° (.60)	18° (.64)	17° (.65)

(b) Soil to Geotextile Friction Angles

Geomembrane \ Soil	Concrete Sand (ϕ = 30°)	Ottawa Sand (ϕ = 28°)	Mica Schist (ϕ = 26°)
CZ 600	30° (1.00)	26° (.93)	25° (.96)
Typar 3401	26° (.87)	-	-
Polyfilter X	26° (.87)	-	-
500 X	24° (.80)	24° (.86)	23° (.88)

(c) Geomembrane to Geotextile Friction Angles

Geotextile \ Geomembrane	EPDM	PVC (R)	PVC (S)	CSPE	HDPE
CZ 600	23°	23°	21°	15°	8°
Typar 3401	18°	20°	18°	21°	11°
Polyfilter X	17°	11°	10°	9°	6°
500 X	21°	20°	24°	13°	10°

INTERPRETATION OF RESULTS

Table 1, parts "a" and "b" show the results of the direct shear tests for friction between various soils and synthetic materials in terms of friction angle (ϕ or δ)

TABLE 9-1
Typical Properties of Compacted Materials

Group symbol	Soil type	Range of maximum dry unit weight, pcf	Range of optimum moisture, percent	Typical value of compression: At 1.4 tsf (20 psi) percent of original height	Typical value of compression: At 3.6 tsf (50 psi) percent of original height	Typical strength characteristics: Cohesion (as compacted) psf	Typical strength characteristics: Cohesion (saturated) psf	Typical strength characteristics: ϕ (Effective stress envelope) degrees	Typical strength characteristics: Tan ϕ	Typical coefficient of permeability ft/min.	Range of CBR values	Range of subgrade modulus k lb/cu in.
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix.	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	.....	.....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	.....	.....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	5×10^{-5}	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	2×10^{-6}	.....	
SC	Clayey sands, poorly graded sand-clay mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	5×10^{-7}	5 - 20	100 - 300
ML	Inorganic silts and clayey silts .	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	10^{-5}	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	5×10^{-7}	.....	
CL	Inorganic clays of low to med. plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	10^{-7}	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	5×10^{-7}	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	10^{-7}	15 or less	50 - 150
OH	Organic clays and silty clays ...	65 - 100	45 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	25 - 100

Avg. 32 psf 22°

Notes:

1. All properties are for condition of "standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown.

(....) indicates insufficient data available for an estimate.

7-9-2

10/37

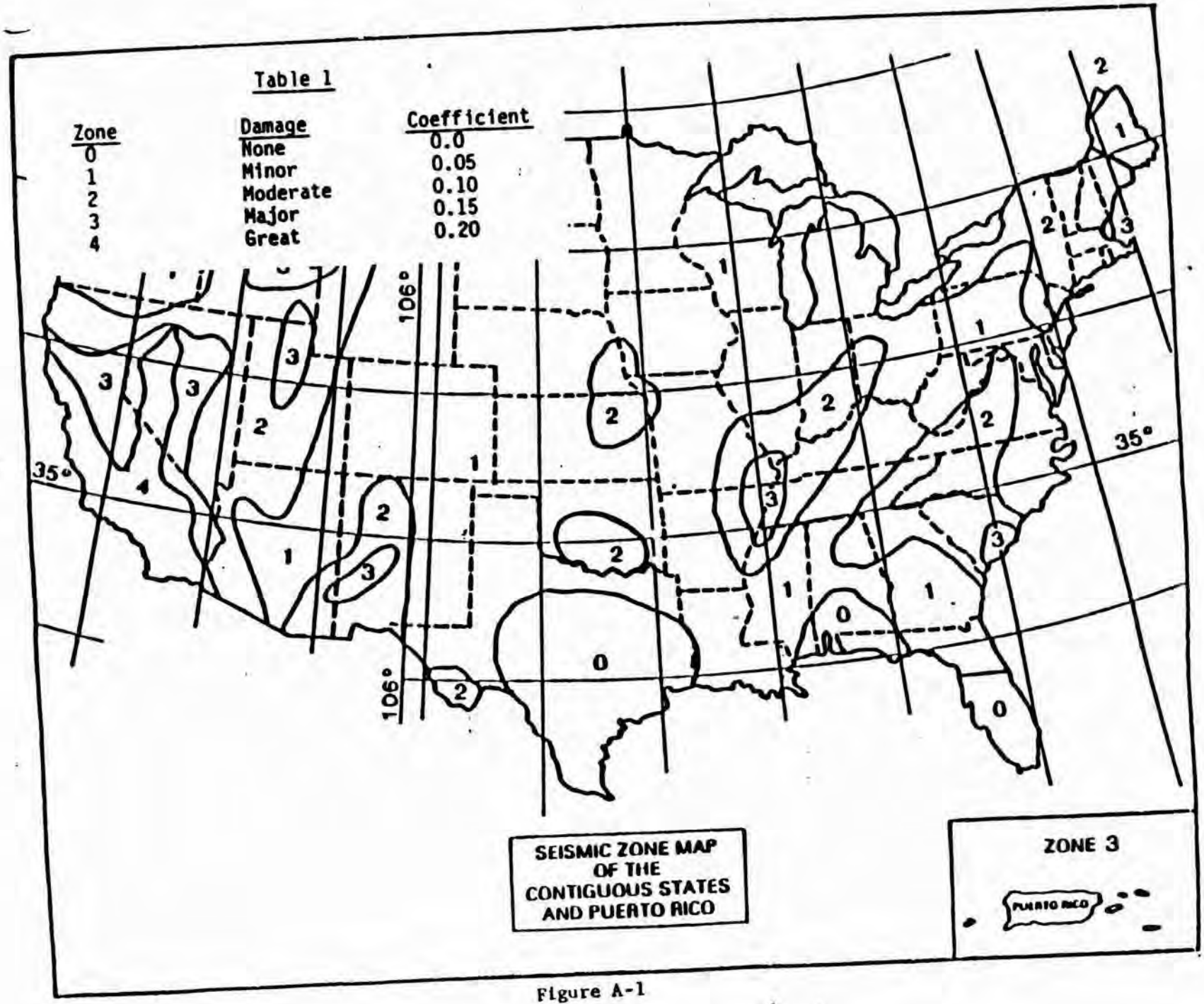


Table 1
Zone
0
1
2
3
4
Damage
None
Minor
Moderate
Major
Great
Coefficient
0.0
0.05
0.10
0.15
0.20
35°
106°
SEISMIC ZONE MAP
OF THE
CONTIGUOUS STATES
AND PUERTO RICO
ZONE 3
PUERTO RICO

Figure A-1

CASE 1

7 TYPE(S) OF SOIL

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PI
1	1	100.0	100.0	.0	30.0	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	2000.0	20.0	.00	.0	
1	7	125.0	125.0	2000.0	20.0	.00	.0	

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .150 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =-2115.0 PSF
A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM
TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

CASE 1

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = 150.00 FT.
AND X = 190.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 290.00 FT.
AND X = 360.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

10.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

RESTRICTIONS HAVE BEEN IMPOSED UPON THE ANGLE OF INITIATION.
THE ANGLE HAS BEEN RESTRICTED BETWEEN THE ANGLES OF -30.0 AND .0 DI

G.

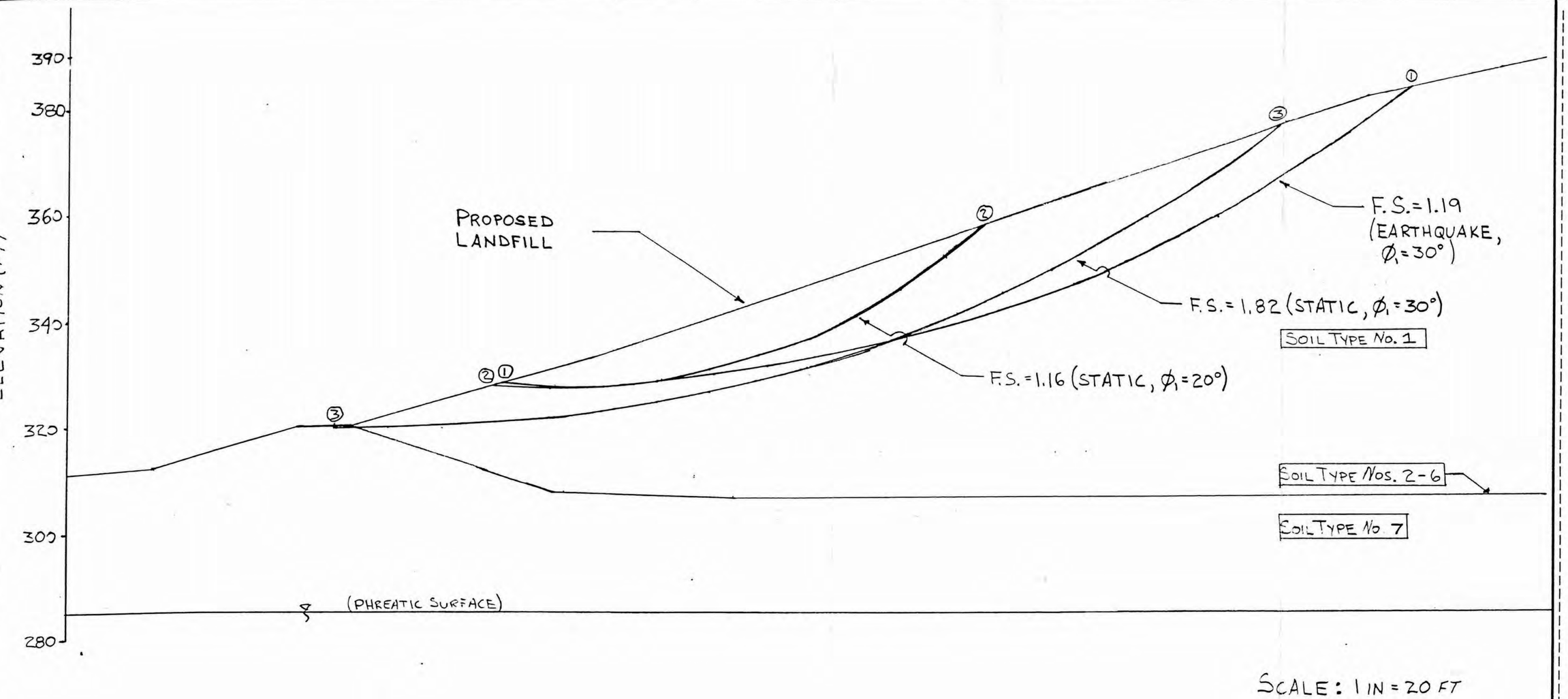
1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL
FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL
FIRST.

SAFETY FACTORS ARE CALCULATED BY THE MODIFIED BISHOP METHOD.

FAILURE SURFACE SPECIFIED BY 20 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	181.11	48.14
2	191.11	48.00
3	201.11	48.24
4	211.09	48.86
5	221.04	49.86
6	230.94	51.23
7	240.79	52.98
8	250.56	55.10
9	260.25	57.59
10	269.83	60.44
11	279.30	63.65
12	288.64	67.22
13	297.84	71.15
14	306.89	75.41
15	315.76	80.02
16	324.46	84.96
17	332.96	90.22
18	341.25	95.81
19	349.33	101.70
20	353.31	104.84

*** 1.192 ***



SCALE: 1 IN = 20 FT

BY RCM DATE 2/5/86
CHK'D DATE
SUBJECT PROTECO SLOPE STABILITY
LANDFILL 1
JOB NO. 13511

FRED C. HART ASSOCIATES, INC.

SHEET OF
PAGE

ISOTROPIC SOIL

CASE 2

7 TYPE(S) OF SOIL

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PI
1	1	100.0	100.0	.0	20.0	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	2000.0	20.0	.00	.0	
1	7	125.0	125.0	2000.0	20.0	.00	.0	

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

CASE2

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = 145.00 FT.
AND X = 190.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 200.00 FT.
AND X = 400.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

10.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

RESTRICTIONS HAVE BEEN IMPOSED UPON THE ANGLE OF INITIATION.
THE ANGLE HAS BEEN RESTRICTED BETWEEN THE ANGLES OF -30.0 AND .0 D
G.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL
FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL
FIRST.

SAFETY FACTORS ARE CALCULATED BY THE MODIFIED BISHOP METHOD.

FAILURE SURFACE SPECIFIED BY 12 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	180.00	47.82
2	189.99	47.32
3	199.98	47.63
4	209.92	48.75
5	219.73	50.68
6	229.36	53.39
7	238.73	56.87
8	247.80	61.09
9	256.49	66.04
10	264.75	71.67
11	272.53	77.96
12	273.26	78.65

*** 1.165 ***

7 TYPE(S) OF SOIL

CASE3

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	P
1	1	100.0	100.0	.0	30.0	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	2000.0	20.0	.00	.0	
1	7	125.0	125.0	2000.0	20.0	.00	.0	

1 1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

CASE3

18/37

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = 80.00 FT.
AND X = 170.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 250.00 FT.
AND X = 340.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

10.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

RESTRICTIONS HAVE BEEN IMPOSED UPON THE ANGLE OF INITIATION.
THE ANGLE HAS BEEN RESTRICTED BETWEEN THE ANGLES OF -30.0 AND .0 DE
G.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL
FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL
FIRST.

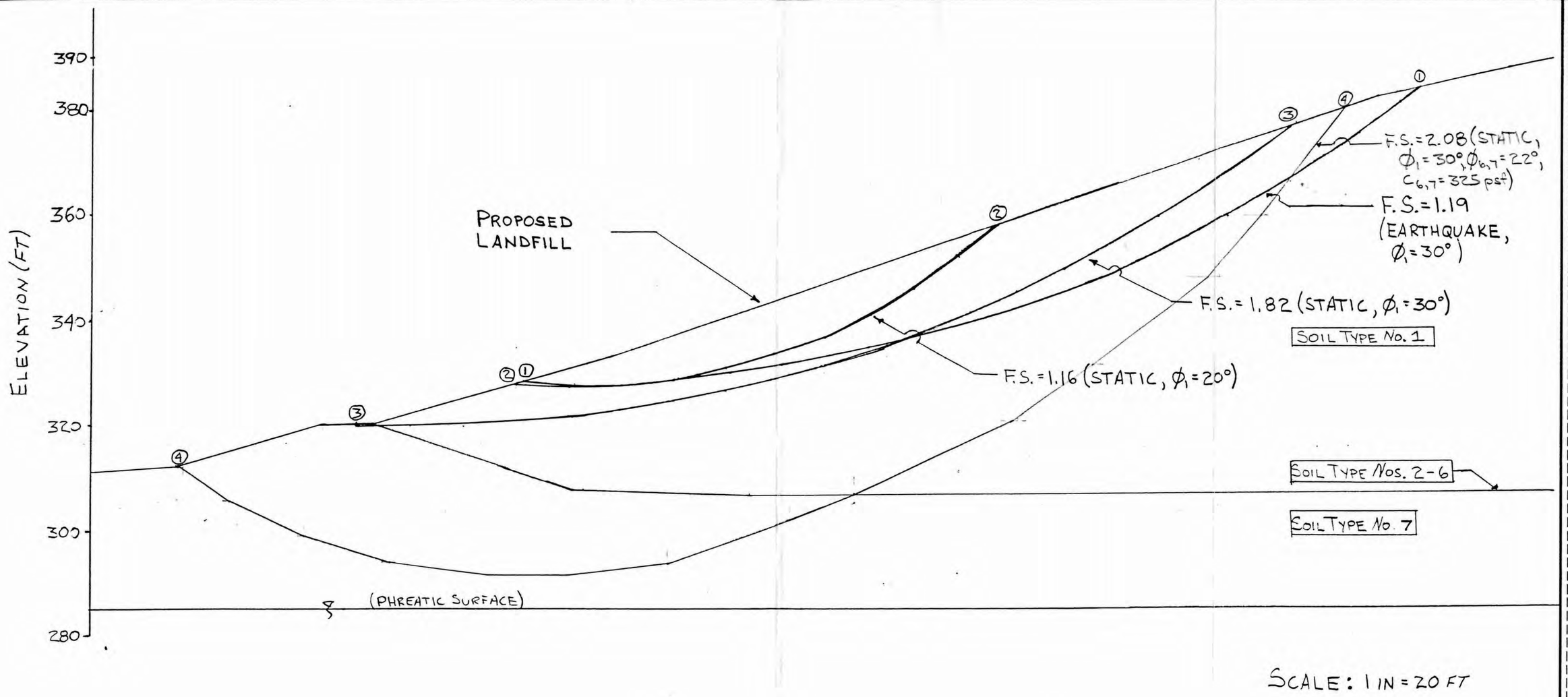
SAFETY FACTORS ARE CALCULATED BY THE MODIFIED BISHOP METHOD.

FAILURE SURFACE SPECIFIED BY 21 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	150.00	40.04
2	160.00	39.90
3	170.00	40.12
4	179.98	40.70
5	189.94	41.64
6	199.85	42.93
7	209.72	44.58
8	219.51	46.58
9	229.23	48.92
10	238.86	51.62
11	248.39	54.66
12	257.80	58.04
13	267.09	61.75
14	276.24	65.79
15	285.23	70.16
16	294.06	74.85
17	302.72	79.85
18	311.20	85.16
19	319.48	90.76
20	327.55	96.66
21	328.65	97.53

*** 1.822 ***

CASE 4



SUBJECT PROTECO SLOPE STABILITY LANDFILL 1

JOB NO. 8591

CASE 4

7 TYPE(S) OF SOIL

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PI
1	1	100.0	100.0	.0	30.0	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	325.0	22.0	.00	.0	
1	7	125.0	125.0	325.0	22.0	.00	.0	

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

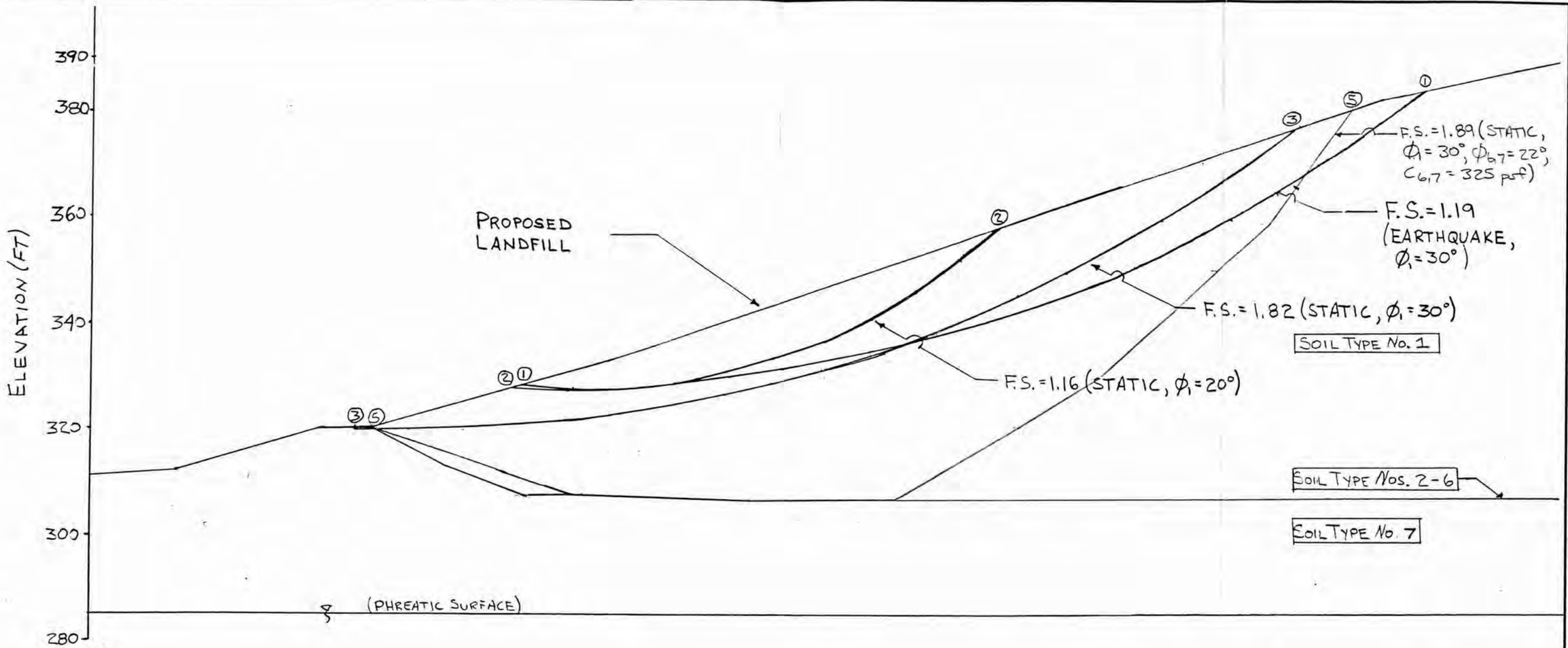
1 TRIAL FAILURE SURFACE SPECIFIED BY 14 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	116.00	32.04
2	125.00	26.50
3	139.00	20.00
4	156.00	15.00
5	175.00	12.50
6	190.00	12.50
7	210.00	15.00
8	234.00	22.00
9	246.00	27.00
10	277.00	43.50
11	292.00	53.50
12	315.00	72.50
13	325.00	82.50
14	341.00	101.74

******<<<<< CALLED PLTN - STBL 430 >>>>>******
******<<<<< RETURNED FROM PLTN - STBL 430 >>>>>******

FACTOR OF SAFETY FOR THE PRECEDING SPECIFIED SURFACE = 2.082

CASE 5



SCALE: 1 IN = 20 FT

SUBJECT PROTECO SLOPE STABILITY
LANDFILL 1

JOB NO. 8511

ISOTROPIC SOIL PARAMETERS

CASE 5

22/37

7 TYPE(S) OF SOIL

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PI
1	1	100.0	100.0	.0	30.0	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	325.0	22.0	.00	.0	
1	7	125.0	125.0	325.0	22.0	.00	.0	

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

1 TRIAL FAILURE SURFACE SPECIFIED BY 10 COORDINATE POINTS

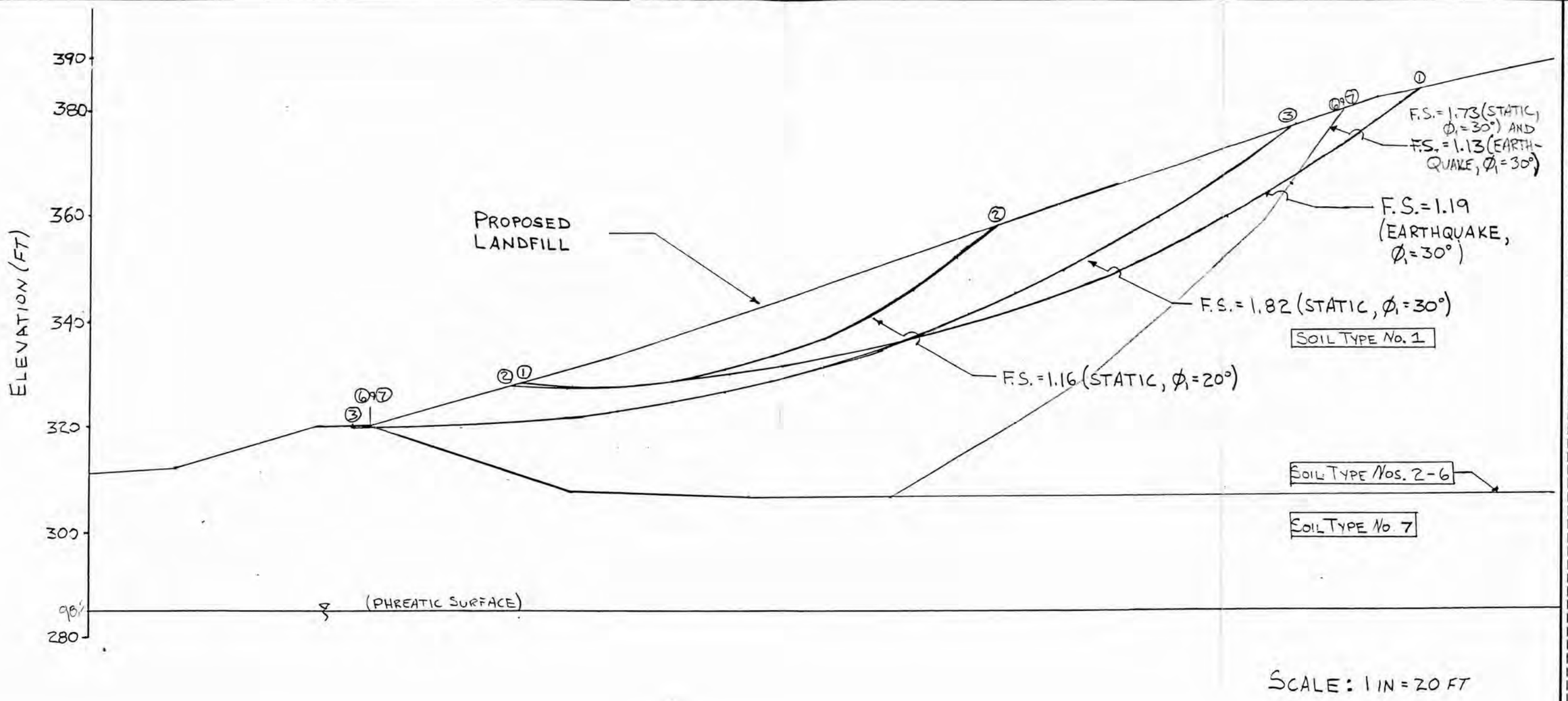
POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	153.00	40.04
2	165.00	34.00
3	180.00	29.00
4	224.00	24.02
5	246.00	24.15
6	277.00	43.50
7	292.00	53.50
8	315.00	72.50
9	325.00	82.50
10	341.00	101.74

******<<<<< CALLED PLTN - STBL 430 >>>>>******
******<<<<< RETURNED FROM PLTN - STBL 430 >>>>>******

FACTOR OF SAFETY FOR THE PRECEDING SPECIFIED SURFACE = 1.892

1

CASES 6 AND 7



SUBJECT PROTECO SLOPE STABILITY LANDFILL 1

JOB NO. 1851

CASE6

7 TYPE(S) OF SOIL

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PI
1	1	100.0	100.0	.0	30.0	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	325.0	22.0	.00	.0	
1	7	125.0	125.0	325.0	22.0	.00	.0	

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

1 TRIAL FAILURE SURFACE SPECIFIED BY 9 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	153.00	40.04
2	191.00	25.02
3	224.00	24.02
4	246.00	24.15
5	277.00	43.50
6	292.00	53.50
7	315.00	72.50
8	325.00	82.50
9	341.00	101.74

******<<<<< CALLED PLTN - STBL 430 >>>>>******
******<<<<< RETURNED FROM PLTN - STBL 430 >>>>>******

FACTOR OF SAFETY FOR THE PRECEDING SPECIFIED SURFACE = 1.726

CASE 7

7 TYPE(S) OF SOIL

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PT
1	1	100.0	100.0	.0	30.0	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	325.0	22.0	.00	.0	
1	7	125.0	125.0	325.0	22.0	.00	.0	

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

U T OF WATER = 62.40

PIEZOMETRIC SURFACE NO. IFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y ER T)
1	.00	5.00
2	343.00	5.00

HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .150 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =-2115.0 PSF

1 TRIAL FAILURE SURFACE SPECIFIED BY 9 COORDINATE POINTS

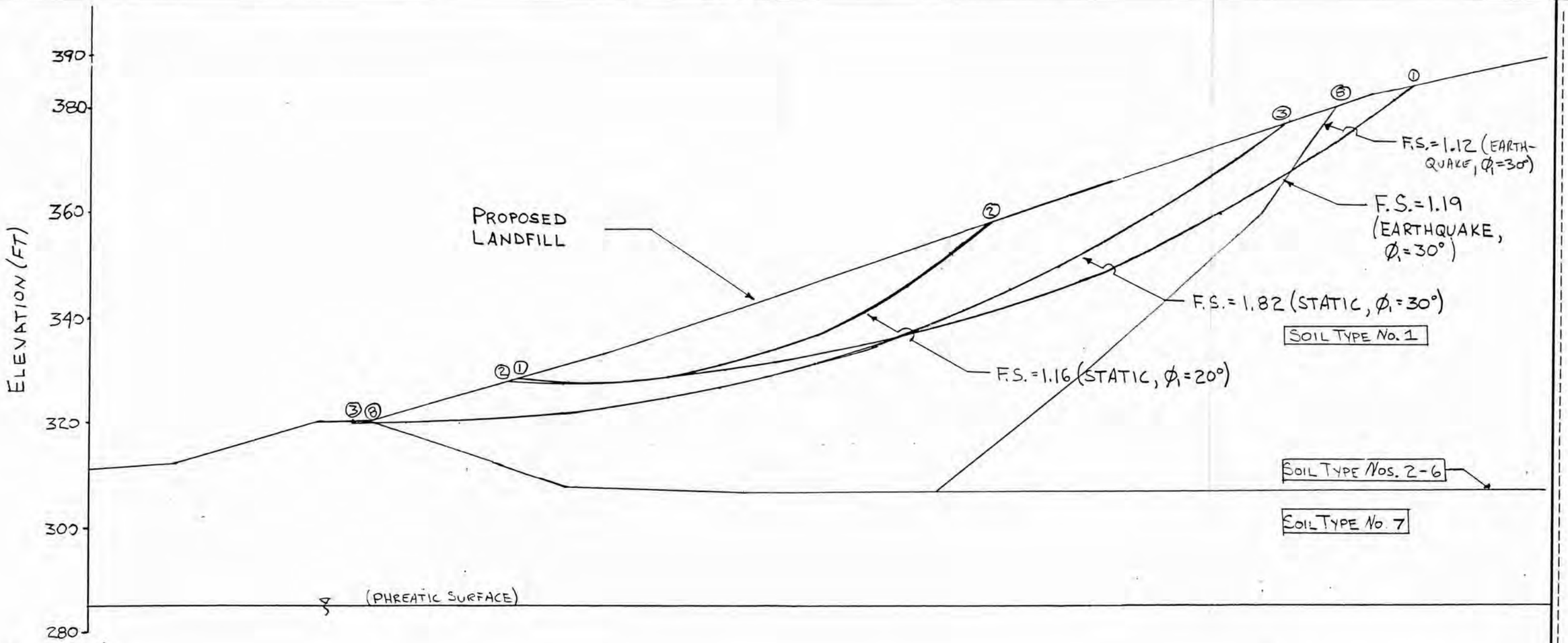
POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	153.00	40.04
2	191.00	35.02
3	224.00	24.02
4	246.00	24.15
5	277.00	43.50
6	292.00	53.50
7	315.00	72.50
8	325.00	82.50
9	341.00	101.74

************* CALLED PLTN - STBL 430 >>>>>>******

************* RETURNED FROM PLTN - STBL 430 >>>>>>******

FACTOR OF SAFETY FOR THE PRECEDING SPECIFIED SURFACE = 1.129

CASE 8



SCALE: 1 IN = 20 FT

SUBJECT PROTECO SLOPE STABILITY LANDFILL 1

JOB NO. 8591

CASE 8

7 TYPE(S) OF SOIL

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	P
1	1	100.0	100.0	.0	30.0	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	325.0	22.0	.00	.0	
1	7	125.0	125.0	325.0	22.0	.00	.0	

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .150 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =-2115.0 PSF

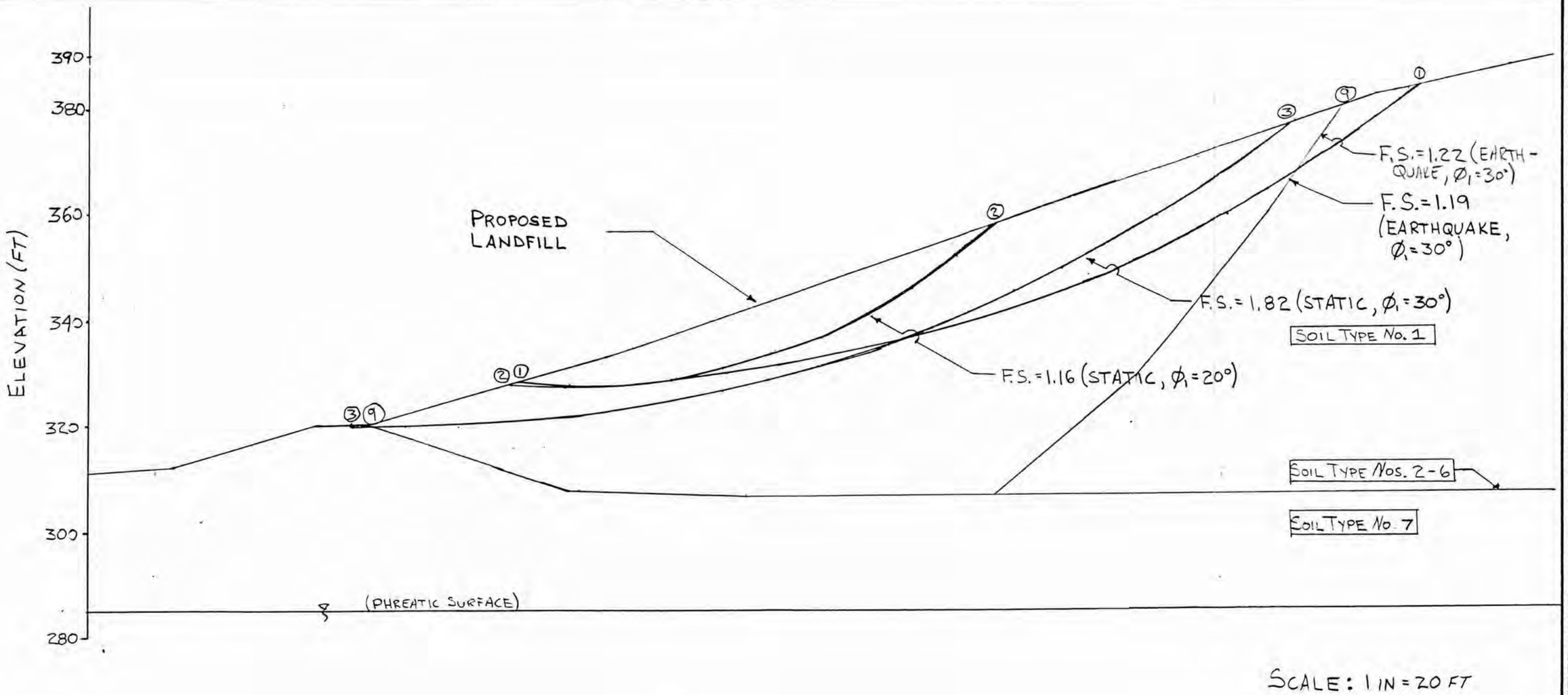
1 TRIAL FAILURE SURFACE SPECIFIED BY 9 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	153.00	40.04
2	191.00	25.02
3	224.00	24.02
4	256.00	24.21
5	277.00	43.50
6	292.00	53.50
7	315.00	72.50
8	325.00	82.50
9	341.00	101.74

******<<<<< CALLED PLTN - STBL 430 >>>>>******
******<<<<< RETURNED FROM PLTN - STBL 430 >>>>>******

FACTOR OF SAFETY FOR THE PRECEDING SPECIFIED SURFACE = 1.118

CASE 9



SCALE: 1 IN = 20 FT

SUBJECT PROTECO SLOPE STABILITY LANDFILL 1

JOB NO 1351

ISOTROPIC SOIL PARAMETERS

CASE9

7 TYPE(S) OF SOIL

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	P
1	1	100.0	100.0	.0	30.0	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	325.0	22.0	.00	.0	
1	7	125.0	125.0	325.0	22.0	.00	.0	

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .150 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =-2115.0 PSF

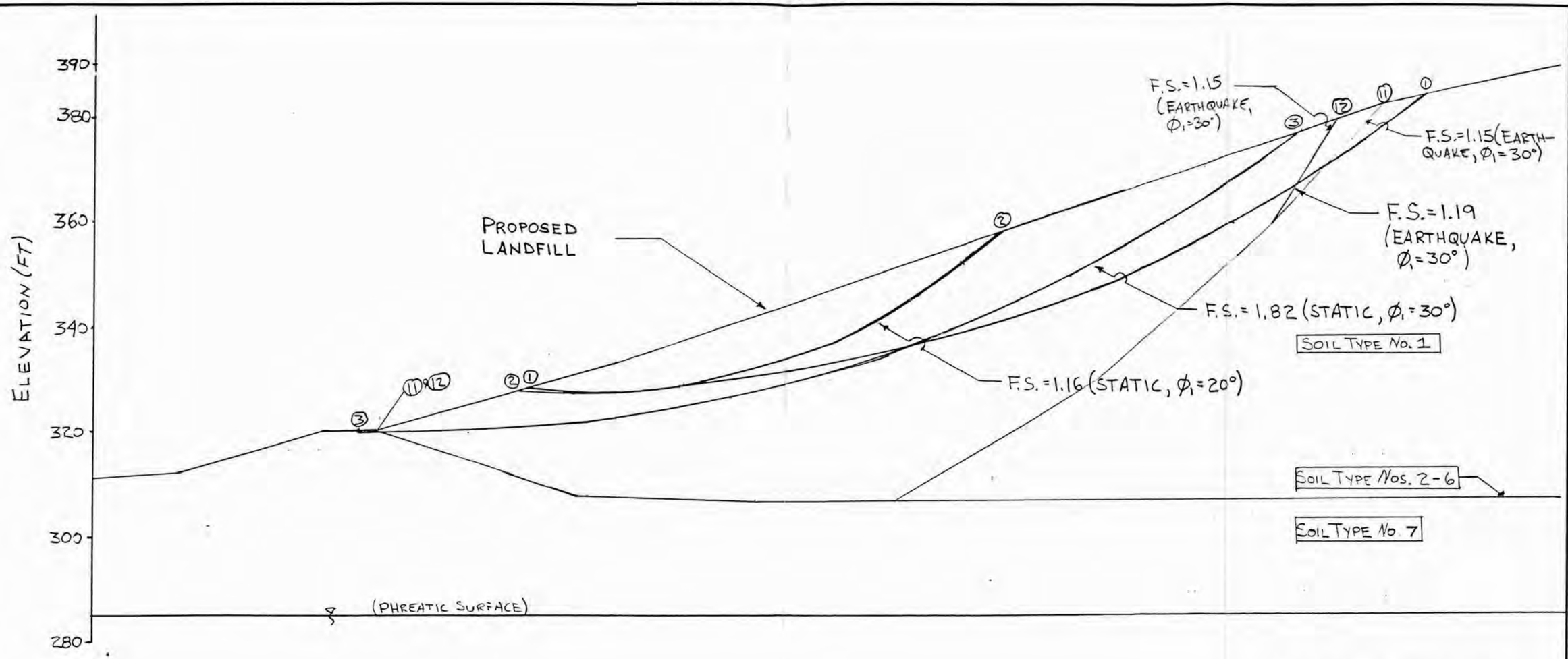
1 TRIAL FAILURE SURFACE SPECIFIED BY 9 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	153.00	40.04
2	191.00	25.02
3	224.00	24.02
4	266.00	24.27
5	277.00	43.50
6	302.00	49.00
7	315.00	72.50
8	325.00	82.50
9	341.00	101.74

******<<<<< CALLED PLTN - STBL 430 >>>>>******
******<<<<< RETURNED FROM PLTN - STBL 430 >>>>>******

FACTOR OF SAFETY FOR THE PRECEDING SPECIFIED SURFACE = 1.220

CASES 10 AND 11



SCALE: 1 IN = 20 FT

SUBJECT PROTECO SLOPE STABILITY
LANDFILL 1
JOB NO. 8551

ISOTROPIC SOIL PARAMETERS

CASE10

7 TYPE(S) OF SOIL

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	P
1	1	100.0	100.0	.0	30.0	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	325.0	22.0	.00	.0	
1	7	125.0	125.0	325.0	22.0	.00	.0	

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .150 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =-2115.0 PSF

1 TRIAL FAILURE SURFACE SPECIFIED BY 9 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	153.00	40.04
2	191.00	25.02
3	224.00	24.02
4	266.00	24.27
5	277.00	43.50
6	292.00	53.50
7	315.00	72.50
8	325.00	82.50
9	345.00	103.10

******<<<<< CALLED PLTN - STBL 430 >>>>>******
******<<<<< RETURNED FROM PLTN - STBL 430 >>>>>******

FACTOR OF SAFETY FOR THE PRECEDING SPECIFIED SURFACE = 1.146

1 ISOTROPIC SOIL PARAMETERS

CASE 11

7 TYPE(S) OF SOIL

32/37

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	P
1	1	100.0	100.0	.0	30.0	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	325.0	22.0	.00	.0	
1	7	125.0	125.0	325.0	22.0	.00	.0	

1 1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .150 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =-2115.0 PSF

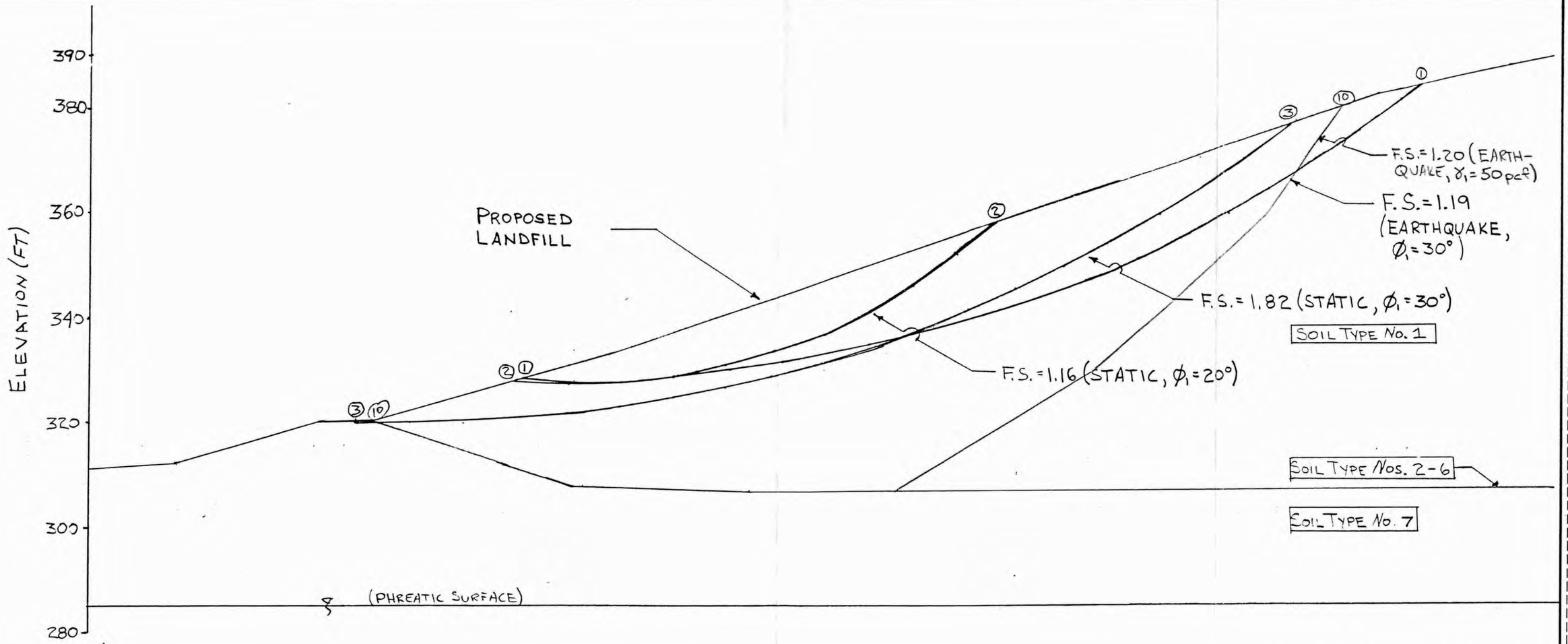
1 TRIAL FAILURE SURFACE SPECIFIED BY 9 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	153.00	40.04
2	191.00	25.02
3	224.00	24.02
4	266.00	24.27
5	277.00	43.50
6	292.00	53.50
7	315.00	72.50
8	325.00	82.50
9	335.00	99.69

******<<<<< CALLED PLTN - STBL 430 >>>>>******
******<<<<< RETURNED FROM PLTN - STBL 430 >>>>>******

FACTOR OF SAFETY FOR THE PRECEDING SPECIFIED SURFACE = 1.153

CASE 12



SCALE: 1 IN = 20 FT

SUBJECT PROTECO SLOPE STABILITY LANDFILL 1

JOB NO. 8511

CASE 12

7 TYPE(S) OF SOIL

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	P
1	1	50.0	50.0	.0	30.0	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	325.0	22.0	.00	.0	
1	7	125.0	125.0	325.0	22.0	.00	.0	

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .150 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =-2115.0 PSF

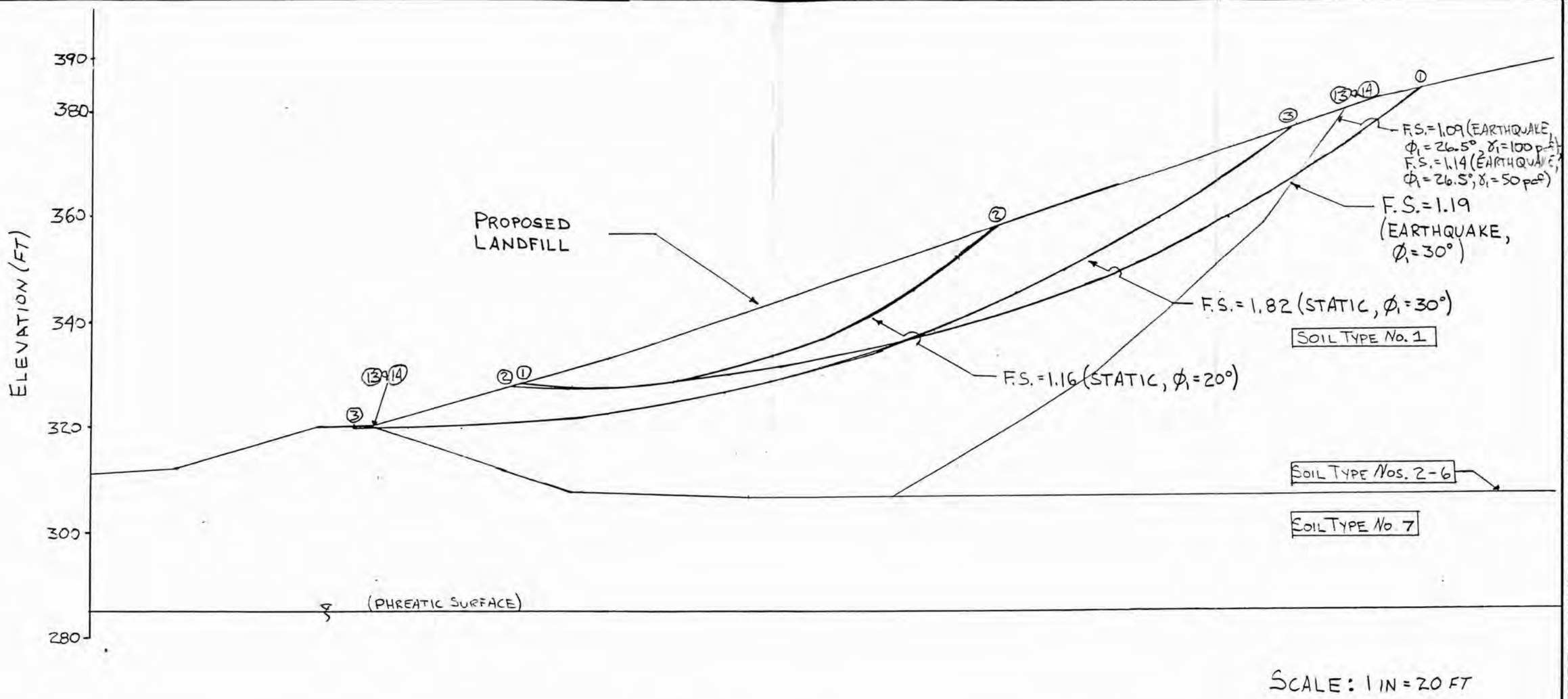
1 TRIAL FAILURE SURFACE SPECIFIED BY 9 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	153.00	40.04
2	191.00	25.02
3	224.00	24.02
4	266.00	24.27
5	277.00	43.50
6	292.00	53.50
7	315.00	72.50
8	325.00	82.50
9	341.00	101.74

**********<<<<< CALLED PLTN - STBL 430 >>>>>*******
**********<<<<< RETURNED FROM PLTN - STBL 430 >>>>>*******

FACTOR OF SAFETY FOR THE PRECEDING SPECIFIED SURFACE = 1.202

CASES 13 AND 14



SCALE: 1 IN = 20 FT

SUBJECT PROTECO SLOPE STABILITY
LANDFILL 1

JOB NO. 8591

ITEM 5

STABILITY ANALYSIS OF CUT SLOPES

PROBLEM STATEMENT

DETERMINE THE FACTOR OF SAFETY FOR CRITICAL CUT SLOPES IN THE NATURAL MATERIAL ABUTTING LANDFILL 1 AND 2.

ASSUMPTIONS

- INSPECTION OF CUTS AROUND LANDFILLS 1 AND 2 SHOWS THAT THE PROBABLE CRITICAL FAILURE SLOPE IS LOCATED AT LANDFILL 2. THE SLOPE IS A 1.5H/1V SLOPE DEFINING THE EAST SIDE OF THE PERIMETER BERM AS SHOWN BY SECTION C-C ON THE FOLLOWING PAGE. (REFERENCE DRAWING NO. BS11E-L16)
- FOR THE PURPOSE OF SIMPLIFYING ANALYSIS, TWO SOIL TYPES ARE USED. THE FOLLOWING EFFECTIVE STRESS PARAMETERS ARE USED:

  SOIL 1 - UNIT WT. OF 125 PCF
  - COHESION OF 2000 PSF
  - FRICTION ANGLE OF 20°

  SOIL 2 - UNIT WT. OF 140 PCF
  - COHESION OF 2000 PSF
  - FRICTION ANGLE OF 35°

  PARAMETERS FOR SOIL 1 ARE APPROXIMATE AND ARE TYPICAL FOR INORGANIC CLAYS OF HIGH PLASTICITY.[1] PARAMETERS FOR SOIL 2 ARE APPROXIMATE. BORING LOGS SHOW THAT SOIL 2 IS ACTUALLY LIMESTONE. (REFERENCE DRAWING NO. BS11E - UNNUMBERED) SOIL 2 PARAMETERS ARE THEREFORE HIGH SINCE POTENTIAL CRITICAL FAILURE SURFACES WILL NOT PASS THROUGH IT.
- LIQUEFACTION POTENTIAL AT THE SITE IS NOT CONSIDERED SINCE THE WATER TABLE IS WELL BELOW THE POTENTIAL CRITICAL FAILURE SURFACE AND BECAUSE CLAY IS NOT SUSCEPTIBLE TO LIQUEFACTION PHENOMENA.

[1] NAVFAC DM-7, TABLE 9-1

TABLE 9-1
Typical Properties of Compacted Materials

Group symbol	Soil type	Range of maximum dry unit weight, pcf	Range of optimum moisture, percent	Typical value of compression: At 1.4 tsf (20 psi)	Typical value of compression: At 3.6 tsf (50 psi)	Typical strength characteristics: Cohesion (as compacted) psf	Typical strength characteristics: Cohesion (saturated) psf	Typical strength characteristics: ϕ (Effective stress envelope) degrees	Typical strength characteristics: Tan ϕ	Typical coefficient of permeability ft/min.	Range of CBR values	Range of subgrade modulus k lb/cu in.
				percent of original height								
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix.	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	.....	.....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	.....	.....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	5×10^{-5}	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	2×10^{-6}	.....	
SC	Clayey sands, poorly graded sand-clay mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	5×10^{-7}	5 - 20	100 - 300
ML	Inorganic silts and clayey silts .	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	10^{-5}	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	5×10^{-7}	.....	
CL	Inorganic clays of low to med. plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	10^{-7}	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	5×10^{-7}	10 or less	50 - 100
* CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	10^{-7}	15 or less	50 - 150
OH	Organic clays and silty clays . . .	65 - 100	45 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	25 - 100

Notes:

1. All properties are for condition of "standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown.
   (....) indicates insufficient data available for an estimate.

* Use $C \approx 2000$ psf $\phi = 20°$

3/3

ANALYSIS

THE SLOPE STABILITY FACTOR OF SAFETY AGAINST SLIDING WAS CALCULATED USING THE 'STABL3' COMPUTER PROGRAM. STABL3 ANALYZED FAILURE SURFACES BY THE SIMPLIFIED JANBU METHOD.

- EARTHQUAKE LOADING
  - STABL3 PLACES A PSEUDO-STATIC LOAD ON THE SOIL BASED ON SEISMIC COEFFICIENTS.
  - USING FIGURE A-3 (SEISMIC ZONE MAP OF THE CONTIGUOUS STATES AND PUERTO RICO)[2], WITH PUERTO RICO IN ZONE 3, A SEISMIC COEFFICIENT OF 0.15 IS SPECIFIED.
  - CAVITATION PRESSURE = -2115 PSF[3] FOR GRANULAR SOILS. THE POTENTIAL CRITICAL FAILURE SURFACE IS IN A COHESIVE SOIL. HOWEVER, CAVITATION PRESSURE IS NOT AS CRITICAL IN COHESIVE SOILS DUE TO THE PORE SIZE. ALSO, THE WATER TABLE IS WELL BELOW THE POTENTIAL CRITICAL FAILURE SURFACE.

RESULTS

POTENTIAL CRITICAL FAILURE SURFACES ARE SHOWN IN FIGURE 1. A SEISMIC COEFF. OF 0.20 WAS USED IN THE ANALYSIS INSTEAD OF 0.15 (CONSERVATIVE). COMPUTER OUTPUT IS GIVEN IN PAGES A1 THROUGH C3.

CONCLUSIONS

THE CALCULATED FACTORS OF SAFETY FOR THE MOST CRITICAL POTENTIAL FAILURE SURFACES ARE ABOVE THE GENERALLY ACCEPTED VALUE OF 1.5 (STATIC) AND 1.2 (EARTHQUAKE).

[2] ENGINEERING DESIGN AND ANALYSIS FOR CORPS OF ENGINEER PROJECTS ER-1110-2-1806 (16 MAY 1983)

[3] COMPUTER SLOPE STABILITY ANALYSIS FOR INDIANA HIGHWAYS, VOLUME 1, JHRP-77-25

BY RCM DATE 1/29/86

CHK'D [illegible] DATE 2/11/86

FRED C. HART ASSOCIATES, INC.

SHEET 5 OF 13 PAGE

SUBJECT PROTECO

SLOPE STABILITY ANALYSIS

JOB NO. B511E

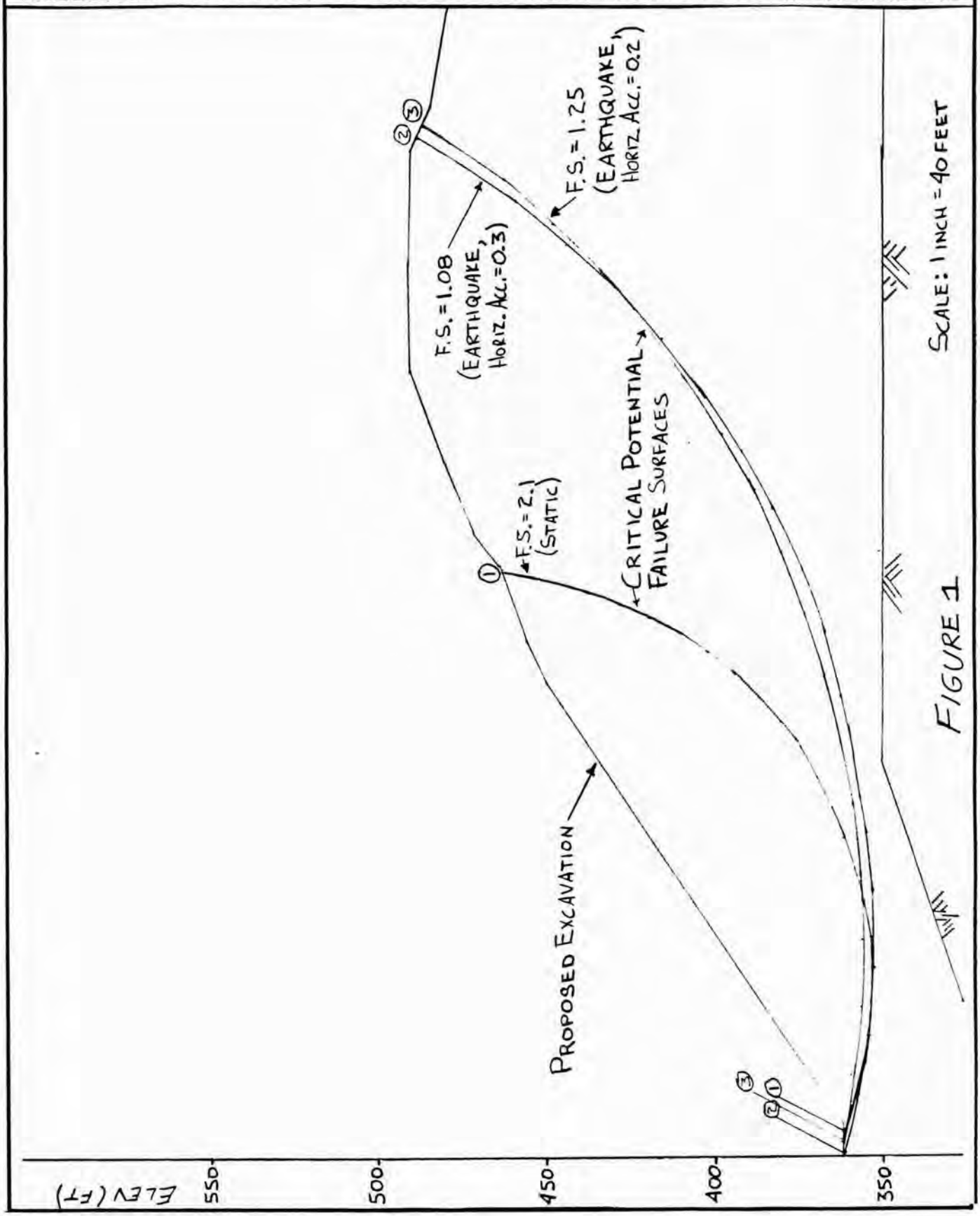


FIGURE 1

480
460
420
1.5H:1V
E407,150
3H:1V
5% GRADE
N66,130
351.5
3H:1V
293
292

CASE 13

7 TYPE(S) OF SOIL

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	P
1	1	100.0	100.0	.0	26.5	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	325.0	22.0	.00	.0	
1	7	125.0	125.0	325.0	22.0	.00	.0	

1 1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .150 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =-2115.0 PSF

1 TRIAL FAILURE SURFACE SPECIFIED BY 9 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	153.00	40.04
2	191.00	25.02
3	224.00	24.02
4	266.00	24.27
5	277.00	43.50
6	292.00	53.50
7	315.00	72.50
8	325.00	82.50
9	341.00	101.74

******<<<<< CALLED PLTN - STBL 430->>>>>******
******<<<<< RETURNED FROM PLTN - STBL 430 >>>>>******

FACTOR OF SAFETY FOR THE PRECEDING SPECIFIED SURFACE = 1.088

ISOTROPIC SOIL PARAMETERS

CASE 14

7 TYPE(S) OF SOIL

ZOMETRIC URFACE NO.	SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PI
1	1	50.0	50.0	.0	26.5	.00	.0	
1	2	120.0	130.0	.0	30.0	.00	.0	
1	3	140.0	140.0	.0	26.0	.00	.0	
1	4	140.0	140.0	.0	18.0	.00	.0	
1	5	140.0	140.0	.0	18.0	.00	.0	
1	6	125.0	125.0	325.0	22.0	.00	.0	
1	7	125.0	125.0	325.0	22.0	.00	.0	

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	5.00
2	543.00	5.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .150 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =-2115.0 PSF

1 TRIAL FAILURE SURFACE SPECIFIED BY 9 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	153.00	40.04
2	191.00	25.02
3	224.00	24.02
4	266.00	24.27
5	277.00	43.50
6	292.00	53.50
7	315.00	72.50
8	325.00	82.50
9	341.00	101.74

******<<<<< CALLED PLTN - STBL 430 >>>>>******
******<<<<< RETURNED FROM PLTN - STBL 430 >>>>>******

FACTOR OF SAFETY FOR THE PRECEDING SPECIFIED SURFACE = 1.141

BOUNDARY COORDINATES

6/13

8 TOP BOUNDARIES
15 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	295.00	50.00	295.00	1
2	50.00	295.00	250.00	362.00	1
3	250.00	362.00	295.00	362.00	1
4	295.00	362.00	427.00	450.00	1
5	427.00	450.00	440.00	456.00	1
6	440.00	456.00	462.00	464.00	1
7	462.00	464.00	472.00	472.00	1
8	472.00	472.00	493.00	480.00	1
9	.00	100.00	52.00	100.00	2
10	52.00	100.00	135.00	200.00	2
11	135.00	200.00	195.00	250.00	2
12	195.00	250.00	256.00	300.00	2
13	256.00	300.00	404.00	350.00	2
14	404.00	350.00	492.00	350.00	2
15	492.00	350.00	493.00	350.00	2

ISOTROPIC SOIL PARAMETERS

2 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIE
1	125.0	125.0	2000.0	20.0	.00	.0	
2	140.0	140.0	2000.0	35.0	.00	.0	

METRIC
URFACE
NO.
1
1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	210.00
2	135.00	210.00

A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

200 TRIAL SURFACES HAVE BEEN GENERATED.

20 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = 200.00 FT

20 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = 280.00 FT.
AND X = 300.00 FT.

A2

7/13

EACH SURFACE TERMINATES BETWEEN X = 430.00 FT.
AND X = 460.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

10.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

RESTRICTIONS HAVE BEEN IMPOSED UPON THE ANGLE OF INITIATION.
THE ANGLE HAS BEEN RESTRICTED BETWEEN THE ANGLES OF -30.0 AND .0 DEG.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL FIRST.

SAFETY FACTORS ARE CALCULATED BY THE MODIFIED BISHOP METHOD.

FAILURE SURFACE SPECIFIED BY 24 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	293.33	362.00
2	302.74	358.61
3	312.40	356.01
4	322.24	354.22
5	332.19	353.24
6	342.19	353.09
7	352.16	353.76
8	362.05	355.25
9	371.78	357.55
10	381.29	360.64
11	390.51	364.51
12	399.39	369.13
13	407.85	374.46
14	415.84	380.47
15	423.31	387.12
16	430.20	394.36
17	436.48	402.15
18	442.09	410.42
19	447.00	419.14
20	451.17	428.22
21	454.59	437.62
22	457.21	447.27
23	459.03	457.10
24	459.64	463.14

*** FS = 2.100 ***

BOUNDARY COORDINATES

13 TOP BOUNDARIES
24 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	295.00	50.00	295.00	1
2	50.00	295.00	250.00	362.00	1
3	250.00	362.00	295.00	362.00	1
4	295.00	362.00	427.00	450.00	1
5	427.00	450.00	440.00	456.00	1
6	440.00	456.00	462.00	464.00	1
7	462.00	464.00	472.00	472.00	1
8	472.00	472.00	493.00	480.00	1
9	493.00	480.00	520.00	491.00	1
10	520.00	491.00	553.00	491.00	1
11	553.00	491.00	586.00	490.00	1
12	586.00	490.00	600.00	485.00	1
13	600.00	485.00	630.00	480.00	1
14	.00	100.00	52.00	100.00	2
15	52.00	100.00	135.00	200.00	2
16	135.00	200.00	195.00	250.00	2
17	195.00	250.00	256.00	300.00	2
18	256.00	300.00	404.00	350.00	2
19	404.00	350.00	492.00	350.00	2
20	492.00	350.00	493.00	350.00	2
21	493.00	350.00	520.00	350.00	2
22	520.00	350.00	553.00	350.00	2
23	553.00	350.00	586.00	350.00	2
24	586.00	350.00	630.00	350.00	2

1 ISOTROPIC SOIL PARAMETERS

2 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	125.0	125.0	2000.0	20.0	.00	.0	1
2	140.0	140.0	2000.0	35.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	210.00
2	135.00	210.00

1	.00	210.00
2	135.00	210.00

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A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .300 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =-2115.0 PSF
1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM
TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

20 SURFACES INITIATE FROM EACH OF 5 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = 265.00 FT.
AND X = 285.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 590.00 FT.
AND X = 620.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

10.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

RESTRICTIONS HAVE BEEN IMPOSED UPON THE ANGLE OF INITIATION.
THE ANGLE HAS BEEN RESTRICTED BETWEEN THE ANGLES OF -30.0 AND .0 D
G.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL
FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL
FIRST.

SAFETY FACTORS ARE CALCULATED BY THE MODIFIED BISHOP METHOD.

FAILURE SURFACE SPECIFIED BY 37 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	285.00	362.00
2	294.71	359.62
3	304.50	357.59
4	314.36	355.92
5	324.28	354.61
6	334.23	353.66
7	344.22	353.08
8	354.21	352.87
9	364.21	353.02
10	374.20	353.53
11	384.16	354.41

FAILURE SURFACE SPECIFIED BY 37 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	285.00	362.00
2	294.71	359.62
3	304.50	357.59
4	314.36	355.92
5	324.28	354.61
6	334.23	353.66
7	344.22	353.08
8	354.21	352.87
9	364.21	353.02
10	374.20	353.53
11	384.16	354.41
12	394.08	355.65
13	403.95	357.25
14	413.76	359.21
15	423.49	361.53
16	433.12	364.20
17	442.66	367.23
18	452.07	370.60
19	461.36	374.30
20	470.50	378.35
21	479.49	382.73
22	488.32	387.43
23	496.97	392.45
24	505.42	397.79
25	513.68	403.43
26	521.73	409.36
27	529.55	415.59
28	537.15	422.10
29	544.50	428.88
30	551.59	435.93
31	558.43	443.23
32	564.99	450.77
33	571.27	458.55
34	577.27	466.55
35	582.97	474.77
36	588.37	483.19
37	591.27	488.12

1

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

11/13

PROBLEM DESCRIPTION BERM

BOUNDARY COORDINATES

13 TOP BOUNDARIES
24 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	295.00	50.00	295.00	1
2	50.00	295.00	250.00	362.00	1
3	250.00	362.00	295.00	362.00	1
4	295.00	362.00	427.00	450.00	1
5	427.00	450.00	440.00	456.00	1
6	440.00	456.00	462.00	464.00	1
7	462.00	464.00	472.00	472.00	1
8	472.00	472.00	493.00	480.00	1
9	493.00	480.00	520.00	491.00	1
10	520.00	491.00	553.00	491.00	1
11	553.00	491.00	586.00	490.00	1
12	586.00	490.00	600.00	485.00	1
13	600.00	485.00	630.00	480.00	1
14	.00	100.00	52.00	100.00	2
15	52.00	100.00	135.00	200.00	2
16	135.00	200.00	195.00	250.00	2
17	195.00	250.00	256.00	300.00	2
18	256.00	300.00	404.00	350.00	2
19	404.00	350.00	492.00	350.00	2
20	492.00	350.00	493.00	350.00	2
21	493.00	350.00	520.00	350.00	2
22	520.00	350.00	553.00	350.00	2
23	553.00	350.00	586.00	350.00	2
24	586.00	350.00	630.00	350.00	2

1 ISOTROPIC SOIL PARAMETERS

2 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	125.0	125.0	2000.0	20.0	.00	.0	1
2	140.0	140.0	2000.0	35.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

1 1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40 12/13

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	210.00
2	135.00	210.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .200 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =-2115.0 PSF
1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM
TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

140 TRIAL SURFACES HAVE BEEN GENERATED.

20 SURFACES INITIATE FROM EACH OF 7 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = 280.00 FT.
AND X = 310.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 570.00 FT.
AND X = 610.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

10.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

RESTRICTIONS HAVE BEEN IMPOSED UPON THE ANGLE OF INITIATION.
THE ANGLE HAS BEEN RESTRICTED BETWEEN THE ANGLES OF -30.0 AND .0 DE
G.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL
FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL
FIRST.

SAFETY FACTORS ARE CALCULATED BY THE MODIFIED BISHOP METHOD.

FAILURE SURFACE SPECIFIED BY 36 COORDINATE POINTS

POINT	X-SURF	Y-SURF

FAILURE SURFACE SPECIFIED BY 36 COORDINATE POINTS

13/13

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	290.00	362.00
2	299.82	360.14
3	309.71	358.61
4	319.64	357.42
5	329.60	356.56
6	339.59	356.05
7	349.58	355.87
8	359.58	356.04
9	369.57	356.54
10	379.53	357.39
11	389.46	358.57
12	399.35	360.09
13	409.17	361.95
14	418.93	364.14
15	428.61	366.66
16	438.20	369.50
17	447.68	372.68
18	457.05	376.17
19	466.30	379.98
20	475.41	384.10
21	484.37	388.53
22	493.18	393.26
23	501.83	398.28
24	510.30	403.60
25	518.58	409.21
26	526.67	415.09
27	534.55	421.24
28	542.22	427.66
29	549.66	434.33
30	556.88	441.26
31	563.85	448.43
32	570.58	455.82
33	577.05	463.45
34	583.26	471.29
35	589.20	479.33
36	594.45	486.98

*** F.S. = 1.254 ***

ITEM 6

STABILITY ANALYSIS OF COMPACTED EARTH DIKES

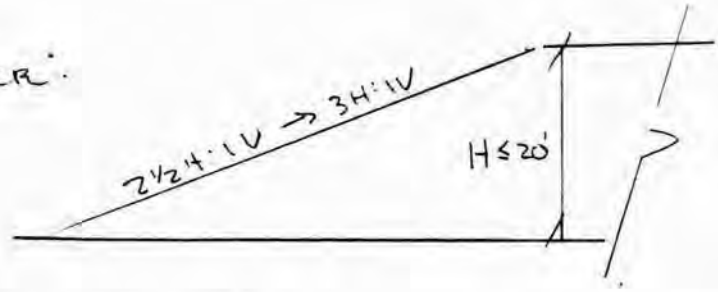
BY BMM DATE 2/3/86	FRED C. HART ASSOCIATES, INC.	SHEET 1 OF 1 PAGE
CHK'D RCM DATE 2/4/86		

SUBJECT STABILITY EVALUATION – PROTECTO JOB NO.
COMPACTED, EARTH DIKE

CHECK STABILITY OF TYPICAL COMPACTED EARTH DIKE (INTERIM BERM) VIA TAYLOR'S CHARTS

REF: FUNDAMENTALS OF SOIL MECHANICS; TAYLOR, 13TH, DEC 1963 p. 459.

TYPICAL DIKE:



FROM AVG. LAB DATA
$\phi = 10^\circ$
$C = 830$ PSF

$\gamma \cong 120$ PCF

$\therefore$ FOR $\iota = \text{ARCTAN}(2\tfrac{1}{2}:1V) \cong 22^\circ$

$\phi = 10^\circ$

$C_d/\gamma H = 0.05$

$\therefore C_d = \gamma(H)(0.05) = 120(20)(0.05) = 120$ PSF

$C_{(AVAIL)} = 830$ PSF

$FS \approx 830/120 \cong 7 \quad \therefore$ OK

$1.01 \times 10^{-5}\ cm^3/s$ per cm of length

$$\frac{cm^3}{s \cdot cm} \cdot \frac{100\ cm}{m} \cdot \frac{m^3}{(100\ cm)^3} \cdot \frac{3600\ s}{hr} \cdot \frac{24\ hr}{day} = \frac{m^3}{day \cdot m}$$

$$= 8.7273 \times 10^{-8}\ \frac{m^3}{day} \text{ per } m$$

~~Time to penetrate~~

$$q = K \frac{dh}{dl} = \frac{10^{-7}\ cm/s\ (609.6)}{2682\ cm} = 1.23 \times 10^{-7}\ cm/s$$

$$\frac{1.23 \times 10^{-7}\ cm}{s} \qquad \frac{2682\ cm}{1.23 \times 10^{-7}\ cm/s} = 11{,}811.4\ s$$

from Schwab p 215+216 soil + water conservation engineering

$$\cancel{q = \frac{4Kh^2}{9L}}$$

$$\cancel{V = \frac{4(10^{-7}\,cm/s)\,20\,ft^2}{9\,L(ft)} = \frac{4\times10^{-7}\,cm/s \cdot 400\,ft^2}{9 \cdot 88\,ft}}$$

~~2.02 ft~~

Seepage through

$$L = (2Z + h - e/2)\cot\alpha + w + .3m$$

$$= \left(0 + 20 - \frac{20}{6}\right)\cot 18.4349 + 20 + .3(60)$$

$$= 50 + 20 + 18 = 88'$$

$$q = \frac{4Kh^2}{9L}$$

$$= \frac{4(10^{-7}\,cm/s)(20\,ft)^2}{9 \cdot 88\,ft} = \frac{4(10^{-7}\,cm/s)(609.6\,cm)^2}{9 \cdot 2682.24\,cm}$$

$$= 1.01\times10^{-8}\,cm^3/s \text{ per cm of length}$$

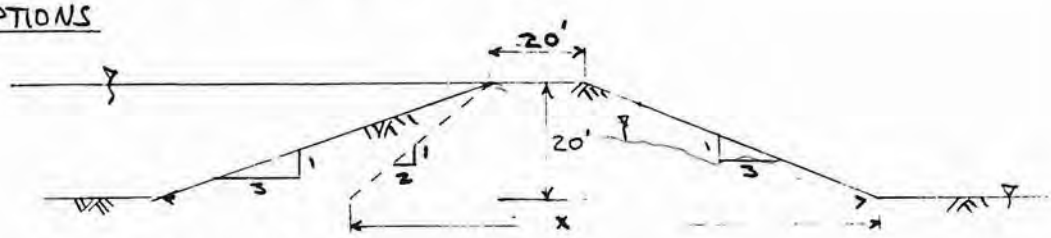
BY RCM DATE 2/4/86	FRED C. HART ASSOCIATES, INC.	SHEET 1 OF 8 PAGE
CHK'D BMM DATE 2/10/86		

SUBJECT PROTECO - GEOTECHNICAL ANALYSIS JOB NO. B511
STABILITY ANALYSIS - EARTH DIKE

PROBLEM STATEMENT

CHECK STABILITY OF DIKE.

ASSUMPTIONS



$\phi = 3.2$ PSF
$C' \; 1150$ PSF } TOTAL STRESS VALUES FROM U-U TEST ON REC. CLAY

$\gamma = 120$ PSF $\quad e = 0.6 \rightarrow n = \frac{e}{1+e} = .375$
$\gamma_{SAT} = 125$ PSF

CONSIDER THE MOST CRITICAL CASE. (SHOWN ABOVE WITH AN APPROXIMATE PHREATIC SURFACE)

SOLUTION

- CALCULATE TIME FOR PHREATIC SURFACE (SHOWN ABOVE) TO DEVELOP.

$T = kt/n_e h$

where k = coeff. of permeability $\approx 1 \times 10^{-7}$ CM/S (CLAY)
$\approx 3.28 \times 10^{-9}$ FT/S $\approx .104$ FT/YR

n_e = effective porosity = .375
$h = 20$ FT $\quad$ 13.7 if lie 6.3' up slope
t = time (YR)
$x = 20 + 3/1(\cancel{20}) + 2/1(\cancel{20}) = 120$ FT (DISTANCE TO TOE)
13.7 $\quad$ ~~= 161~~ 61

$x/h = 120/20 = 6$ $\quad$ $x/h = \cancel{7.4}$ 4.5 $\quad$ $T = \cancel{18}$ 4.0
FIGURE 2 $\rightarrow T = 10.0$

$t = Tn_e h/k$
$t = 10(.375)(20)/.104$ $\quad$ $= \cancel{18}\,4.0(.375)(13.7)/.104$
$t = 721$ YEARS $\quad$ $= \cancel{88.9 \text{ yrs}}$ $= 198$ yrs

SUBJECT PROTECO - GEOTECHNICAL ANALYSIS JOB NO. 8511
STABILITY ANALYSIS - EARTH DIKE

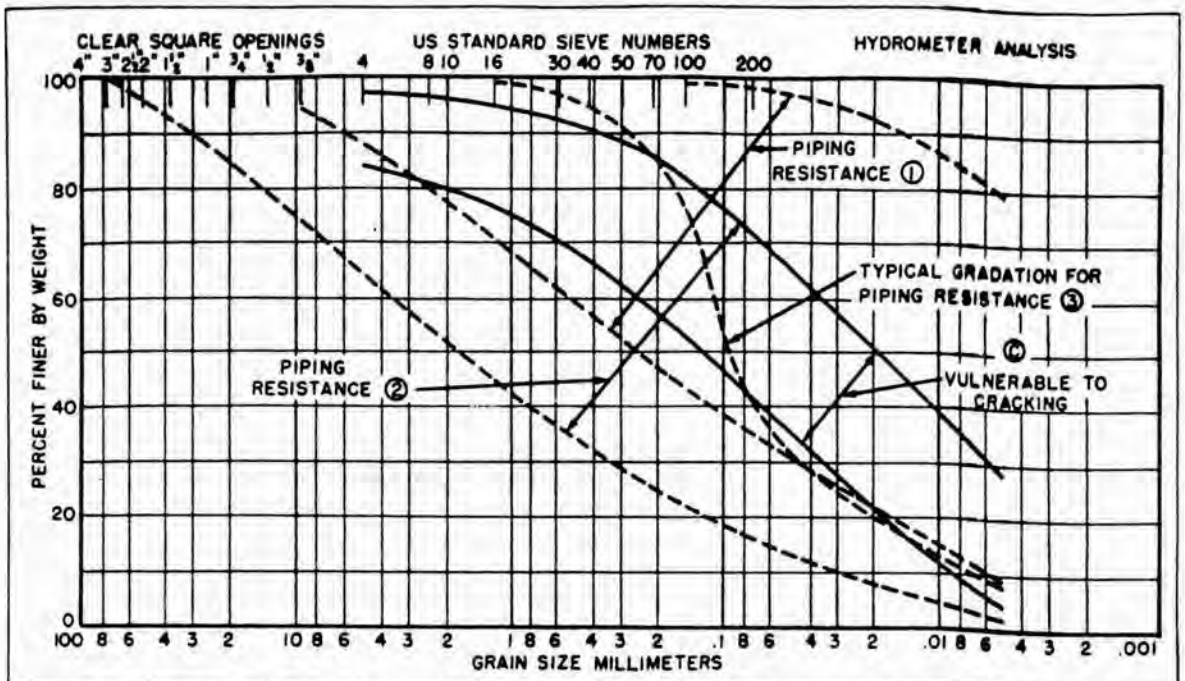
$k = 1 \times 10^{-6}$ cm/s ≈ 1.04 FT/YR	$k = 1 \times 10^{-5}$ cm/s $= 10.35$ FT/YR
$x/h = 6$	$x/h = 6$
FIGURE 2 → $T = 10.0$	FIGURE 2 → $T = 10.0$
$t = T n_e h / k = 7.2$ YEARS	$t = 7.2$ YEARS

- CALCULATE SLOPE STABILITY UNDER EARTHQUAKE CONDITIONS. SEE ATTACHED STABL3 OUTPUT.

CONCLUSIONS

- A FACTOR OF SAFETY OF 1.80 WAS OBTAINED FOR THE POTENTIAL CRITICAL FAILURE SURFACE. THE FACTOR OF SAFETY EXCEEDS THE RECOMMENDED VALUE OF 1.1 AND IS THEREFORE ACCEPTABLE.
- IT WOULD TAKE OVER 700 YEARS FOR THE PHREATIC SURFACE TO REACH THE TOE ON THE OUTSIDE OF THE DIKE WITH $K = 1 \times 10^{-7}$ cm/sec. AN INCREASE IN THE COEFFICIENT OF PERMEABILITY OF THE CLAY TO 1×10^{-5} cm/sec WOULD RESULT IN A TIME OF 7.2 YEARS FOR THE ASSUMED PHREATIC SURFACE TO REACH THE TOE. THUS, LEACHATE PASSING THROUGH THE DIKE WOULD TAKE A CONSIDERABLE AMOUNT OF TIME. NOTE THAT THE DIKE IS LINED WITH HDPE AND THAT MANHOLES ARE PLACED IN THE BERM IN ORDER TO DETECT LEACHATE.

TABLE 9-2
Resistance of Earth Dam Embankment Materials to Piping and Cracking



COBBLES	COARSE	FINE	COARSE	MEDIUM	FINE	SILT OR CLAY (PLASTIC OR NON-PLASTIC)
	GRAVEL		SAND			

Category	Material	Characteristics
Piping resistance:		
①	CL and CH with PI > 15, well graded SC with PI > 15.	Greatest resistance to piping, small and medium concentrated leaks will heal themselves. Embankment may fail as a result of slowly progressive piping caused by leak of about 1/2 cfs.
②	CL and ML with PI < 15, well graded SC and GC with 15 > PI > 7.	Intermediate resistance to piping. Safely resists saturation of lower portion of downstream slope indefinitely. May fail eventually as a result of erosion caused by a small concentrated leak or by progressive sloughing. If a large leak develops, piping causes failure in a short time.
③	SP and uniform SM and ML with PI < 7.	Least resistance to piping. Usually fails in a few years after first reservoir filling if seepage is able to break out on downstream slope. Small concentrated leak on downstream slope can cause failure in a short period of time. High density from compaction increases resistance significantly.
Cracking resistance:		
Ⓐ	CH with D_{50} < 0.02mm and PI > 20.	High postconstruction settlement, particularly if compacted dry. Has sufficient deformability to undergo large shear strains from differential settlement without cracking.
Ⓑ	GC, SC, SM, SP with D_{50} > 0.15mm.	Small postconstruction settlement. Little chance for cracking unless poorly compacted and large settlement is imposed on embankment by consolidation of the foundation.
Ⓒ	CL, ML and SM with PI < 20, 0.15 mm > D_{50} > 0.02 mm.	Medium to high postconstruction settlement and vulnerable to cracking. Should be compacted as wet as possible consistent with strength requirements.

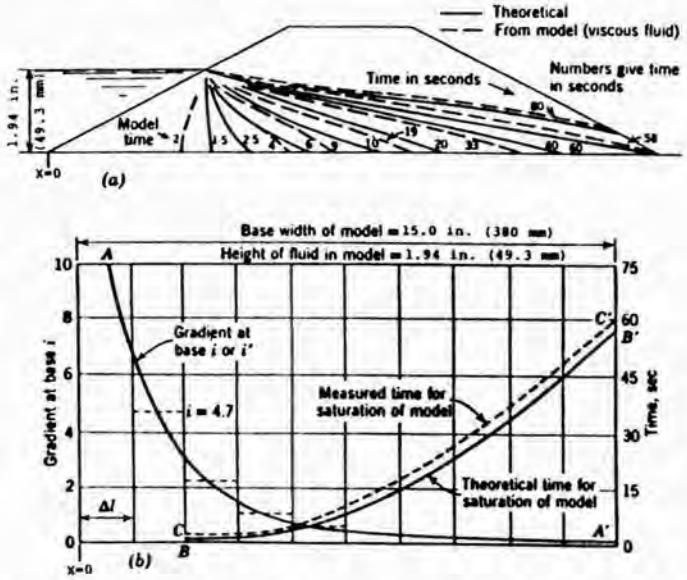


FIG. 1.—Comparison of Phreatic Surfaces between Theory and Model: (*a*) Progress of Phreatic Surface; (*b*) Hydraulic Gradient and Time for Saturation at Base

The time required for a phreatic surface to advance a given distance along the base can be determined by

$$\Delta t = \frac{\Delta l}{v_s} \quad (1)$$

in which Δt = time increment; Δl = incremental distance traveled; and v_s = seepage velocity. According to Darcy's law

$$v_s = \frac{ki}{n_e} \quad (2)$$

in which k = coefficient of permeability; i = hydraulic gradient; and n_e = effective porosity, which includes the air void but not the water that cannot be drained. Substituting Eq. 2 into Eq. 1

$$\Delta t = \frac{n_e \Delta l}{ki} \quad (3)$$

Because the hydraulic gradient is not uniform along the base, the distance x traveled must be divided into a number of equal increments, Δl, as shown in Fig. 1(*b*). For each increment, the average hydraulic gradient is determined from line AA'. The time required to advance a given distance x, as indicated by line BB' can be determined by

$$t = \Sigma \Delta t = \sum \frac{n_e \Delta l}{ki} = \frac{n_e \Delta l}{k} \sum \frac{1}{i} \quad (4)$$

TABLE 1.—Time-Distance Relationship

$x/\Delta l$ (1)	x/h (2)	t (sec) (3)	T (4)
1	0.77	0.22	0.07
2	1.54	0.73	0.24
3	2.31	1.80	0.59
4	3.08	3.96	1.29
5	3.85	8.27	2.70
6	4.62	14.5	4.73
7	5.39	23.3	7.60
8	6.16	34.0	11.1
9	6.93	45.0	14.7
10	7.70	56.9	18.5

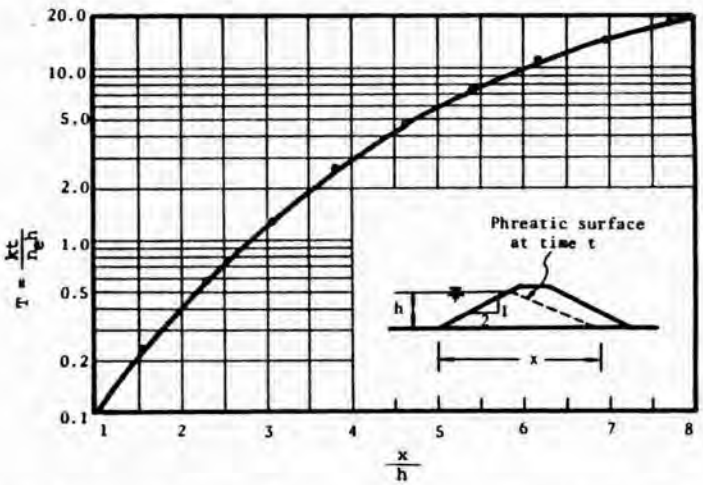


FIG. 2.—Relationship between Dimensionless Time and Dimensionless Distance

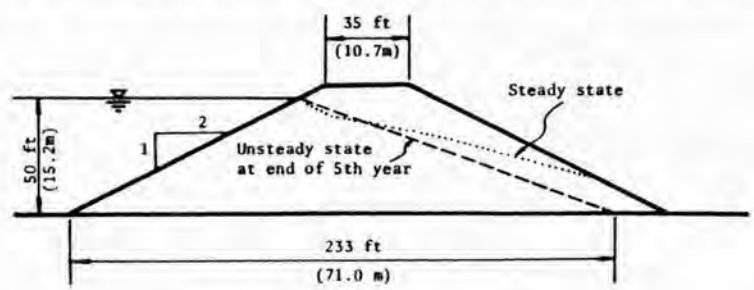


FIG. 3.—Location of Phreatic Surface

in which i = the average hydraulic gradient for each increment.

Line BB′, which is based on a pool depth, h, of 1.94 in. (49.3 mm) and given values of n_e and k, can be applied to other values of h, n_e, and k, as long as the upstream slope is 2:1 (horizontal to vertical). This can be achieved by changing Δl and t to dimensionless terms by converting Eq. 4 to

$$T = \frac{kt}{n_e h} = \frac{\Delta l}{h} \sum \frac{1}{i} \quad (5)$$

in which T = dimensionless time; and $\Delta l/h$ = dimensionless increment. To convert t into T, it is necessary to know k/n_e. The hydraulic gradients at the six intervals of Δl, as measured from line AA′, are 10.7, 4.7, 2.2, 1.1, 0.55, and 0.38, respectively. From Eq. 4 with t = 14.5 sec, as obtained from line BB′, and Δl = 1.5 in. (38 mm), n_e/k = 14.5/[1.5(1/10.7 + 1/4.7 + 1/2.2 + 1/1.1 + 1/0.55 + 1/0.38)] = 1.58 sec/in. (0.062 sec/mm). Due to the difficulty of measuring the time accurately from line BB′, Eq. 4 was used to determine the time when $t < 14.5$ sec. When $t \geq 14.5$, the time was measured from line BB′. The time-distance relationship is shown in Table 1. Note that x/h are obtained from $x/\Delta l$ and T from t by

$$\frac{x}{h} = \frac{\Delta l}{h}\frac{x}{\Delta l} = 0.77\frac{x}{\Delta l} \quad (6)$$

$$T = \frac{kt}{n_e h} = \frac{t}{1.58 \times 1.94} = 0.326t \quad (7)$$

Fig. 2 is a plot of dimensionless time, T, versus dimensionless distance, x/h. At any given time, t, the distance x can be determined from Fig. 2 and the phreatic surface located. It should be pointed out that Fig. 2 is valid only with cases involving an upstream slope of 2:1 and a horizontal impervious base. However, it can be applied to other cases with conservative results, as will be examined later.

Example

Fig. 3 shows a temporary dam with a horizontal impervious base. The soil in the dam has a permeability of 3×10^{-7} ft/sec (2.7×10^{-5} cm/s) and an effective porosity of 0.2. If the dam is used for only five years, determine the location of the unsteady state phreatic surface at the end of the 5th year. Given t = 5 yrs, $k = 3 \times 10^{-7}$ ft/sec = 9.5 ft/yr; n_e = 0.2 and h = 50 ft; $T = 9.5 \times 5/(0.2 \times 50) = 4.75$. From Fig. 2, x/h = 4.66, or $x = 4.66 \times 50 = 233$ ft (71.0 m). The location of the phreatic surface at the end of the 5th year is shown in Fig. 3 by the dashed line.

It will be interesting to see how the design life affects the safety factor of the dam. If the dam is permanent, the phreatic surface will finally become steady state, as indicated by the dotted line. By assuming that the soil has an effective cohesion of 200 psf (9.6 kN/m²), an effective friction angle of 30°, and a mass unit weight of 125 pcf (19.6 kN/m³), the factor of safety obtained by the REAME computer program (2) based

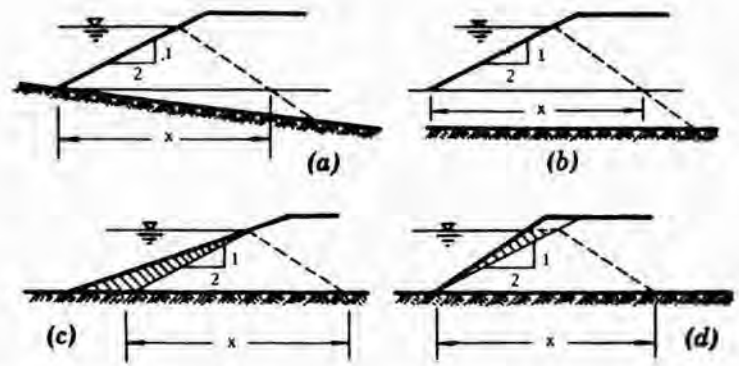


FIG. 4.—Applications to Special Cases: (a) Sloping Base; (b) Soil Foundation; (c) Flatter Slope; (d) Steeper Slope

on the simplified Bishop method and the steady state phreatic surface is only 1.178. If the unsteady state phreatic surface is used, the factor of safety is 1.544.

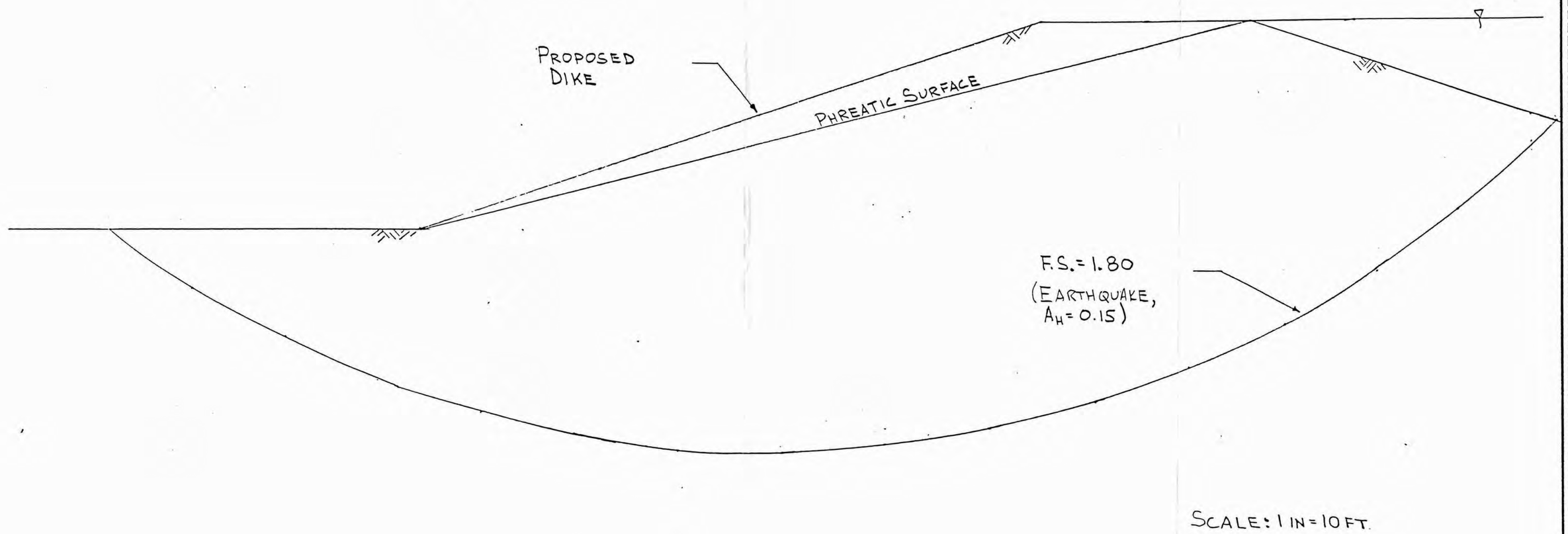
Analysis

The method presented is based on the results obtained by Cedergren for dams with a horizontal impervious base and an upstream slope of 2:1. For dams with other configurations, the procedures shown in Fig. 4 can be used. If the impervious base is not horizontal or the dam is placed on a soil foundation, an imaginary horizontal base is assumed and the method applied as usual. The extra flow region below the imaginary base will lower the phreatic surface, so the assumption of a horizontal base is on the safe side. If the slope is flatter than 2:1, a 2:1 slope is drawn from the pool elevation; if the slope is steeper than 2:1, a 2:1 slope is drawn from the toe. In both cases, the hatched portion of the dam is considered as water instead of soil, and the method is applied as usual. The replacement of soil by water will cause the phreatic surface to rise and is therefore on the safe side.

In the development of Fig. 2, the value of n_e/k used for obtaining line BB′ in Fig. 1 must be determined. Although a great care has been exercised in measuring the hydraulic gradients from line AA′, the value of n_e/k may not be very accurate. However, even if n_e/k or Fig. 1 is not accurate, the procedure for extending Fig. 1 to Fig. 2 is still valid and can be applied once more accurate solutions are obtained.

Summary

A simple method was developed for estimating the unsteady state phreatic surface in earth dams as a function of time. The method is useful in the steady state analysis of temporary earth dams where the steady state phreatic surface may not develop during the life span of the dams. Although the method is based on transient flownets and yields only approximate results, the uncertainty in determining the permeability and


PROPOSED DIKE
PHREATIC SURFACE
F.S. = 1.80
(EARTHQUAKE, A_H = 0.15)
SCALE: 1 IN = 10 FT.

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

7/8

PROBLEM DESCRIPTION BASIN

BOUNDARY COORDINATES

5 TOP BOUNDARIES
5 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	40.00	200.00	40.00	1
2	200.00	40.00	260.00	60.00	1
3	260.00	60.00	280.00	60.00	1
4	280.00	60.00	340.00	40.00	1
5	340.00	40.00	400.00	40.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)
1	120.0	125.0	c = 1150.0	φ = 3.2	.00	.0

ZOMETRIC URFACE NO. 1

1 1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 4 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	40.00
2	200.00	40.00
3	280.00	60.00
4	400.00	60.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .150 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =-2115.0 PSF

CAVITATION PRESSURE [illegible]

1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

8/8

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED ALONG THE GROUND SURFACE BETWEEN X = 170.00 FT.
AND X = 230.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 240.00 FT.
AND X = 310.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

10.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

RESTRICTIONS HAVE BEEN IMPOSED UPON THE ANGLE OF INITIATION. THE ANGLE HAS BEEN RESTRICTED BETWEEN THE ANGLES OF -40.0 AND .0 DE

G.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL FIRST.

SAFETY FACTORS ARE CALCULATED BY THE MODIFIED BISHOP METHOD.

FAILURE SURFACE SPECIFIED BY 17 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	170.00	40.00
2	178.26	34.37
3	187.02	29.53
4	196.19	25.54
5	205.69	22.43
6	215.44	20.22
7	225.36	18.94
8	235.35	18.60
9	245.34	19.19
10	255.22	20.72
11	264.91	23.17
12	274.34	26.51
13	283.40	30.73
14	292.04	35.78
15	300.16	41.62
16	307.69	48.19
17	309.55	50.15

*** F.S. = 1.802 ***

FAILURE SURFACE SPECIFIED BY 16 COORDINATE POINTS

POINT	X-SURF	Y-SURF

ITEM 7

EQUIVALENT GEODRAIN/SAND

TRANSMISSIVITY

BY RCM DATE 2/6/86	FRED C. HART ASSOCIATES, INC.	SHEET 1 OF 2 PAGE
CHK'D BMM DATE 2/10/86		

SUBJECT PROTECO - GEOTECHNICAL ANALYSES JOB NO. B511
TRANSMISSIVITY

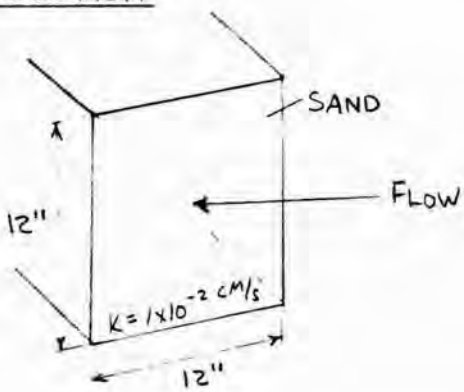
PROBLEM STATEMENT

DETERMINE IF THE TRANSMISSIVITY OF THE GEONET IS EQUIVALENT TO 12 INCHES OF SAND AT 1×10^{-2} CM/SEC

ASSUMPTIONS

- DETERMINE FOR FLATTEST SLOPE (3H:1V)
- ASSUME 100 FT OF OVERBURDEN AT 100 PSF FOR SOIL/WASTE FILL (CONSERVATIVE)
- USE POLY-NET PN-3000

SOLUTION



$$Q = kiA$$

$$Q = 1 \times 10^{-2}\ \text{cm/s}\left(\frac{\text{IN}}{2.54\ \text{cm}}\right)\overset{i}{(.33)}\overbrace{(12)(12)}^{A}$$

$$Q = 0.192\ \text{IN}^3/\text{SEC}/\text{FT WIDTH}$$

LOAD = 100 FT (100 PCF)
LOAD = 10000 PSF

HYDRAULIC GRADIENT, $i = 1/3 = 0.33$

FROM POLYNET TRANSMISSIVITY CHART

$$T_{NET} = 4.8 \times 10^{-4}\ \text{M}^3/\text{S}/\text{M WIDTH}\left(\frac{39.37\ \text{IN}}{\text{M}}\right)^3\left(\frac{\text{M}}{39.37\ \text{IN}}\right)\left(\frac{12\ \text{IN}}{\text{FT}}\right)$$

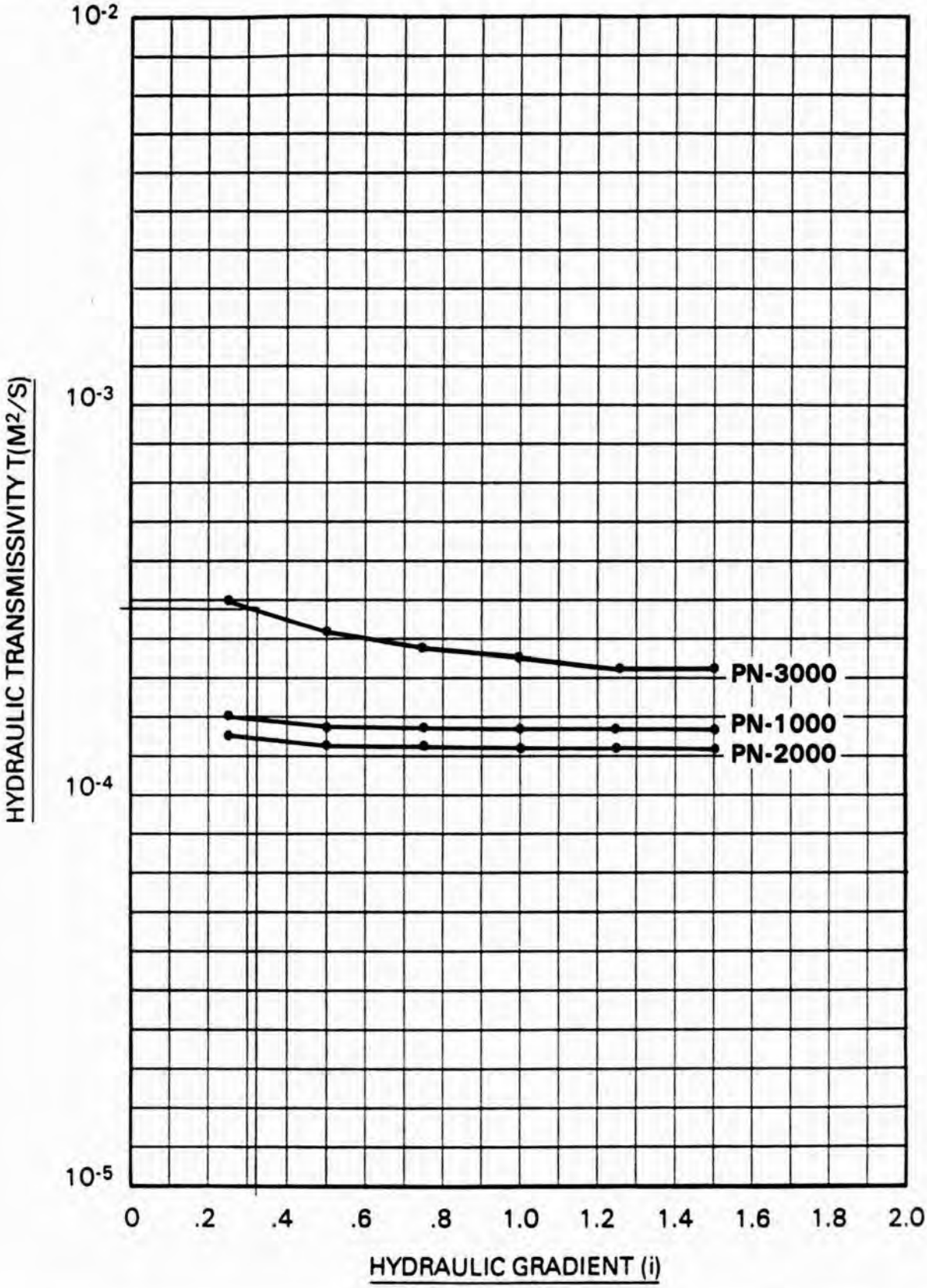
$$T_{NET} = 8.93\ \text{IN}^3/\text{SEC}/\text{FT WIDTH} \gg T_{SAND} = 0.192\ \text{IN}^3/\text{SEC}/\text{FT WIDTH}$$

CONCLUSION

THE TRANSMISSIVITY OF THE GEONET EXCEEDS THE TRANSMISSIVITY OF THE 12 IN SAND LAYER. $T_{NET} = 8.93\ \text{IN}^3/\text{SEC}/\text{FT WIDTH} \gg 0.192\ \text{IN}^3/\text{SEC}/\text{FT WIDTH}$

P. 2 of 2

POLY-NET™ TRANSMISSIVITY	INSTALLATION METHOD CLAY SUBGRADE GEOTEXTILE (TREVIRA 2125) POLY-NET AND 60 MIL LINER	HYDRAULIC PRESSURE 10000 P.S.F.



NSC NATIONAL SEAL COMPANY

600 N. FIRST BANK DRIVE
PALATINE, ILLINOIS 60067
800/323-3820
312/991-6926

ITEM 8

STRESSES ON LEACHATE PIPES

BY RCM DATE 2/6/86	FRED C. HART ASSOCIATES, INC.	SHEET 1 OF 8 PAGE
CHK'D SMM DATE 2/13/86		

SUBJECT PROTECO - GEOTECH[illegible] JOB NO. B511

STRESS [illegible] MENT AND OVERBURDEN

SCHEDULE 80 PVC PIPE

PROBLEM STATEMENT

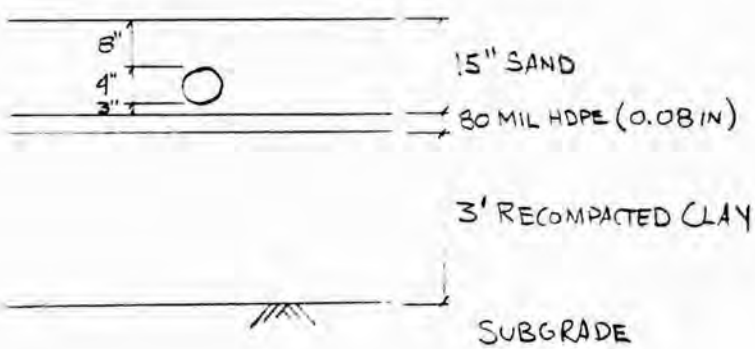
DETERMINE THE STRESSES CAUSED BY EQUIPMENT ON: A) THE HDPE LINER, AND B) THE PVC PIPE.

ASSUMPTIONS AND REFERENCES

- ASSUME A MAXIMUM EQUIPMENT LOAD OF A D8L DOZER (CONSERVATIVE)
- ASSUME THE MINIMUM SAND LAYER THICKNESS OF 15 INCHES.
- ASSUME A 2:1 STRESS DISTRIBUTION.
- USE ELASTIC THEORY FOR SETTLEMENT CALCULATION.
- REF. - CATERPILLAR PEFORMANCE HANDBOOK, 16TH EDITION
- REF. - SOIL MECHANICS, PERLOFF AND BARON, p. 199
- REF. - NIPAKpipe Engineering Manual
- REF. - ESLON PIPING PRODUCTS

SOLUTION

LINER SYSTEM



SUBJECT PROTECO - GEOTECHNICAL ANALYSIS JOB NO. B511
STRESS DUE TO EQUIPMENT AND OVERBURDEN
SCHEDULE 80 PVC LINE

D8L DOZER

WIDTH OF STANDARD TRACK SHOE = 22 IN. = 1.83 FT.

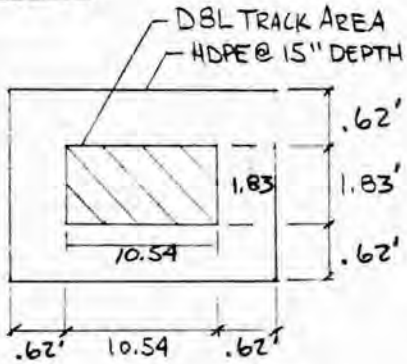
LENGTH OF TRACK ON GROUND = 10'6.5 IN = 126.5 IN. = 10.54 FT.

OPERATING WEIGHT = 82,489 LB.

GROUND CONTACT AREA = 5566 IN^2 (2 TRACKS)

$$\text{GROUND PRESSURE} = \frac{82489}{5566} = 14.82 \text{ PSI}$$ USE 20 PSI

LINER STRESS



$A_{SURFACE} = 22(126.5)/144$

$A_{LINER} = (22+15)(126.5+15)/144$

$\frac{A_{SURFACE}}{A_{LINER}} = 0.532$

$\sigma_{LINER} = 20 \text{ PSI}(0.532)$

$\sigma_{LINER} = 10.64 \text{ PSI}$

SETTLEMENT

$L = 10.54 + 2(.62) = 11.8'$

$B = 1.83 + 2(.62) = 3.1'$

$L/B = 3.81$

$C_s = 1.91$ BY LINEAR INTERPOLATION

(PERLOFF & BARON TEXT, TABLE 5.1 FOR CENTER OF LOADED RECT. AREA ON SURFACE OF ELASTIC HALF SPACE.

SUBJECT PROTECO - GEOTECHNICAL ANALYSIS JOB NO. B511
STRESS DUE TO EQUIPMENT LOAD AND OVERBURDEN
SCHEDULE 80 CMC PIPE

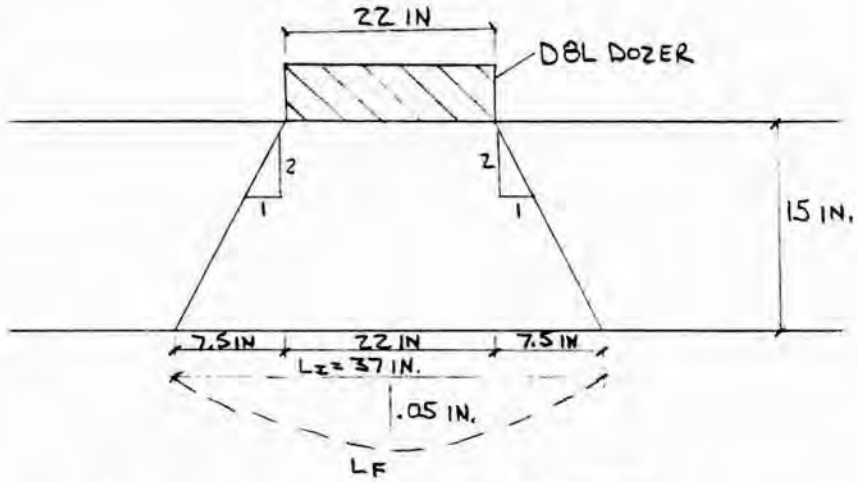
$$\rho_D = C_s q B \left(\frac{1-\mu^2}{E}\right)$$

$$\rho_D = 1.91(10.64 \times 144)\, 3.1 \left(\frac{1-0.5^2}{1500 \times 10^3}\right)$$

$$\rho_D = .0045 \text{ FT}$$

$$\rho_D = .054 \text{ IN}$$

A)



$$L_F = \sqrt{4x^2+y^2} + \frac{y^2}{2x} \ln\left[\frac{2x+\sqrt{4x^2+y^2}}{y}\right]$$

$$x = .05,\ y = 37/2 = 18.5$$

$$L_F = 18.500 + 3422.5 \quad (0.0054)$$

$$L_F = 37 \text{ IN.}$$

$$\Delta L = L_I - L_F \approx 0$$

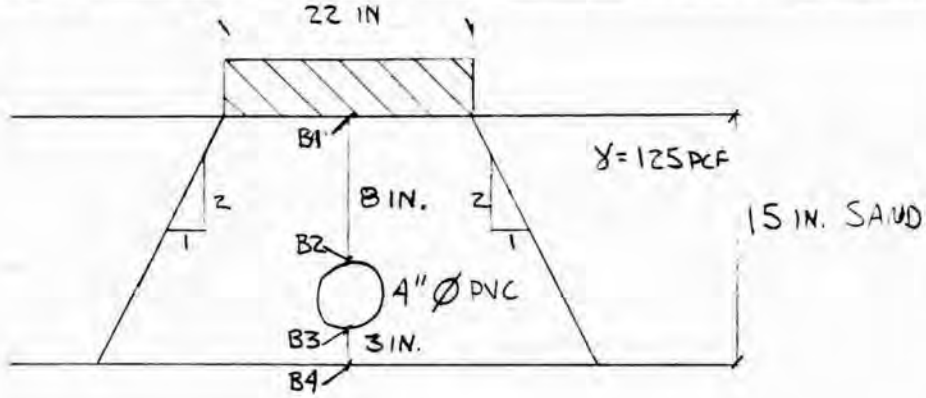
$$\text{STRAIN} = \Delta L / L \approx 0$$

$\therefore$ MINIMAL STRESS INDUCED.

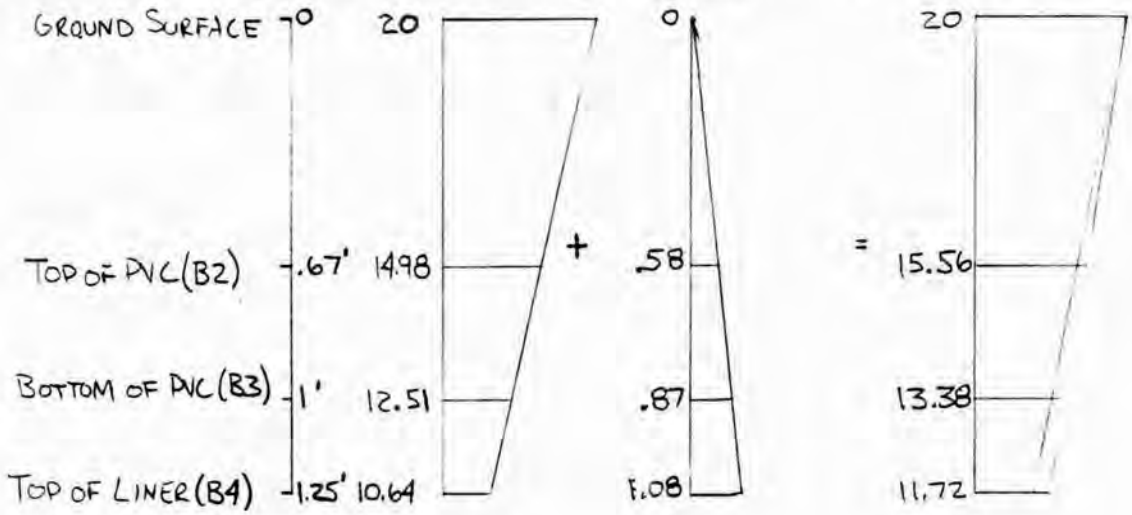
BY RCM DATE 2/6/86	FRED C. HART ASSOCIATES, INC.	SHEET 4 OF 8 PAGE
CHK'D BMM DATE 2/13/86		

SUBJECT PROTECO - GEOTECHNICAL ANALYSIS JOB NO. B511
STRESS DUE TO EQUIPMENT AND OVERBURDEN
SCHEDULE 80 PVC

B)



VERTICAL STRESS (PSI) → DBL DOZER + SOIL = TOTAL STRESS



PIPE DEFLECTION

$$\text{PIPE DEFLECTION}(\%) = \frac{D \times K \times P \times 100}{[2E/3\,(DR-1)^3] + 0.061\,E'} \quad \text{(SPANGLER)}$$

WHERE γ = SOIL DENSITY
E = MODULUS OF ELASTICITY OF PIPE
P = PRISM LOAD = γH
K = BEDDING CONSTANT
E' = MODULUS OF SOIL REACTION
D = DEFLECTION LAG FACTOR = 1.50
DR = DIMENSION RATIO = OUTSIDE DIA./TH

SUBJECT PROTECO - GEOTECHNICAL ANALYSIS JOB NO. 8511
STRESS DUE TO EQUIPMENT, OVERBURDEN
SCHEDULE 80 PVC PIPE

$\gamma = 125$ PCF

$E = 4.2 \times 10^5$ PSI (Schedule 80 PVC from R. Sloane Manufacturing)

$P = 15.56$ PSI (CONSERVATIVE

$K = 0.10$ 0.11 - 0.083 typical, 0.11 is conservative (p.218 WPCF #9)

$E' = 3000$ PSI coarse-grained soil, density > 70% (Bulson text)

$L = 1.50$ for polyethylene (NIPAK)

p. 220 WPCF Manual of Practice #9 suggests...

use $D = 1.50$ and $K = 0.10$

$DR = 4.500/.337 = 13.35$ (R. Sloane)

$$\text{PIPE DEFLECTION} = \frac{1.50(0.10)(15.56)100}{\frac{2(4.2\times10^5)}{3(13.35-1)^3} + .061(3000)}$$

$= 0.704\% < 7\%$ OK

PIPE DEFLECTION IS TOLERABLE FROM A PIPE CRUSHING STANDPOINT.

SUBJECT PROTECO - GEOTECHNICAL ANALYSIS JOB NO. B511
STRESS DUE TO EQUIPMENT AND OVERBURDEN
SCHEDULE 80 PVC PIPE

B) DETERMINE IF 100 FT. OF OVERBURDEN ON TOP OF PIPE IS ACCEPTABLE.

$$P = \gamma H = 125\ PCF\left(100\ FT\right)\left(\frac{FT}{12\ IN}\right)^2 = 86.8\ PSI$$

$$\text{PIPE DEFLECTION} = \frac{1.50(0.10)(86.8)100}{\frac{2(4.2\times10^5)}{3(13.35-1)^3} + .061(3000)}$$

$$= 3.92\% < 7\% \quad \text{OK}$$

PIPE DEFLECTION IS TOLERABLE FROM A PIPE CRUSHING STANDPOINT.

Selection Data – Hi-Strength Schedule 80™ PVC Fittings

Compressed Air or Gases

R&G Sloane strongly recommends against testing assembled PVC piping systems with compressed air or other compressed gases and against using PVC systems for distribution of compressed air or gases.

Physical & Thermal Properties of Polyvinyl Chloride(PVC) Pipe (See note #2 below)

Specific Gravity	1.4
Izod Impact Strength (ft lbs/inch of notch)	0.8
Tensile Modulus, psi	4.2×10^5
Ultimalte Tensile Strength, psi	7200
Working Stress @ 73°F, psi	2000
Working Stress at Upper Temperature limit, (140°F) psi	440
Hazen & Williams "C" factor	150
Coefficient of Linear Expansion 10^{-5} in/in/°F	3.0
Thermal Conductivity BTU/hr/ft²/in/°F	1.10
Upper Temperature Limit	140°F

Flammability — Burns only when in contact with ignition source.

Chemical Resistance of PVC

Weak acids	Resistant
Strong acids	Resistant in most situations
Weak bases	Resistant
Strong bases	Resistant
Solvents	Resist alcohols, aliphiatic hydrocarbons, and oils. Soluble or swell in ketones and esters. Swell in aromatics.
Halogens	Attacked by elemental halogens. Resists water solutions.

Sample Specification for PVC Schedule 80 Pressure Fittings and Pipe

PVC Schedule 80 pressure fittings shall be manufactured by R&G Sloane and to the requirements of ASTM D-2464 (threaded type) and ASTM D-2467 (Solvent cement socket type) except that the socket wall thickness of socket type fittings and the socket wall thickness over the threads of threaded type fittings shall be at least 125% of the wall of the equivalent size of Schedule 80 pipe, and except that the body wall thickness shall be 150% of the wall of the equivalent Scheudle 80 pipe. All internal threads shall be machine tapped to the requirements of ANSI B2.1. The quick burst strength of the fittings shall be equal to or greater than the burst strength of the equivalent Schedule 80 PVC pipe. Shall be made of PVC 12454-B or better.

PRESSURE RATING VS. SERVICE TEMPERATURE PVC SCHEDULE 80 PIPE

Nom. Size	Outside Dia.	Wall	DR = D/t	73°F f = 1 S = 2000 P	80°F f = 0.88 S = 1760 P	90°F f = 0.75 S = 1500 P	100°F f = 0.62 S = 1240 P	110°F f = 0.50 S = 1000 P	120°F f = 0.40 S = 800 P	130°F f = 0.30 S = 600 P	140°F f = 0.22 S = 440 P
⅛	.405	.095	4.623	1230	1082	923	763	615	492	369	271
¼	.540	.119	4.535	1130	994	848	701	565	452	339	249
⅜	.675	.126	5.357	920	810	690	570	460	368	276	202
½	.840	.147	5.714	850	748	638	527	425	340	255	167
¾	1.050	.154	6.818	690	607	518	428	345	276	207	152
1	1.315	.179	7.346	630	554	473	391	315	252	189	139
1¼	1.660	.191	8.691	520	458	390	322	260	208	156	114
1½	1.900	.200	9.500	470	414	353	291	235	188	141	103
2	2.375	.218	10.894	400	352	300	248	200	160	120	88
2½	2.875	.276	10.416	420	370	315	260	210	168	126	92
3	3.500	.300	11.666	370	326	278	229	185	148	111	81
3½	4.000	.318	12.578	350	308	263	217	175	140	105	77
4	4.500	.337	13.353	320	282	240	198	160	128	96	70
5	5.563	.375	14.834	290	255	218	180	145	116	87	64
6	6.625	.432	15.336	280	246	210	174	140	112	84	62

$$P = \frac{2St}{D-t} = \frac{2S}{DR-1} = P_{73°F} f$$

P = Pressure Rating of Pipe at Service Temperature (PSI)

S = Hydrostatic Design Stress (PSI)

D = Outside Diameter Pipe (inches)

t = Pipe Wall Thickness (inches)

f = Derating factor for service temperature

DR = Dimension Ratio = D/t

$P_{73°F}$ = Press. Rating at 73°F

NOTE: 1) Figures for Pressure Rating at 73°F are rounded off from actual calculated values, and are the same as found in Table X2 of ASTM D-1785. Pressure Ratings for other temperatures are calculated from 73° F values.

2) Pressure Rating values are for PVCI pipe, and for most sizes are calculated from the experimentally determined Long Term Strength of PVCI extrusion compounds. Because molding compounds may differ in long term strength and elevated temperature properties from pipe compounds, piping systems consisting of extruded pipe and molded fittings may have lower pressure ratings than those shown here, particularly at the higher temperatures. Caution should be exercised in design of systems operating above 100°F.

COMSTOCK CORPORATION
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Form PVC-1-84 Litho in U.S.A. 11/84

Noting that $e_s = p/E_s$, we get

$$\Delta X = 5pR/E_s, \tag{4.28}$$

which gives a value for ΔX about 25% greater than the approximate equation (4.23).

Returning to equation (4.14), an often-used rearrangement is

$$\Delta X = \frac{\gamma_B \gamma_T W_c}{EI/R^3 + 0.061E'}, \tag{4.29}$$

where $E' = kR$. This was suggested by Watkins and Spangler [4.5], who pointed out that kR was dimensionally correct, and similar to the compressive modulus of elasticity of soil – a pipe–soil interaction modulus in fact. Further, EI/R^3 can be regarded as a 'ring stiffness' factor, because it is the product of the elastic modulus of the pipe wall material and the moment of inertia of unit length of the cross section ($= t^3/12$). Thus,

$$\Delta X = \frac{\text{loading parameter}}{\text{ring stiffness factor} + \text{soil stiffness factor}}. \tag{4.30}$$

Tests by the US Bureau of Reclamation, reported by Howard [4.6], give values for E' in pounds per square inch as in Table 4.2.

These figures were representative of calculations based on measurements in 113 field installations in which initial deflections were measured after construction; pipe deflection was measured between the time of placing the soil to the top of the pipe and completion of backfilling; the horizontal deflection ΔX was measured in most instances. If ΔX was not measured, but the vertical deflection ΔY was known, then it was assumed that $\Delta X = 0.913\Delta Y$, which is the ratio given by equations (4.6) and (4.7). Howard

Table 4.2 Values for E' (psi)

	Relative density (Proctor)			
	Dumped	< 40%	40–70%	> 70%
Fine-grained soils with less than 25% coarse-grained particles	50	200	400	1000
Fine-grained soils with more than 25% coarse-grained particles	100	400	1000	2000
Coarse-grained soils	200	1000	2000	3000
Crushed rock	1000	3000	3000	3000

From Bulson, P.S., Buried Structures, Chapman and Hall, New York, 1985.

ITEM 9

LEACHATE HEAD/DRAIN SYSTEM

EVALUATION

(0655B-10)

SUBJECT PROTECO JOB NO. B511

LEACHATE HEAD/DRAINAGE SYSTEM EVALUATION

APPENDIX D-6.2, ITEM 9

PROBLEM STATEMENT

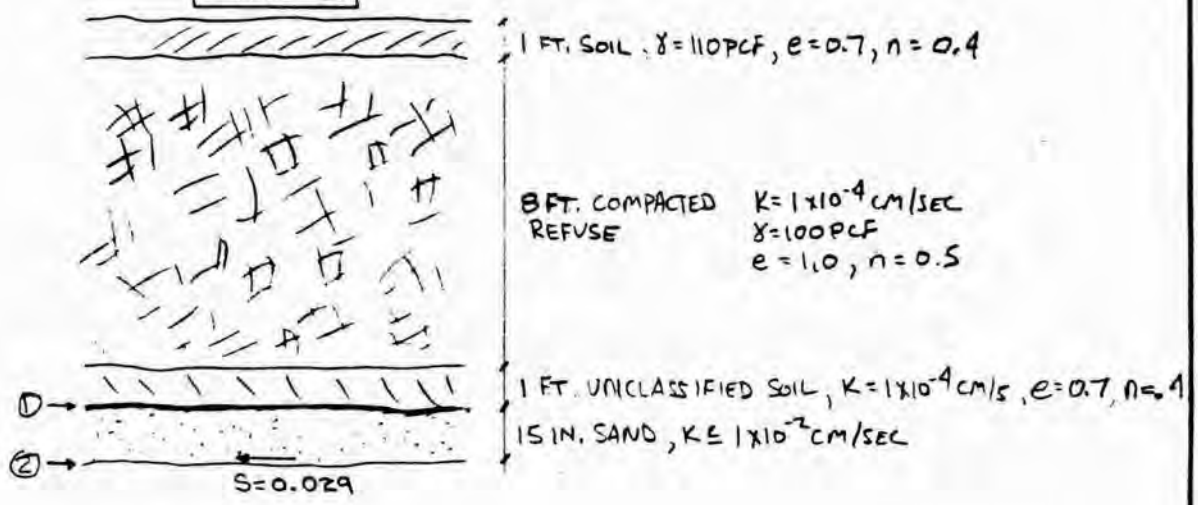
Determine the maximum amount of leachate head on top of the liner.

DESIGN ASSUMPTIONS

Assume the entire area of the cell is covered by one 8-foot lift of refuse with $K = 1 \times 10^{-4}$ cm/sec and 1 foot of soil cover with $k = 1 \times 10^{4}$ cm/sec.

Assuming the cross-section shown ... (see also page 4/10)

CASE 1



1 FILTER CLOTH K > 1 CM/SEC
2 80 MIL HDPE

LEACHATE HEAD GENERATION

Case 1 is the worst case scenario with landfill 1 cell 1 with 1-8 foot lift.

Liner slope = 0.029 (DWG. B511E-L9)

Maximum flow distance to leachate collection pipe = 100 FEET. (DWG B511E-L10)

CASE 2

CASE 2 is an extension of case 1 with the landfill completed and capped.

Refuse height ≈ 395 - 305 = 90 FT. = 1080 IN.

SUBJECT PROTECO JOB NO. B511
LEACHATE HEAD/DRAINAGE SYSTEM EVALUATION
APPENDIX D-6.2, ITEM 9

Assume HELP² data for San Juan, Puerto Rico is appropriate. (CONSERVATIVE)
Use area of 10,000 FT² and multiply areas accordingly.

SOLUTION

① CALCULATE LEACHATE HEAD ON TOP OF LINER, WORST CASE

- WORST CASE 1, CELL 1, LANDFILL 1 WITH ONLY 1 LIFT
- BASIN AREA = 1.59 ACRES = 74650 FT² (CONSERVATIVE)
- THE MAXIMUM HEAD ON THE BASE OF THE LANDFILL IS 9.4 INCHES.

CONCLUSION: THE MAXIMUM HEAD DEVELOPED IS LESS THAN THE MAXIMUM AMOUNT OF HEAD ALLOWED BY THE EPA.

LEACHATE HEAD ON LINER = 9.4 INCHES MAXIMUM (HELP)

LEACHATE HEAD ON LINER ALLOWED = 12 INCHES MAXIMUM (EPA)

9.4 IN < 12 IN ∴ OK

② CALCULATE LEACHATE PRODUCTION, WORST CASE

- WORST CASE IS LANDFILL 1 SINCE IT HAS A GREATER AREA. WORST CASE CELL 1 CAPPED + CELL 2 w/80' NO CAP + CELL 3 w/8'
- LANDFILL 1 CELL 3 WITH 8' LIFT, BASIN AREA = 65570 FT² (CONSERVATIVE)

  $(80.8\ FT^3/10{,}000\ FT^2)\ 65570\ FT^2 = 530\ FT^3/DAY$
- LANDFILL 1 CELL 2 w/80' NO CAP, TOTAL AREA = 114640 FT²

  $(53.9\ FT^3/10000\ FT^2)\ 114640\ FT^2 = 618\ FT^3/DAY$
- LANDFILL 1 CELL 1 CAPPED

  $(0\ FT^3/10000\ FT^2)\ 129930\ FT^2 = 0\ FT^3/DAY$
- TOTAL LEACHATE PRODUCTION = 530 + 618 + 0 = 1148 FT³

  $= 1148\ FT^3 \left(\frac{7.4826\ GAL}{1\ FT^3}\right)$

  = 8589 GAL/DAY

CONCLUSION: THE MAXIMUM AMOUNT OF LEACHATE PRODUCED WILL BE FROM THE WORST CASE GIVEN IN PART ② OR 8600 U.S. GAL./DAY

SUBJECT PROTECO JOB NO. B511
LEACHATE HEAD DRAINAGE SYSTEM EVALUATION
APPENDIX D-62, ITEM a

③ CALCULATE LEACHATE PRODUCTION, ALL CELLS CAPPED AND CLOSED

- LANDFILLS 1 AND 2 AREA - 724,820 FT^2 (CONSERVATIVE)

$(0\ FT^3/10000\ FT^2)\ 724{,}820\ FT^2 = 0\ FT^3 = 0$ GAL (U.S.)

CONCLUSION: THE AMOUNT OF LEACHATE PRODUCED AFTER FINAL CAPPING AND CLOSURE WILL BE 0 GAL. MAXIMUM[3]

REFERENCES

- 'HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE' (HELP) U.S. ARMY ENGINEERS WATERWAYS EXPERIMENT STATION, JUNE 1984
- HART DRWGS. - B511E-L9, L10, AND L15

NOTE

1. LEACHATE VOLUMES GIVEN ARE THE WORST CASE DAY FOR THE PERIOD FROM JAN. 1974 TO DEC. 1978.
2. THE HELP PROGRAM INPUTTED PARAMETERS ARE:
   1) CLIMATOLOGIC DATA
   2) VEGETATIVE COVER DATA
   3) DESIGN AND SOIL DATA
3. THE AMOUNT OF LEACHATE PRODUCED AFTER FINAL CAPPING DECREASES TO ZERO GALLONS OVER AN INDETERMINANT PERIOD OF TIME. NOTE THAT THE EXCESS MOISTURE IN THE LANDFILL DRAINS AND THE WASTE RETURNS TO FIELD CAPACITY.

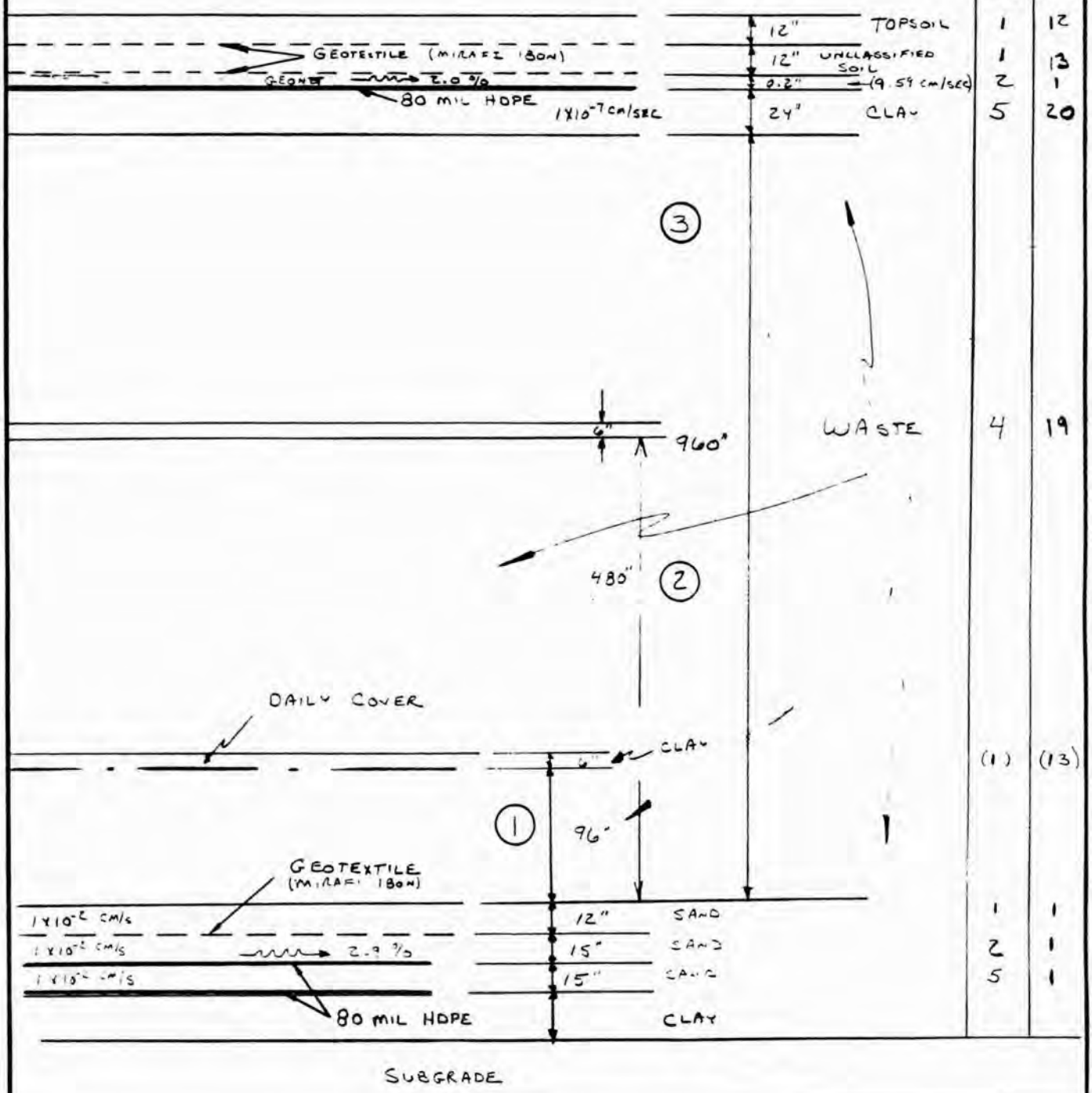
BY HPM DATE 3/7/86	FRED C. HART ASSOCIATES, INC.	SHEET 4 OF 12 PAGE
CHK'D GRB DATE 3/11/86		

SUBJECT PROTECO JOB NO. B511

TYPICAL SUBGRADE AND CROSS-SECTION OF LANDFILL FOR RUNNING OF HELP PROGRAM

INFORMATION FOR COMPUTER RUN:

AREAS = 10,000 FT²



MIRAFI 180N: POROSITY = 300 GAL/MIN/FT² = 0.67 CFS/FT²

k ≅ 20.39 cm/sec ← NON RESTRICTIVE

Penn Center West -II Suite 106
Pittsburgh, PA. 15276

JOB NO. 8511
BY: RCM
DATE: 3/11/86
PAGE NO 5\10

DESCRIPTION: PROTECO
LANDFILLS 1 & 2
AREAS

HORIZONTAL SCALE: 1 INCH = 110 **VERTICAL SCALE:** 1 INCH = 110

AREA IDENTIFICATION		AREA, SQ. IN.	AREA, SQ. FT.	AREA, ACRES
LANDFILL 1	CELL 1	10.74	129931.60	2.98
LANDFILL 1	2	5.47	114644.40	2.63
LANDFILL 1	3	12.34	149296.30	3.43
LANDFILL 2	1	8.38	101377.90	2.33
LANDFILL 2	2	9.10	110149.40	2.53
LANDFILL 2	3	9.87	119418.90	2.74
			Σ=724,818	

FRED C. HART ASSOCIATES.Inc
Penn Center West -II Suite 106
Pittsburgh, PA. 15276

JOB NO.B511
BY:RCM
DATE:3/11/86
PAGE NO 6|10

DESCRIPTION:PROTECO
LANDFILL BASIN AREAS

HORIZONTAL SCALE: 1 INCH = 110 VERTICAL SCALE: 1 INCH = 110

AREA IDENTIFICATION	AREA,SQ.IN.	AREA,SQ.FT.	AREA,ACRES
L1C1	6.17	74648.58	1.71
L1C2	5.68	68763.40	1.58
L1C3	5.42	65572.84	1.51
L2C1	1.63	19669.16	0.45
L2C2	3.11	37589.86	0.86
L2C3	1.62	19568.15	0.45

SUBJECT PROTECO JOB NO. B511
HELP OUTPUT SUMMARY
CAPPED AND CLOSED

CAPPED AND CLOSED CONDITION - VEGETATED
PROTECO, PENUELAS, PUERTO RICO
MARCH 10, 1986

****

PEAK DAILY VALUES FOR 74 THROUGH 78

	(INCHES)	(CU. FT.)
PRECIPITATION	3.87	3225.0
RUNOFF	3.503	2918.9
PERCOLATION FROM BASE OF COVER	0.0	0.0
PERCOLATION FROM BASE OF LANDFILL	0.0	0.0
DRAINAGE FROM BASE OF COVER	0.027	22.6
DRAINAGE FROM BASE OF LANDFILL	0.0	0.0
HEAD ON BASE OF COVER	24.1	
HEAD ON BASE OF LANDFILL	0.0	
SNOW WATER	0.0	0.0

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.5350

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.2220

****
****

SUBJECT PROTECO JOB NO. B511
HELP OUTPUT SUMMARY
80' WASTE NO CAP

END OF PHASE CONSTRUCTION - 80 FOOT HEIG
PROTECO, PENUELAS, PUERTO RICO
MARCH 11, 1986

****

PEAK DAILY VALUES FOR 74 THROUGH 78

	(INCHES)	(CU. FT.)
PRECIPITATION	3.87	3225.0
RUNOFF	3.118	2598.5
PERCOLATION FROM BASE OF LANDFILL	0.0	0.0
DRAINAGE FROM BASE OF LANDFILL	0.065	53.9
HEAD ON BASE OF LANDFILL	6.1	
SNOW WATER	0.0	0.0

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.3910

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1964

****
****

SUBJECT PROTECO JOB NO. 8511
HELP OUTPUT SUMMARY
40' OF WASTE

MID-PHASE CONSTRUCTION - 40 FT. OF WASTE
PROTECO, PENUELAS, PUERTO RICO
MARCH 10, 1986

****

PEAK DAILY VALUES FOR 74 THROUGH 78

	(INCHES)	(CU. FT.)
PRECIPITATION	3.87	3225.0
RUNOFF	3.116	2596.5
PERCOLATION FROM BASE OF LANDFILL	0.0	0.0
DRAINAGE FROM BASE OF LANDFILL	0.081	67.1
HEAD ON BASE OF LANDFILL	7.6	
SNOW WATER	0.0	0.0

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.3910

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1964

****
****

SUBJECT PROTECO JOB NO. B511
HELP OUTPUT SUMMARY
8' LIFT

INITIAL 8 FOOT LIFT FOR CELL LINER AREA
PROTECO, PENUELAS, PUERTO RICO
MARCH 10, 1986

****

PEAK DAILY VALUES FOR 74 THROUGH 78

	(INCHES)	(CU. FT.)
PRECIPITATION	3.87	3225.0
RUNOFF	3.115	2595.6
PERCOLATION FROM BASE OF LANDFILL	0.0	0.0
DRAINAGE FROM BASE OF LANDFILL	0.097	80.8
HEAD ON BASE OF LANDFILL	9.4	
SNOW WATER	0.0	0.0

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.3910

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1964

****
****

ITEM 10 [1]

SOILS LABORATORY TESTING DATA

[1] This item was added to Revision 2 of this Application, June 1986

E[illegible] Briggs
D[illegible] ASSIGNED 6/28/85
DATE DUE

JOB No. -5.1
JOB NAME [illegible] I
Puerto Rico

DATE REC.
DATE CMP.
REC BY
Page No

SUMMARY OF LABORATORY TEST RESULTS

ORING and MPLE No	DEPTH feet	CLASSIFICATION	SPECIAL TESTS	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		UNCON COMPRESS		UNIT DRY WGT (pcf)	SPECIFIC GRAVITY	GRAIN SIZE		OPT MOIST	CONSOLID	TRIAXIAL			
					LIQUID LIMIT	PLASTIC LIMIT	STRESS (tsf)	STRAIN (%)			SIEVE	HYDR.			UU	$\overline{CIU}$	CELL PRESSURE (psi)	BACK PRESSURE (psi)
–	–	Bucket Sample #1 Clay Borrow	/ Proctor		/	/												
–	–	Bucket Sample #2 Clay Borrow	/ Proctor		/	/												
–	–	Bucket Sample #3 Clay Borrow	/ Proctor		/	/												
–	–	Shelby Tube #1	/ Tube Log		/	/												
–	–	Shelby Tube #2	/ Tube Log		/	/												

See Test Curves

V TEST COMP and CHECKED

● TEST IN PROGRESS

Page 0 of 47

MOISTURE-DENSITY RELATIONSHIP TEST

Page 1 of

TEST No.: _____ DATE : _____ TEST LOCATION: N _____ E _____ ELEVATION: _____

Sample	1	2	3
$\gamma_{d\,max}$	105.0	101.5	97.5
Opt m/c	18.0	19.5	20.0
LL	48	61	69
Sample	252	151	253

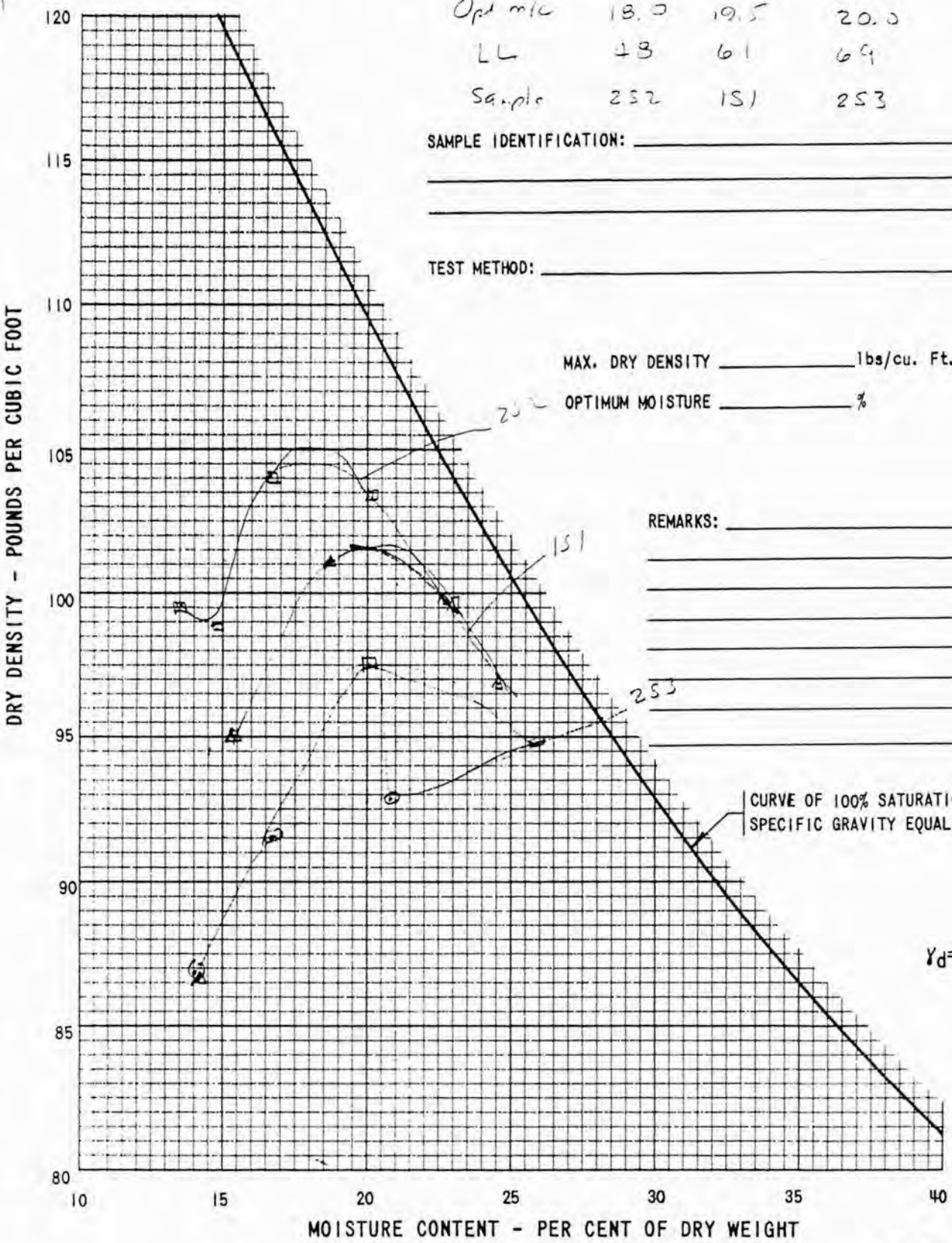
SAMPLE IDENTIFICATION: _____

TEST METHOD: _____

MAX. DRY DENSITY _____ lbs/cu. Ft.

OPTIMUM MOISTURE _____ %

REMARKS: _____



$$\gamma_d = \frac{G\gamma_w}{1 + \frac{wG}{S}}$$

RE FORM 107(6-73)

MOISTURE-DENSITY RELATIONSHIP TEST

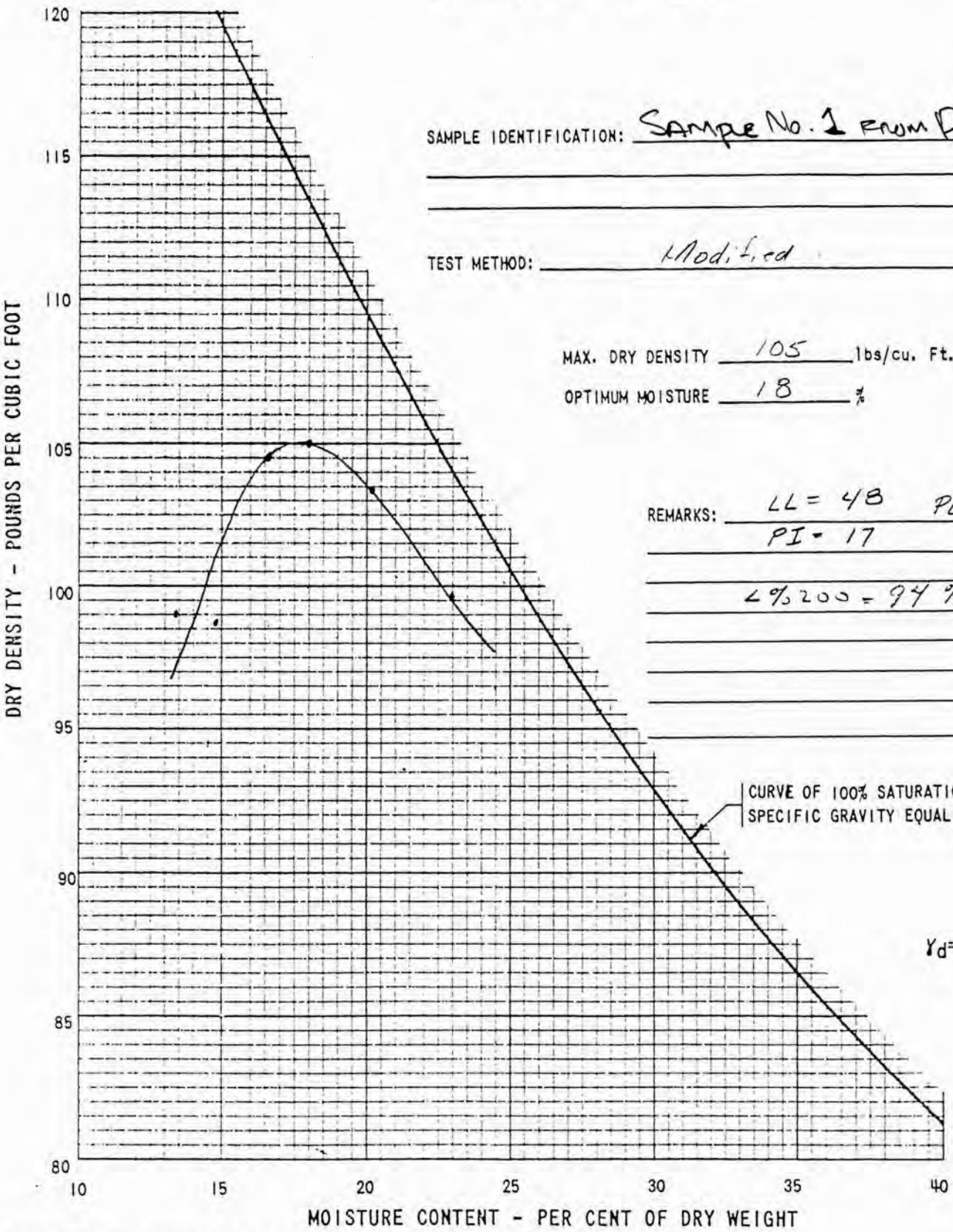
Page 2

Job Name Fred C Hart Job No 85/3071-01 Date

Boring No Sample No 252 Depth Preparation

Material

Data from Carlos Sierra
13 MAY 85 @ 430 PM



SAMPLE IDENTIFICATION: Sample No. 1 from Page 1

TEST METHOD: Modified

MAX. DRY DENSITY 105 lbs/cu. Ft.

OPTIMUM MOISTURE 18 %

REMARKS: LL = 48 PL = 31
PI = 17
< % 200 = 94 %

$$\gamma_d = \frac{G\gamma_w}{1 + \frac{wG}{S}}$$

J & L TESTING COMPANY

Check Test to locate Sample #

ATTERBERG LIMITS DETERMINATION B511

Project HART Job No.

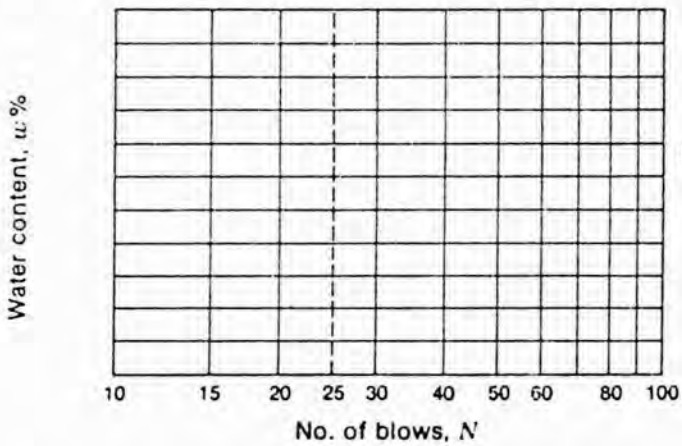
Location of Project SCI Boring No. 252 Sample No.

Description of Soil

Depth of Sample — Tested By GJ Date 6/26/85

Liquid Limit Determination

Can no.	A717					
Wt. of wet soil + can	[illegible]					
Wt. of dry soil + can	12.22					
Wt. of can	4.28					
Wt. of dry soil	7.94					
Wt. of moisture	3.98					
Water content, w%	50.1					
No. of blows, N	50					



Flow index F_f =

Liquid limit = 51

Plastic limit = 28

Plasticity index I_P = 23

Plastic Limit Determination

Can no.	A749			
Wt. of wet soil + can	17.54			
Wt. of dry soil + can	14.64			
Wt. of can	4.31			
Wt. of dry soil	10.33			
Wt. of moisture	2.90			
Water content, w% = w_P	28			

Page 4
Sheet 1 of ___
B-511

TRIAXIAL PERMEABILITY TEST DATA

Project Fred C. Hart Assoc. Job No. 85B078-01

Location SCI- P.R. Boring No. IM2-52 Sample No. TES P-

Description of Soil Borrow For Liner Depth of Sample Bucket 11.2

Tested By JBG. Date of Testing ___

Undisturbed ___ / Compacted ✓ Method ___

Desired Density (pcf) ___ M/C % ___

Diameter (in.) ___ Height (in.) ___

COMPACTION DATA

No. Layers ___ Wet Weight (grams) ___ per layer

Specific Gravity, (G_s) ___ Assumed Yes ✓ No ___

Cell No. 1 Desired "B" parameter B=1

Test Pressures:

Cell 50 psi Back 40/32 psi Head 8 psi

Saturation Increments:

Cell ___ psi Top ___ psi Bottom ___ psi

Deaired Water used for Saturation: Yes ✓ No ___

Other Liquid used for Saturation: TAP

Stones Saturated Yes ✓ No ___

Lines Saturated Yes ✓ No ___

Notes: ___

TRIAXIAL PERMEABILITY TEST DATA (cont.)

UNDISTURBED SAMPLE DATA

Initial Data

Height _____ in. (H_o)

Diameter _____ in. (D_o)

Area _____ sq. in. (A_o)

Volume _____ cu. in. (V_o)

Weight _____ g. (W_o)

Wet Density _____ pcf (γ_{wo})

Water content determination (TRIMMINGS)	
Wt. of can + wet soil =	[illegible]
Wt. of can + dry soil =	[illegible]
Wt. of can =	[illegible]
Wt. of water =	100.6
Wt. of dry soil =	[illegible]
Initial water content w_i =	[illegible]

COMPACTED SAMPLE DATA

Total Weight of Soil for Compaction [illegible] g (wet)

Initial Moisture Content 31.2 %

No. of Layers 4 Weight per Layer _____ g (wet)

Height 3.71 in. (H_o)

Diameter 2.50 in. (D_o)

Area 6.15752 sq. in. (A_o)

Volume [illegible] cu. in. (V_o)

Wet Density 121.2 pcf (γ_{wo})

(γ_d = 92.4 pcf)

Initial Height of Soil, H_i _____ Computed Dry Weight of Soil, W_s' = _____

Weight of Wet Soil, W_t = _____ Oven Dry Weight of Soil, W_s = _____

Computed Height of Solids, $H_o = W_s / G_s A_o$ = _____

Initial Height of Voids, $H_v = H_i - H_o$ = _____

Initial Degree of Saturation, $S_i = (W_t - W_s)/(H_i - H_o) A_o$ = _____

Initial Void Ratio, $e_o = H_v / H_o$ = _____

TRIAXIAL PERMEABILITY TEST DATA (cont.)

CHAMBER DATA - "SATURATION"

DATE	TIME	CELL PSI	TOP PSI	BOTTOM PSI	'B'	H(DIAL)	ΔH in.	A CORRECT	OTHER

Notes: _____

See Test Plot

TEST PARAMETERS

COMPUTED

Height _____ in. (H_f):(L)

Area _____ sq. in. (A_f):(A)

Computed Wet Weight _____ g

CONFINEMENT / HEAD

Cell Pressure _____ psi (C)

Top Pressure _____ psi (TS)

Bottom Pressure _____ psi (BS)

Effective Head _____ psi (h)

Test data — *Test data used*

Test No	t, sec	Q, cu cm	T, °C	Test No	t, sec	Q, cu cm	T, °C
1							
2							
3							
4							
				Average ᵃ			

$k_T = QL/Aht =$ _____

$\eta_T/\eta_{20} =$ _____

$=$ _____ cm/sec

$k_{20} = k_T\eta_T/\eta_{20} =$ _____ cm/sec

TRIAXIAL PERMEABILITY TEST DATA (cont.)

Notes: _____

Initial Dial Reading _____

Final Dial Reading _____

Change in Sample Height _____

Final Height of Voids, H_{vf} _____

Final Void Ratio, $e_f = H_{vf}/H_o$ _____

DISASSEMBLED CELL (Measured Dimensions)

Height 3.69 in. (H_f)

Diameter 2.80 in. (D_f)

Area 6.1575 sq. in. (A_f)

Volume [illegible] cu. in. (V_f)

Wet Weight [illegible] g. (W_f)

Wet Density 123.1 pcf (γ_f)

Dry Density 92.6 pcf (γ_{df})

FINAL MOISTURE CONTENT

Final Wet Weight + Tare [illegible] g

Final Dry Weight + Tare 562.4 g

Weight of Water 181.8 g

Tare Weight [illegible] 9.4 g

Oven Dry Weight of Soil, W_s 553.0 g

Final Water Content, W_f 32.9 %

Final Degree of Saturation, S _____ %

AFTER

FOR COMPACTED SAMPLES

Maximum Density (Dry) 105 pcf

Optimum M/C 18 %

Percent Compaction 88 %

Before

83%

DATA SHEET

TRIAXIAL PERMEABILITY TEST DATA

Job No: _____

Sample No: _____

Sheet No: _____

Description: _____

DATE	TIME	CELL PRESS	BURRETT	DELTA CELL BUR	TOP PRESS.	BURRETT	DELTA TOP BURR	BOT PRESS.	BURRETT	DELTA BOT BURR
21 MAY	2123	25	5.8		17	13.-		20	6.-	
	2200	25	5.6		17	18.6		2[illegible]	6.6	
	2211	2[illegible]	5.6		17	18.5		2[illegible]	6.6	
22 MAY	0534	2[illegible]	5.8		2[illegible]	18.5			6.6	
	0500	35	6.5		27	13.6		3[illegible]	6.7	
	1803	3[illegible]	6.5		27	18.5		3[illegible]	7.4	
	2104	35	6.6		27	13.[illegible]		3[illegible]	7.5	
	21[illegible]	3[illegible]	7.0		32	19.[illegible]		4[illegible]	7.5	
	[illegible]	4[illegible]	7.[illegible]		[illegible]	15.[illegible]		[illegible]	7.[illegible]	
23 MAY	0511	4[illegible]	7.2		32	17.4		[illegible]	8.1	
	1735	[illegible]	7.3		32	22.1		[illegible]	3.8	
	1836	4[illegible]	7.[illegible]	60	[illegible]	21.5	[illegible]	[illegible]	4.1	0.[illegible]
	2003	4[illegible]	7.3	152	32	3[illegible]	0.7	[illegible]	4.5	0.7
	21[illegible]	4[illegible]	7.3	209	[illegible]	[illegible]	1.0	[illegible]	[illegible]	1.0
	2230	4[illegible]	7.3	299	32	20.7	1.4	4[illegible]	5.3	1.5
24 MAY	0326	4[illegible]	7.[illegible]	713	32	15.[illegible]	3.4	[illegible]	7.[illegible]	3.5
	1800	[illegible]	7.3	1511	32	15.[illegible]	7.0	[illegible]	11.0	7.2

BY _____ DATE _____ | J & L TESTING Co | SHEET ___ OF ___ PAGE

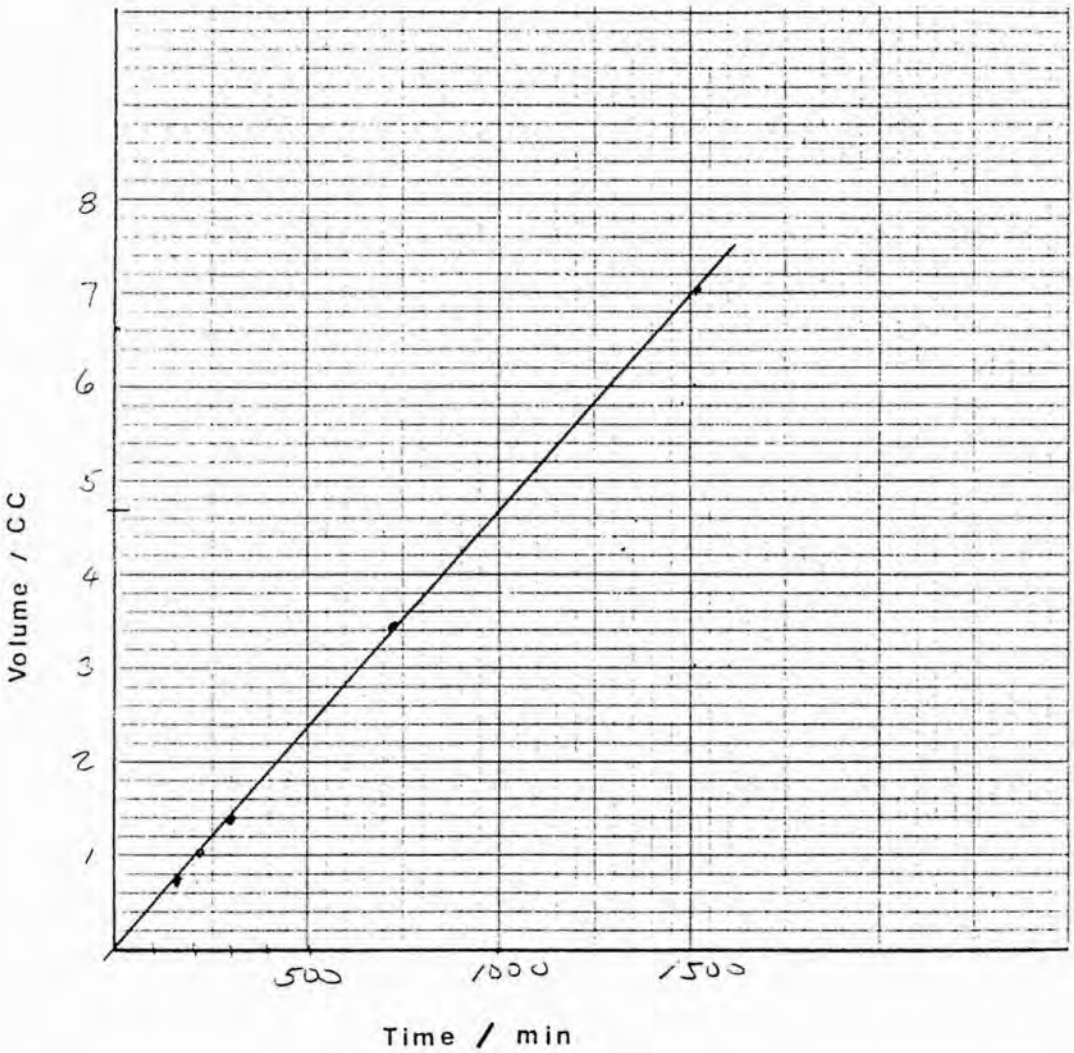
CHK'D _____ DATE _____

SUBJECT _____ JOB NO. _____

FLOW vs TIME

$\Delta V = 2$

$K = \frac{QL}{Aht}$



Q = 47 cc

L = 3.69 in.

A = 0.0027604 sq ft.

h = 8 cm

t = 1000 min = 60,000 sec

K = 3.28×10^{-8} cm/sec.

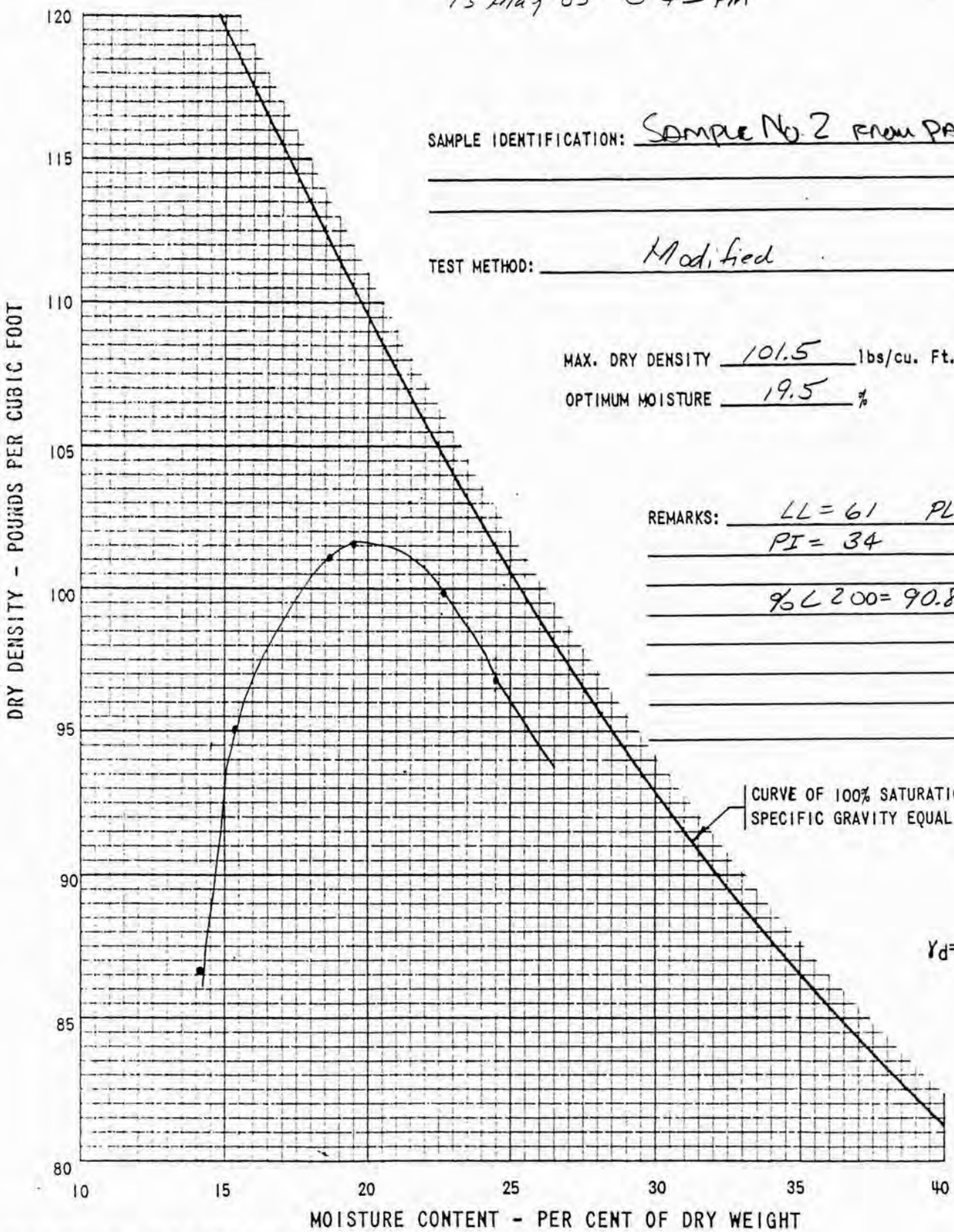
MOISTURE-DENSITY RELATIONSHIP TEST

Job Name: Fred C. Hart Job No: 85B078-01 Date:

Boring No: Sample No: 151 Depth: Preparation:

Material:

Data from Carlos Sierra
13 May 85 @ 4:30 pm



SAMPLE IDENTIFICATION: Sample No. 2 from Page 1

TEST METHOD: Modified

MAX. DRY DENSITY 101.5 lbs/cu. Ft.

OPTIMUM MOISTURE 19.5 %

REMARKS: LL = 61 PL = 27
PI = 34
%L200 = 90.8%

$$\gamma_d = \frac{G\gamma_w}{1 + \frac{wG}{S}}$$

J & L TESTING COMPANY

Check Test to locate Sample #

ATTERBERG LIMITS DETERMINATION B511

Project HART Job No.

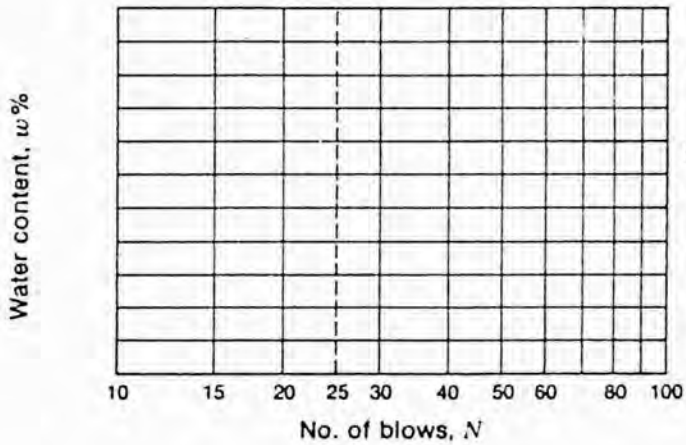
Location of Project SCI Boring No. 1S1 Sample No.

Description of Soil

Depth of Sample — Tested By Date 6/28/85

Liquid Limit Determination

Can no.	A711					
Wt. of wet soil + can	19.46					
Wt. of dry soil + can	13.77					
Wt. of can	4.29					
Wt. of dry soil	9.48					
Wt. of moisture	5.67					
Water content, w%	60.0					
No. of blows, N	2[illegible]					



Flow index F_i =

Liquid limit = 59

Plastic limit = 27

Plasticity index I_P = 32

Plastic Limit Determination

Can no.	A735			
Wt. of wet soil + can	17.13			
Wt. of dry soil + can	14.37			
Wt. of can	4.31			
Wt. of dry soil	10.06			
Wt. of moisture	2.76			
Water content, $w\% = w_P$	27			

TRIAXIAL PERMEABILITY TEST DATA

B51

Project Hart Job No. 85B078-01

Location SCI Boring No. 151 Sample No. ___

Description of Soil ___ Depth of Sample ___

Tested By [illegible] Date of Testing ___

Undisturbed ___ / Compacted ✓ Method ___

Desired Density (pcf) ___ M/C % ___

Diameter (in.) ___ Height (in.) ___

COMPACTION DATA

No. Layers ___ Wet Weight (grams) ___ per layer

Specific Gravity, (G_s) ___ Assumed Yes ✓ No ___

Cell No. ___ Desired "B" parameter B=1

Test Pressures:

Cell 45 psi Back 40/32 psi Head 8 psi

Saturation Increments:

Cell ___ psi Top ___ psi Bottom ___ psi

Deaired Water used for Saturation: Yes ✓ No ___

Other Liquid used for Saturation: TAP

Stones Saturated Yes ✓ No ___

Lines Saturated Yes ✓ No ___

Notes: ___

TRIAXIAL PERMEABILITY TEST DATA (cont.)

UNDISTURBED SAMPLE DATA

Initial Data

Height _____ in. (H_o)

Diameter _____ in. (D_o)

Area _____ sq. in. (A_o)

Volume _____ cu. in. (V_o)

Weight _____ g. (W_o)

Wet Density _____ pcf (γ_{wo})

Water content determination (TRIMMINGS)	
Wt. of can + wet soil	= 457.4
Wt. of can + dry soil	= 362.6
Wt. of can M30	= 9.5
Wt. of water	= 94.8
Wt. of dry soil	= 353.1
Initial water content w_i	= 26.8

COMPACTED SAMPLE DATA

Total Weight of Soil for Compaction _____ g (wet)

Initial Moisture Content _____ %

No. of Layers _____ Weight per Layer _____ g (wet)

Height 2.65 in. (H_o)

Diameter 2.30 in. (D_o)

Area 6.1575 sq. in. (A_o)

Volume 0.0094429 cu. in. (V_o)

Wet Density 113.1 pcf (γ_{wo})

γ_d = 89.2 pcf

Initial Height of Soil, H_i _____ Computed Dry Weight of Soil, W_s' = _____

Weight of Wet Soil, W_t = _____ Oven Dry Weight of Soil, W_s = _____

Computed Height of Solids, $H_o = W_s / G_s A_o$ = _____

Initial Height of Voids, $H_v = H_i - H_o$ = _____

Initial Degree of Saturation, $S_i = (W_t - W_s)/(H_i - H_o) A_o$ = _____

Initial Void Ratio, $e_o = H_v / H_o$ = _____

TRIAXIAL PERMEABILITY TEST DATA (cont.)

CHAMBER DATA - "SATURATION"

DATE	TIME	CELL PSI	TOP PSI	BOTTOM PSI	'B'	H(DIAL)	ΔH in.	A CORRECT	OTHER

Notes: _____

See Data Sheet

TEST PARAMETERS

COMPUTED

Height _____ in. (H_t): (L)

Area _____ sq. in. (A_t): (A)

Computed Wet Weight _____ g

CONFINEMENT / HEAD

Cell Pressure _____ psi (C)

Top Pressure _____ psi (TS)

Bottom Pressure _____ psi (BS)

Effective Head _____ psi (h)

Test data *Test data used*

Test No	*t*, sec	Q, cu cm	*T*, °C	Test No	*t*, sec	Q, cu cm	*T*, °C
1							
2							
3							
4							
				Average [a]			

$k_T = QL/Aht =$ _____ $\eta_T/\eta_{20} =$ _____

$=$ _____ cm/sec $k_{20} = k_T\eta_T/\eta_{20} =$ _____ cm/sec

TRIAXIAL PERMEABILITY TEST DATA (cont.)

Notes: _____

Initial Dial Reading _____

Final Dial Reading _____

Change in Sample Height _____

Final Height of Voids, H_{vf} _____

Final Void Ratio, $e_f = H_{vf}/H_o$ _____

DISASSEMBLED CELL (Measured Dimensions)

Height [illegible] in. (H_f)

Diameter [illegible] in. (D_f)

Area 6.3793967 sq. in. (A_f)

Volume 0.01000[illegible] cu. in. (V_f)

Wet Weight [illegible] g. (W_f)

Wet Density 114.6 pcf (γ_f)

Dry Density 84.2 pcf (γ_{df})

Sample Expanded

FINAL MOISTURE CONTENT

Final Wet Weight + Tare [illegible] g

Final Dry Weight + Tare 380.0 g

Weight of Water 133.2 g

Tare Weight [illegible] g

Oven Dry Weight of Soil, W_s 370.7 g

Final Water Content, W_f 36.1 %

Final Degree of Saturation, S 100 %

FOR COMPACTED SAMPLES

Maximum Density (Dry) 101.5 pcf

Optimum M/C 19.5 %

Percent Compaction 83 % After

88 % Before

DATA SHEET

TRIAXIAL PERMEABILITY TEST DATA

Job No: _____

Sample No: _____

Sheet No: _____

Description: _____

DATE	TIME	CELL PRESS	BURRETT	DELTA CELL BUR	TOP PRESS.	BURRETT	DELTA TOP BURR	BOT PRESS.	BURRETT	DELTA BOT BURR
[illegible]	[illegible]	25	[illegible]		17	[illegible]		[illegible]	3.[illegible]	
	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]		[illegible]	[illegible]	
	[illegible]	[illegible]			[illegible]	[illegible]		[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]		[illegible]	23.5		[illegible]	7.0	
	[illegible]	[illegible]	[illegible]		[illegible]	18.[illegible]	[illegible]	[illegible]	7.[illegible]	
	[illegible]	[illegible]	7.		[illegible]	15	[illegible]		[illegible]	
	[illegible]	[illegible]	7.[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]		32	22.5		[illegible]	[illegible]	
	[illegible]		[illegible]			[illegible]		[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]			[illegible]		[illegible]	[illegible]	
		[illegible]	[illegible]			[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]			3.[illegible]	
	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
			[illegible]	[illegible]		[illegible]	2.[illegible]		[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]			[illegible]	[illegible]		[illegible]	
	[illegible]	[illegible]	[illegible]	24		[illegible]	[illegible]		[illegible]	2.5
			[illegible]	57		[illegible]	5.5	[illegible]	[illegible]	5.5
	[illegible]	[illegible]	7[illegible]	9[illegible]		[illegible]	8.2	[illegible]	[illegible]	8.2
	[illegible]	[illegible]	[illegible]	145	[illegible]	[illegible]	12.4	[illegible]	[illegible]	12.4
	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]			[illegible]	
	[illegible]	[illegible]	7[illegible]		[illegible]	[illegible]		[illegible]	[illegible]	
	[illegible]				[illegible]	23.1	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]		[illegible]			[illegible]		

BY _____ DATE _____ | J & L TESTING Co | SHEET ___ OF ___ PAGE

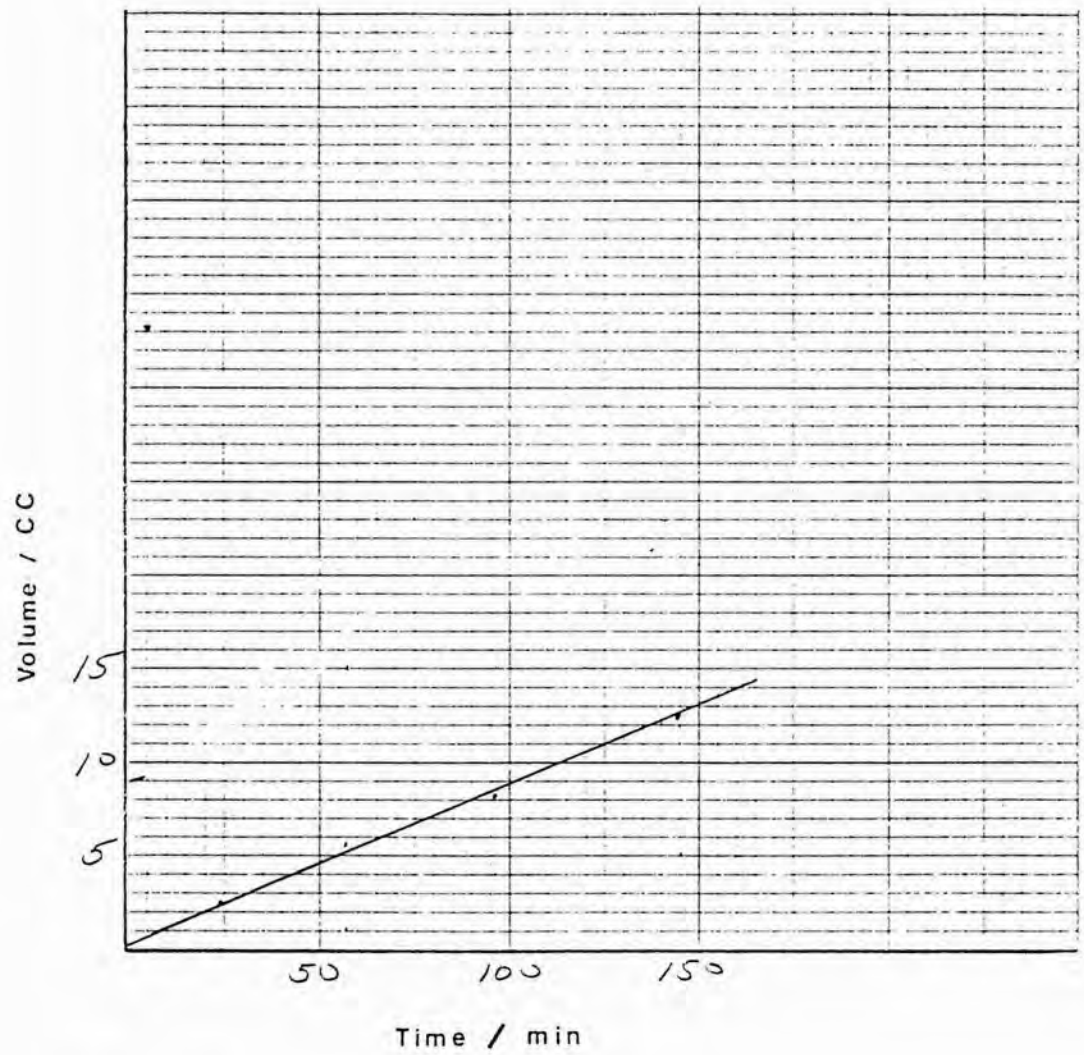
CHK'D _____ DATE _____

SUBJECT _____ JOB NO _____

FLOW vs TIME

$\Delta V = 0$

$K = \frac{QL}{Aht}$



$Q = 9$ cc

$L = 2.72$ in

$A = 0.0100416$ sq in

$h = 3$ psi

$t = 6000$ sec.

$K = 1.97 \times 10^{-6}$ cm/sec.

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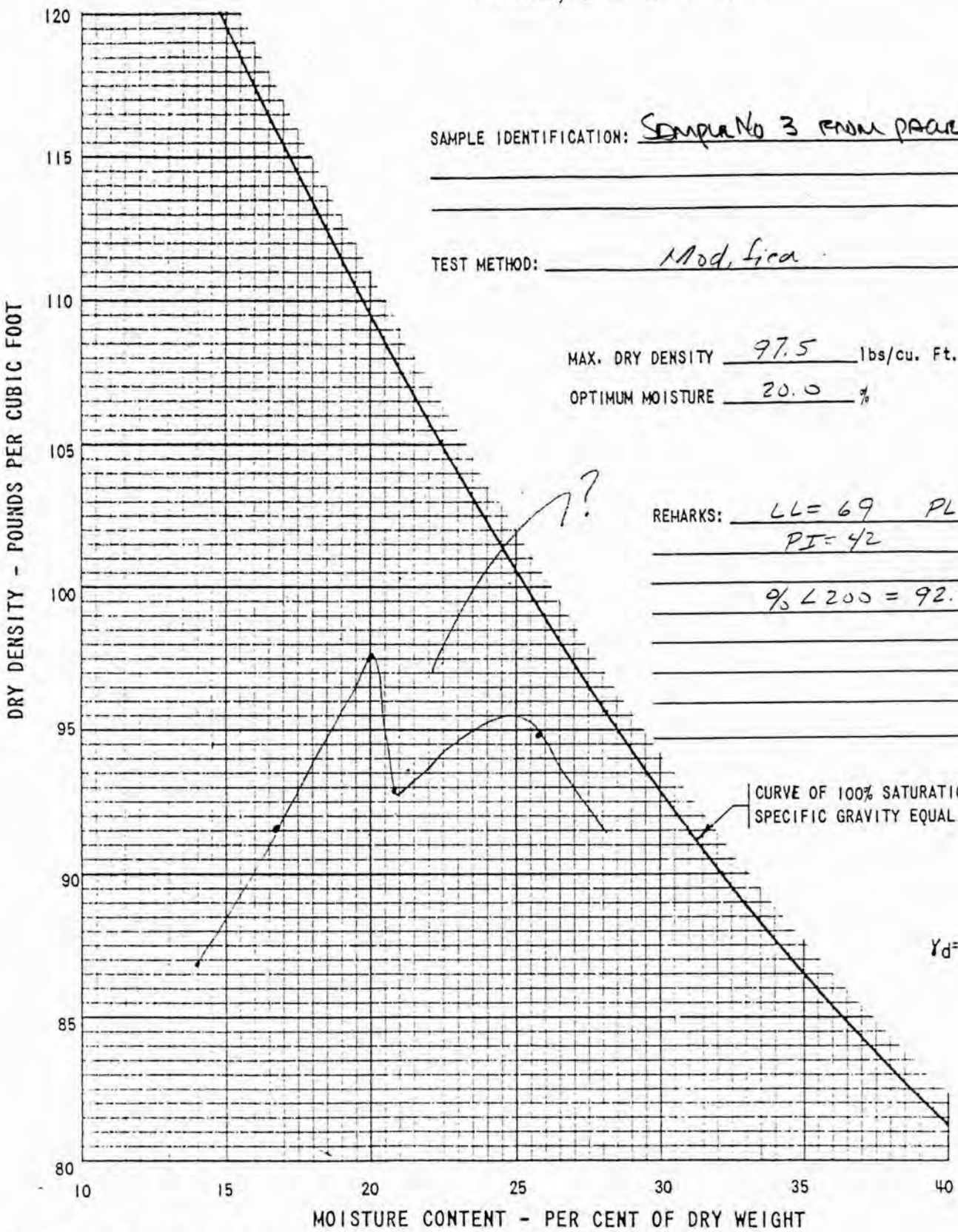
MOISTURE-DENSITY RELATIONSHIP TEST

Job Name: Fred C Hart Job No: 85B076-01 Date:

Boring No: Sample No: 253 Depth: Preparation:

Material:

Data from Carlos Sierra
13 May 85 @ 4:30 pm



SAMPLE IDENTIFICATION: Sample No 3 from page 1

TEST METHOD: Mod, fica

MAX. DRY DENSITY 97.5 lbs/cu. Ft.

OPTIMUM MOISTURE 20.0 %

REMARKS: LL= 69 PL=28
PI= 42
% < 200 = 92.9 %

$$\gamma_d = \frac{G\gamma_w}{1 + \frac{wG}{S}}$$

J & L TESTING COMPANY

ATTERBERG LIMITS DETERMINATION

B 5

Project HART Job No.

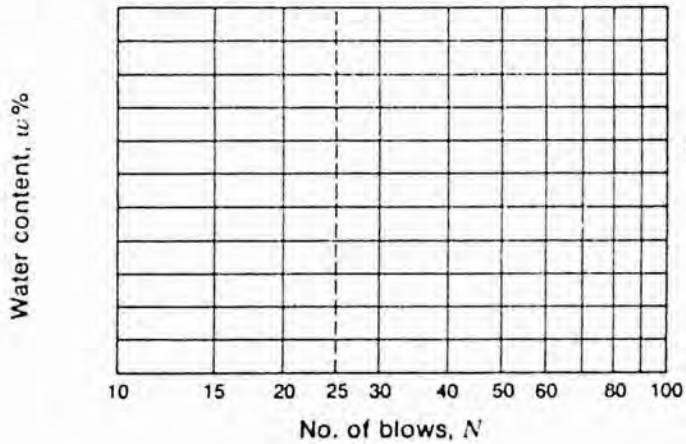
Location of Project SCI Boring No. 253 Sample No.

Description of Soil —

Depth of Sample — Tested By Gb Date 6/20/

Liquid Limit Determination

Can no.	AT6					
Wt. of wet soil + can	16.48					
Wt. of dry soil + can	10.35					
Wt. of can	4.28					
Wt. of dry soil	6.07					
Wt. of moisture	4.13					
Water content, w%	68.1					
No. of blows, N	25					



Flow index F_i =

Liquid limit = 68

Plastic limit = 33

Plasticity index I_P = 35

Plastic Limit Determination

Can no.	AT33			
Wt. of wet soil + can	15.20			
Wt. of dry soil + can	12.51			
Wt. of can	4.26			
Wt. of dry soil	8.27			
Wt. of moisture	2.69			
Water content, w% = w_P	33			

TRIAXIAL PERMEABILITY TEST DATA

8511

Project HART Job No. 85B078-01

Location SCI Boring No. 253 Sample No. ___

Description of Soil ___ Depth of Sample ___

Tested By [illegible] Date of Testing ___

Undisturbed ___ / Compacted ✓ Method ___

Desired Density (pcf) ___ M/C % ___

Diameter (in.) ___ Height (in.) ___

COMPACTION DATA

No. Layers ___ Wet Weight (grams) ___ per layer

Specific Gravity, (G_s) ___ Assumed Yes ✓ No ___

Cell No. ___ Desired "B" parameter B=1

Test Pressures:

Cell 45 psi Back 40/32 psi Head 8 psi

Saturation Increments:

Cell ___ psi Top ___ psi Bottom ___ psi

Deaired Water used for Saturation: Yes ✓ No ___

Other Liquid used for Saturation: TAP

Stones Saturated Yes ✓ No ___

Lines Saturated Yes ✓ No ___

Notes: ___

TRIAXIAL PERMEABILITY TEST DATA (cont.)

UNDISTURBED SAMPLE DATA

Initial Data

Height _____ in. (H_o)

Diameter _____ in. (D_o)

Area _____ sq. in. (A_o)

Volume _____ cu. in. (V_o)

Weight _____ g. (W_o)

Wet Density _____ pcf (γ_{wo})

Water content determination (TRIMMINGS)	
Wt. of can + wet soil	= 460.7
Wt. of can + dry soil	= 352.9
Wt. of can m3	= 9.5
Wt. of water	= 110.9
Wt. of dry soil	= 344.4
Initial water content w_i	= 32.2

COMPACTED SAMPLE DATA

Total Weight of Soil for Compaction 779.5 g (wet)

Initial Moisture Content _____ %

No. of Layers _____ Weight per Layer _____ g (wet)

Height 4.15 in. (H_o)

Diameter 2.80 in. (D_o)

Area 6.15752 sq. in. (A_o)

Volume 0.0147880293 cu. in. (V_o)

Wet Density 116.1 pcf (γ_{wo})

S = 77 %

γ_d = 87.8 pcf

Initial Height of Soil, H_i _____ Computed Dry Weight of Soil, W'_s = _____

Weight of Wet Soil, W_t = _____ Oven Dry Weight of Soil, W_s = _____

Computed Height of Solids, $H_o = W_s / G_s A_o$ = _____

Initial Height of Voids, $H_v = H_i - H_o$ = _____

Initial Degree of Saturation, $S_i = (W_t - W_s)/(H_i - H_o) A_o$ = _____

Initial Void Ratio, $e_o = H_v / H_o$ = _____

TRIAXIAL PERMEABILITY TEST DATA (cont.)

Notes: _____

Initial Dial Reading _____

Final Dial Reading _____

Change in Sample Height _____

Final Height of Voids, H_{vf} _____

Final Void Ratio, $e_f = H_{vf}/H_o$ _____

DISASSEMBLED CELL (Measured Dimensions)

Height 4.14 in. (H_f)

Diameter 2.80 in. (D_f)

Area 6.1575 sq. in. (A_f)
0.0027606 sq ft

Volume 0.01077477 cu. in. (V_f)

Wet Weight 784.8 g. (W_f)

Wet Density 117.0 pcf (γ_f)

Dry Density 87.9 pcf (γ_{df})

G_s = 2.64

FINAL MOISTURE CONTENT

Final Wet Weight + Tare 794.2 g

Final Dry Weight + Tare 599 g

Weight of Water 195.2 g

Tare Weight M26 4 g

Oven Dry Weight of Soil, W_s 587.6 g

Final Water Content, W_f 33.1 %

Final Degree of Saturation, S 100 %

FOR COMPACTED SAMPLES

Maximum Density (Dry) 97.5 pcf

Optimum M/C 20.0 %

Percent Compaction 90.2 % After
90.0 Before

DATA SHEET

TRIAXIAL PERMEABILITY TEST DATA

Job No: B5[illegible]

Sample No: 253

Sheet No:

Description:

DATE	TIME	CELL PRESS	BURRETT	DELTA CELL BUR	TOP PRESS.	BURRETT	DELTA TOP BURR	BOT PRESS.	BURRETT	DELTA BOT BURR
21 MAY										
										B=1
				START	Perm	Test				
24 MAY	0530	45	12.6	0	32	20.8	—	40	6.8	—
	0630	45	12.6	60	32	19.95	0.85	40	7.65	.85
	0720	45	12.6	110	32	19.25	1.55	40	8.35	1.[illegible]
	0805	45	12.6	155	32	18.6	2.2	40	9.0	[illegible]
	0915	45	12.6	225	32	17.65	3.15	40	9.95	3.15

SAME AS OTHER TESTS.

BY [illegible] DATE
CHK'D DATE

J & L TESTING Co

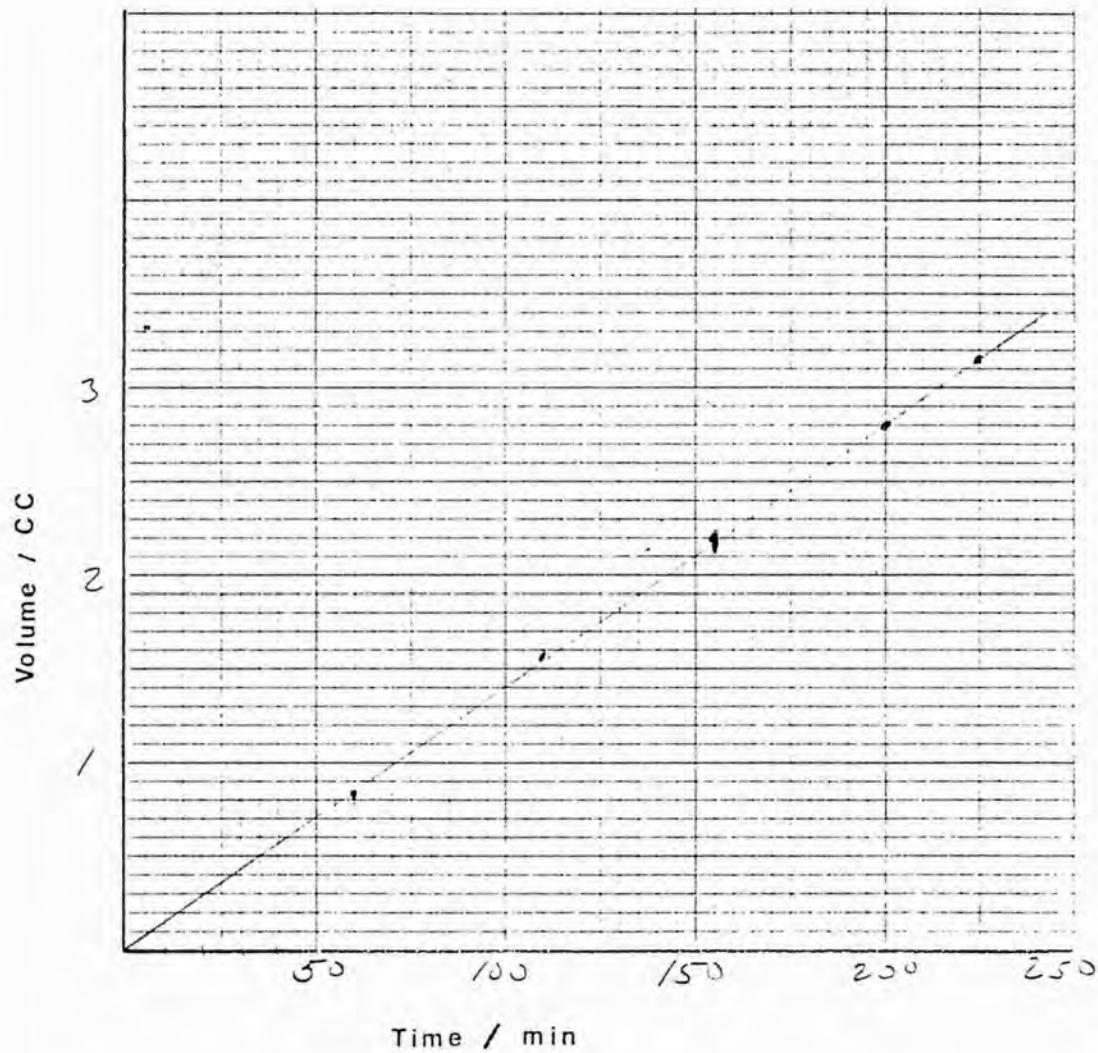
SHEET ___ OF ___ PAGE

SUBJECT 253 JOB NO 1251

FLOW vs TIME

$\Delta V = 0$

$K = \frac{Q}{A h t}$



Q = 2.8

L = 4.14

A = 0.0427636

r = 5 [illegible]

t 12,000

$K = 1.09 \times 10^{-7}$ cm/sec

SUMMARY OF LABORATORY TEST RESULTS

BORING and SAMPLE No.	DEPTH - feet	CLASSIFICATION	SPECIAL TESTS	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		UNCON. COMPRESS.		UNIT DRY WGT. (pcf)	SPECIFIC GRAVITY	GRAIN SIZE	OPT MOIST	CONSOLID.	TRIAXIAL			
					LIQUID LIMIT	PLASTIC LIMIT	STRESS (tsf)	STRAIN (%)			b			U.U.	$\overline{CIU}$	CELL PRESSURE (psi)	BACK PRESSURE (psi)
TB 1 S-4	9.0-10.5	CH		22.6	66	28					93.4						
TB 1 S-9	21.5-23.3	CH		37.1	75	32			82.6		99.8		*				
TB 1 S-12	31.5-33.3	MH		33.6	65	35			X		94.6			X			
TB 1 S-13	33.3-34.8	CH-MH		27.9	64	32					87.1						
TB 1 S-14	40.0-41.5	CH		29.6	64	29					91.5						
TB 1 S-18	56.5-57.25	MH	[a]X	29.4	56	34			X		94.8						
TB 1 S-20	65.0-66.5			29.5							94.6						
TB 1 S-22	71.5-72.1	MH	[a]X	10.9	59	38			X		98.4						
TB 2 S-4	9.0-10.5	CH		29.7	74	27					93.6						
TB 2 S-7	20.0-21.5	CH		26.8	62	26					88.6						
TB 2 S-11	36.5-38.3	MH		31.6	73	37			X		90.8		X				
TB 2 S-13	45.0-46.5	CH-MH		24.3	60	31					100.0						
TB 2 S-16	60.0-60.9	CL		12.4	35	19					50.9						
252	Bulk Storage	Recompacted at field water content															

* See Test Curves

X - unable to complete test, see discussion, pg. / a - Unconfined Compression test / b - % Passing # 200 Sieve

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PERCENT PASSING #200
SIEVE ANALYSIS

TB511K

PROJECT: HART / RESOURCE Mgmt

JOB NO.: 85B081-01

PROJECT LOCATION: P.R.

BY: [illegible]

DATE: 8/5/85

Boring	TB-1	TB-1	TB-1	TB-1	
Sample	S-4	S-13	S-14	S-20	
Depth	9-10½	33.3-34.8	40-41½	65-66½	
Pan No.	D27	D41	D20	D12	
Pan & Dry Soil (gr) (Before Wash)	119.2	136.3	139.4	164.7	
Pan & Dry Soil (gr) (After Wash)	44.5	52.0	47.4	46.3	
Pan (gr)	39.2	39.5	38.8	39.6	
Dry (Before)	80.0	96.8	100.6	125.1	
Dry (After)	5.3	12.5	8.6	6.7	
% Passing	93.4	87.1	91.5	94.6	

Boring	TB-2	TB-2	TB-2	TB-2	
Sample	S-4	S-7	S-13	S-16	
Depth	9-10½	20-21½	45-46½	60-60.9	
Pan No.	D56	D18	D28	D29	
Pan & Dry Soil (gr) (Before Wash)	93.0	114.3	116.8	155.5	
Pan & Dry Soil (gr) (After Wash)	43.0	47.0	38.5	96.3	
Pan (gr)	39.6	38.3	38.5	39.1	
Dry (Before)	53.4	76.0	—	116.4	
Dry (After)	3.4	8.7	0	57.2	
% Passing	93.6	88.6	100.0	50.9	

PERCENT PASSING #200
SIEVE ANALYSIS

B511K

PROJECT: Resource Mgmt

JOB NO.: 85B081-01

PROJECT LOCATION: Puerto Rico

BY: JKy

DATE: 1/9/85

Boring	TB-1	TB-1	TB-1	TB-1	TB-2
Sample	S-9	S-12	S-18	S-22	S-11
Depth	21½-23.5	31.5-33.3	56.5-57.25	71½-72.1	36.5-38.3
Pan No.	D12	D31	D32	D14	D47
Pan & Dry Soil (gr) (Before Wash)	125.3	121.7	132.4	138.2	139.2
Pan & Dry Soil (gr) (After Wash)	39.4	43.6	43.3	39.8	48.4
Pan (gr)	39.2	39.1	38.7	38.2	39.2
Dry (Before)	86.1	82.6	89.1	100.0	101.0
Dry (After)	0.2	4.5	4.6	1.6	9.2
% Passing	99.8	94.6	94.8	98.4	90.8

Boring					
Sample					
Depth					
Pan No.					
Pan & Dry Soil (gr) (Before Wash)					
Pan & Dry Soil (gr) (After Wash)					
Pan (gr)					
Dry (Before)					
Dry (After)					
% Passing					

ORBITAL ENGINEERING INC.

B511K

WATER CONTENT DETERMINATION

Project Resource Mgmt Job No. 85B081-01

Location of Project R.R

Description of Soil

Tested By Date of Testing 8/5/86

Date of Weighing 8/6/86

Boring no.	TB-1 S-4	TB-1 S-13	TB-1 S-14	TB-1 S-20	
Container no. (cup)	D24	D45	D20	D12	
Wt. of cup + wet soil	133.3	159.8	169.2	201.7	
Wt. of cup + dry soil	116.1	133.2	139.4	164.7	
Wt. of cup	39.9	37.9	38.6	39.2	
Wt. of dry soil	76.2	95.3	100.8	125.5	
Wt. of water	17.2	26.6	29.8	37.0	
Water content, w%	22.6	27.9	29.6	29.5	

Boring no.	TB-2 S-4	TB-2 S-7	TB-2 S-13	TB-2 S-16	
Container no. (cup)	D53	D19	D42	D39	
Wt. of cup + wet soil	177.5	199.4	160.5	227.9	
Wt. of cup + dry soil	145.8	165.3	136.7	207.1	
Wt. of cup	39.6	38.3	38.8	39.2	
Wt. of dry soil	106.8	127.0	97.9	167.9	
Wt. of water	31.7	34.1	23.8	20.8	
Water content, w%	29.7	26.8	24.3	12.4	

$$w\% = \frac{\text{(tare plus wet soil)} - \text{(tare plus dry soil)}}{\text{(tare plus dry soil)} - \text{(tare)}} \times 100 = \frac{W_w}{W_s} \times 100$$

Remarks

Technician Computed by Checked by

WATER CONTENT DETERMINATION

Project Resource Mgmt Job No. 85 2081-01

Location of Project Puerto Rico

Description of Soil

Tested By JA Date of Testing 8/7/85

Date of Weighing 8/8/85

Boring no.	TB-1 S-22 71½ - 72.1 ft				
Container no. (cup)	D2				
Wt. of cup + wet soil	340.7				
Wt. of cup + dry soil	311.1				
Wt. of cup D2	39.5				
Wt. of dry soil	271.6				
Wt. of water	29.6				
Water content, w%	10.9				

Boring no.					
Container no. (cup)					
Wt. of cup + wet soil					
Wt. of cup + dry soil					
Wt. of cup					
Wt. of dry soil					
Wt. of water					
Water content, w%					

$$w\% = \frac{(\text{tare plus wet soil}) - (\text{tare plus dry soil})}{(\text{tare plus dry soil}) - (\text{tare})} \times 100 = \frac{W_w}{W_s} \times 100$$

Remarks

Technician Computed by JA Checked by

Sample Disturbed

ATTERBERG LIMITS DETERMINATION

Project Resource Mgmt Job No. 05 B 081-01

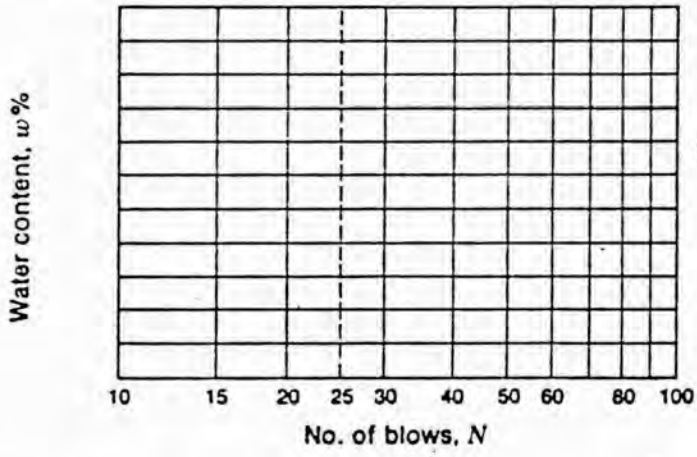
Location of Project Puerto Rico Boring No. TB-1 Sample No. S-22

Description of Soil

Depth of Sample 71½ - 72.1 Tested By JMg Date 8/10/85

Liquid Limit Determination

Can no.	AT49	AT51				D9
Wt. of wet soil + can	15.67	15.18				200.2
Wt. of dry soil + can	11.36	11.04				166.5
Wt. of can	4.31	4.32				38.8
Wt. of dry soil	7.05	6.72				127.7
Wt. of moisture	4.31	4.14				33.5
Water content, w%	61.1	61.6				26.4
No. of blows, N	18	18				



Flow index F_f =

Liquid limit = 59

Plastic limit =

Plasticity index I_P =

Plastic Limit Determination

Can no.	AT53	AT43		
Wt. of wet soil + can	10.40	11.96		
Wt. of dry soil + can	8.73	9.85		
Wt. of can	4.31	4.28		
Wt. of dry soil	4.42	[illegible]		
Wt. of moisture	1.67	2.11		
Water content, $w\% = w_P$	37.8	37.7		

ATTERBERG LIMITS DETERMINATION

Project Resource Mgmt Job No. 85A081-01

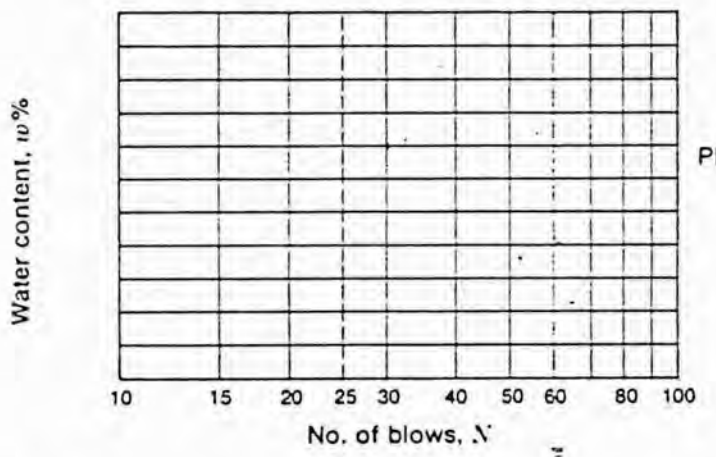
Location of Project P.R Boring No. TB-2 Sample No. S-4

Description of Soil

Depth of Sample 9-10' Tested By [illegible] Date 8/5/85

Liquid Limit Determination

Can no.	AT43	AT5				D53
Wt. of wet soil + can	14.84	14.51				
Wt. of dry soil + can	10.42	10.24				
Wt. of can	4.28	4.31				
Wt. of dry soil	6.14	5.93				
Wt. of moisture	4.42	4.27				
Water content, w%	72.0	72.0				
No. of blows, N	30	30				



Flow index F_i =

Liquid limit = 74

Plastic limit = 27

Plasticity index I_P = 47

Plastic Limit Determination

Can no.	AT1	AT53		
Wt. of wet soil + can	12.10	11.27		
Wt. of dry soil + can	10.45	9.81		
Wt. of can	4.29	4.31		
Wt. of dry soil	6.16	5.50		
Wt. of moisture	1.65	1.46		
Water content, $w\% = w_P$	26.8	26.5		

ATTERBERG LIMITS DETERMINATION

Project Resource MGMT Job No. 858081-01

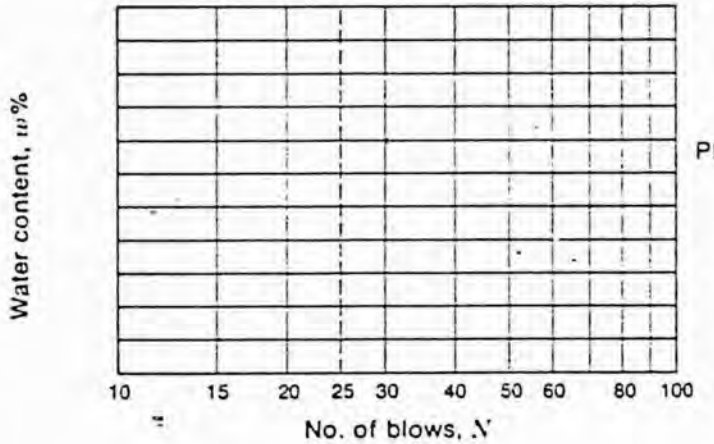
Location of Project P. R. Boring No. TB-2 Sample No. S-7

Description of Soil

Depth of Sample 20-21½ Tested By [illegible] Date 8/5/2.

Liquid Limit Determination

Can no.	AT45	AT54				619
Wt. of wet soil + can	13.66	13.15				
Wt. of dry soil + can	10.06	9.73				
Wt. of can	4.26	4.28				
Wt. of dry soil	5.80	5.45				
Wt. of moisture	3.60	3.42				
Water content, w%	62.1	62.7				
No. of blows, N	23	23				



Flow index F_i =

Liquid limit = 62

Plastic limit = 26

Plasticity index I_p = 36

Plastic Limit Determination

Can no.	AT28	AT22		
Wt. of wet soil + can	12.83	11.57		
Wt. of dry soil + can	11.04	10.04		
Wt. of can	4.28	4.26		
Wt. of dry soil	6.76	5.78		
Wt. of moisture	1.79	1.53		
Water content, $w\% = w_p$	26.5	26.5		

ATTERBERG LIMITS DETERMINATION

Project Resource Mgmt Job No. 852081-01

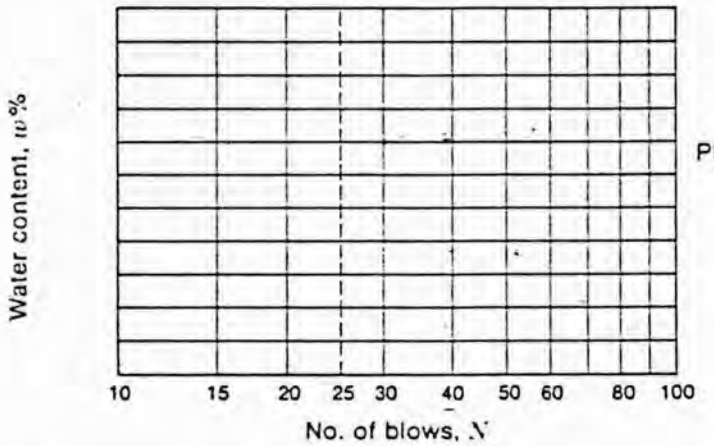
Location of Project Puerto Rico Boring No. TB-2 Sample No. S-11

Description of Soil

Depth of Sample 36½ - 38.3 Tested By [illegible] Date 8/11/85

Liquid Limit Determination

Can no.	AT16	AT96				D1
Wt. of wet soil + can	13.83	12.79				221.2
Wt. of dry soil + can	9.85	9.23				180.3
Wt. of can	4.28	4.28				39.1
Wt. of dry soil	5.57	4.96				141.2
Wt. of moisture	3.98	3.56				40.9
Water content, w%	71.5	71.8				29.0
No. of blows, N	28	28				



Flow index F_i = ____

Liquid limit = 73

Plastic limit = 37

Plasticity index I_P = 36

Plastic Limit Determination

Can no.	AT-1	AT35	—	
Wt. of wet soil + can	9.52	12.09		
Wt. of dry soil + can	8.12	9.99		
Wt. of can	4.29	4.31		
Wt. of dry soil	3.83	5.68		
Wt. of moisture	1.40	2.10		
Water content, $w\% = w_P$	36.6	37.0		

ATTERBERG LIMITS DETERMINATION

Project Resource Mgmt Job No. 85B081-01

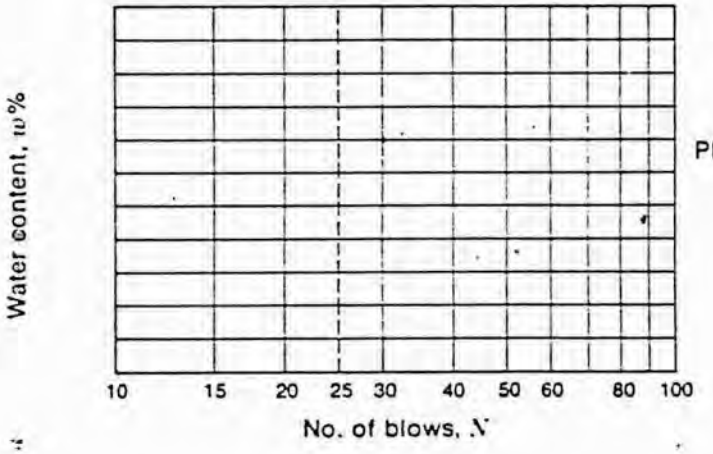
Location of Project P. R. Boring No. TB-2 Sample No. S-13

Description of Soil

Depth of Sample 45-46½ Tested By Jig Date 8/5/85

Liquid Limit Determination

Can no.	AT10	AT4				D42
Wt. of wet soil + can	13.56	11.87				
Wt. of dry soil + can	10.00	8.99				
Wt. of can	4.29	4.31				
Wt. of dry soil	5.76	4.68				
Wt. of moisture	3.56	2.88				
Water content, w%	61.8	61.5				
No. of blows, N	20	20				



Flow index F_i =

Liquid limit = 60

Plastic limit = 31

Plasticity index I_P = 29

Plastic Limit Determination

Can no.	AT33	AT40		
Wt. of wet soil + can	11.14	10.77		
Wt. of dry soil + can	9.49	9.23		
Wt. of can	4.24	4.29		
Wt. of dry soil	5.25	4.94		
Wt. of moisture	1.65	1.54		
Water content, $w\% = w_P$	31.4	31.7		

ATTERBERG LIMITS DETERMINATION

Project Resource Mgmt Job No. 85R081-01

Location of Project Puerto Rico Boring No. TB-2 Sample No. S-16

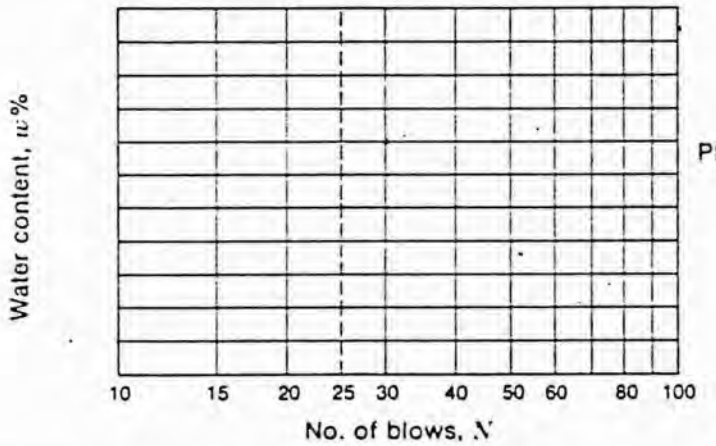
Description of Soil

Depth of Sample 60 – 60.9 Tested By [illegible] Date 4/5/85

Liquid Limit Determination

D39

Can no.	AT38	AT30				
Wt. of wet soil + can	13.38	15.18				
Wt. of dry soil + can	11.03	12.37				
Wt. of can	4.28	4.32				
Wt. of dry soil	6.75	8.05				
Wt. of moisture	2.35	2.81				
Water content, w%	34.8	34.9				
No. of blows, N	35	25				



Flow index F_i =

Liquid limit = 35

Plastic limit = 19

Plasticity index I_P = 16

Plastic Limit Determination

Can no.	AT17	AT26		
Wt. of wet soil + can	10.89	12.02		
Wt. of dry soil + can	9.85	10.80		
Wt. of can	4.28	4.23		
Wt. of dry soil	5.57	6.57		
Wt. of moisture	1.04	1.22		
Water content, $w\% = w_P$	18.7	18.56		

ATTERBERG LIMITS DETERMINATION

Project Resource MGMT Job No. 85B081-01

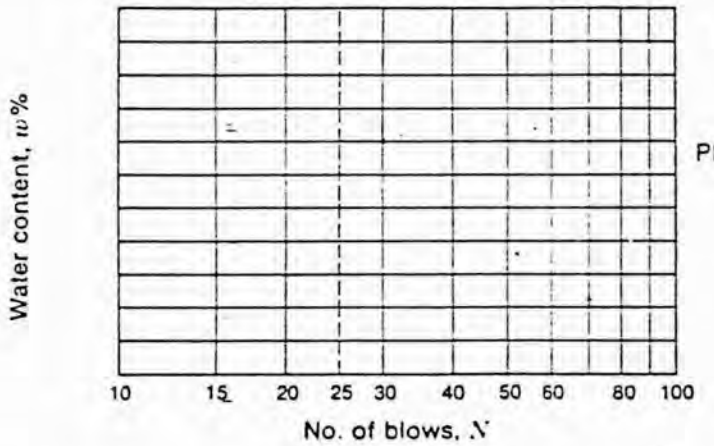
Location of Project P.R. Boring No. TB-1 Sample No. S-4

Description of Soil

Depth of Sample 9-10½ Tested By [illegible] Date 8/5/85

Liquid Limit Determination D 24

Can no.	AT 11	AT 21				
Wt. of wet soil + can	15.58	16.52				
Wt. of dry soil + can	11.08	11.65				
Wt. of can	4.29	4.29				
Wt. of dry soil	6.79	7.36				
Wt. of moisture	4.50	4.87				
Water content, w%	66.3	66.2				
No. of blows, N	24	24				



Flow index F_i =

Liquid limit = 66

Plastic limit = 28

Plasticity index I_P = 38

Plastic Limit Determination

Can no.	AT 6	AT 19		
Wt. of wet soil + can	10.78	11.63		
Wt. of dry soil + can	9.36	10.03		
Wt. of can	4.28	4.28		
Wt. of dry soil	5.08	5.75		
Wt. of moisture	1.42	1.60		
Water content, $w\% = w_P$	28.0	27.8		

ATTERBERG LIMITS DETERMINATION

Project Ros Mgmt Job No. 85B081-01

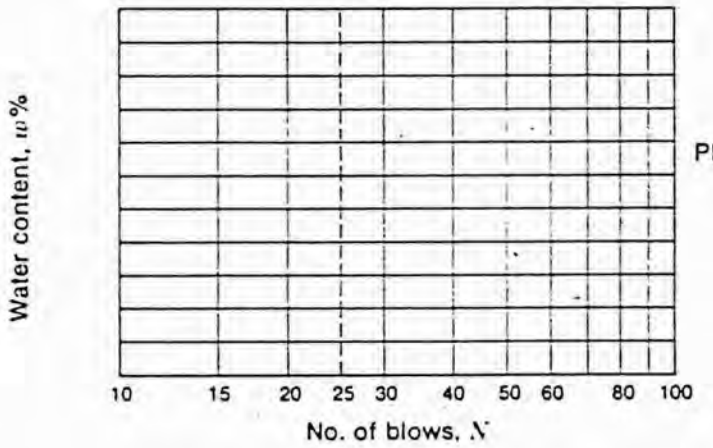
Location of Project Puerto Rico Boring No. TB-1 Sample No. S-9

Description of Soil

Depth of Sample 21.5-23.3 Tested By [illegible] Date 8/10/85

Liquid Limit Determination

Can no.	AT46	AT35				D22
Wt. of wet soil + can	12.82	12.54				177.5
Wt. of dry soil + can	9.16	9.02				140.2
Wt. of can	4.28	4.31				39.7
Wt. of dry soil	4.88	4.71				100.5
Wt. of moisture	3.66	3.52				37.3
Water content, w%	75	74.7				37.12
No. of blows, N	25	25				



Flow index F_i =

Liquid limit = 75

Plastic limit = 32

Plasticity index I_P = 43

Plastic Limit Determination

Can no.	AT49	AT14		
Wt. of wet soil + can	11.07	11.01		
Wt. of dry soil + can	9.43	9.38		
Wt. of can	4.31	4.29		
Wt. of dry soil	5.12	5.09		
Wt. of moisture	1.64	1.63		
Water content, w% = w_P	32.0	32		

ATTERBERG LIMITS DETERMINATION

Project Resource Mgmt Job No. 35C081-01

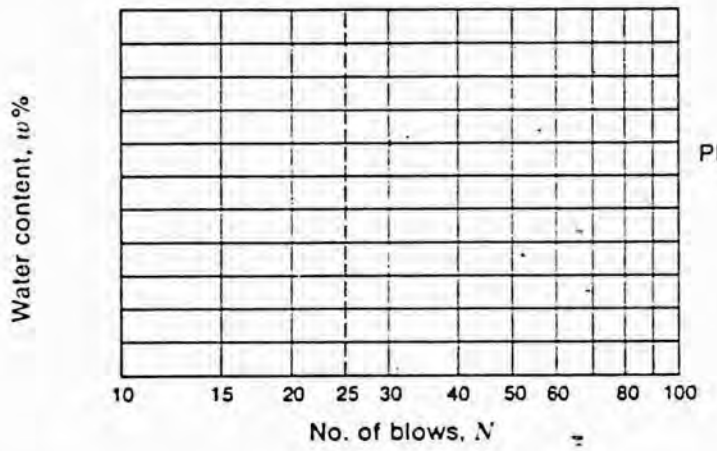
Location of Project Puerto Rico Boring No. TB-1 Sample No. S-12 TUBE

Description of Soil

Depth of Sample 31.5-33.3 Tested By [illegible] Date 8/1/1-

Liquid Limit Determination

Can no.	AT16	AT29				D5
Wt. of wet soil + can	13.47	13.89				175.1
Wt. of dry soil + can	9.69	9.96				140.6
Wt. of can	4.28	4.32				37.8
Wt. of dry soil	5.41	5.64				102.8
Wt. of moisture	3.78	3.93				34.5
Water content, w%	70.0	69.7				33.6 %
No. of blows, N	16	16				



Flow index F_i =

Liquid limit = 65

Plastic limit = 35

Plasticity index I_P = 30

Plastic Limit Determination

Can no.	AT51	AT55		
Wt. of wet soil + can	10.71	10.51		
Wt. of dry soil + can	9.03	8.89		
Wt. of can	6.32	6.29		
Wt. of dry soil	4.71	4.60		
Wt. of moisture	1.68	1.62		
Water content, $w\% = w_P$	35.7	35.2		

ATTERBERG LIMITS DETERMINATION

Project Resource MGMT Job No. 85B[illegible]1-01

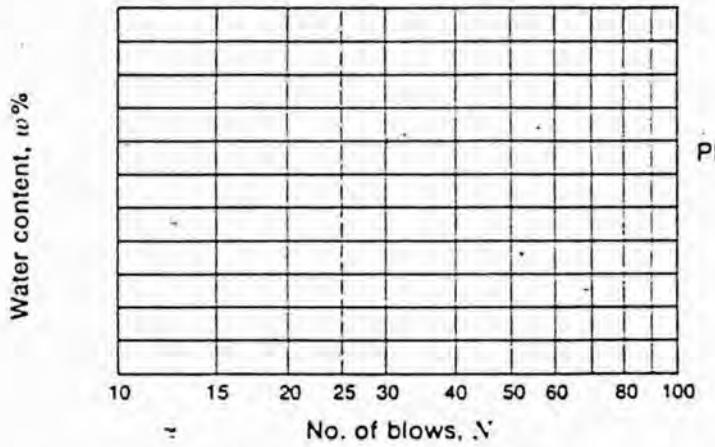
Location of Project P.R. Boring No. TB-1 Sample No. S-13

Description of Soil

Depth of Sample 33.3-34.8 Tested By [illegible] Date 8/5/[illegible]

Liquid Limit Determination D45

Can no.	AT7	AT37				
Wt. of wet soil + can	13.60	16.02				
Wt. of dry soil + can	9.93	11.42				
Wt. of can	4.26	4.31				
Wt. of dry soil	5.67	7.11				
Wt. of moisture	3.67	4.60				
Water content, w%	64.7	64.7				
No. of blows, N	22	22				



Flow index F_i = _____

Liquid limit = 64

Plastic limit = 32

Plasticity index I_P = 32

Plastic Limit Determination

Can no.	AT3	AT47		
Wt. of wet soil + can	10.05	12.45		
Wt. of dry soil + can	8.67	10.96		
Wt. of can	4.30	4.26		
Wt. of dry soil	4.37	6.20		
Wt. of moisture	1.38	1.99		
Water content, $w\% = w_P$	31.6	32.1		

B5111-

ATTERBERG LIMITS DETERMINATION

Project Resource Mgmt Job No. 85B081-01

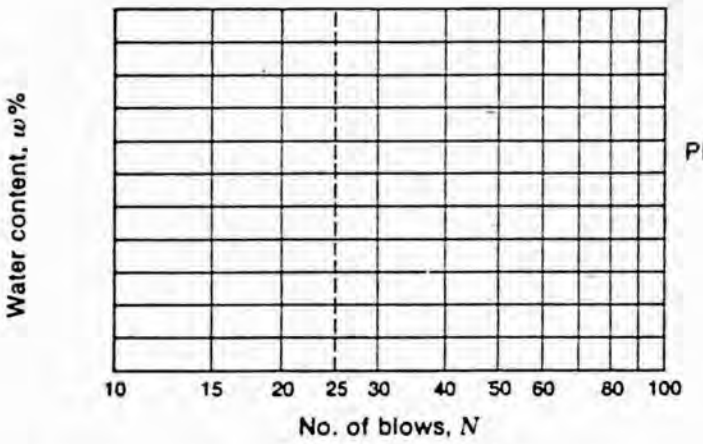
Location of Project Puerto Rico Boring No. TB-1 Sample No. S-14

Description of Soil Highly Plastic Clay

Depth of Sample 40-41.5 Tested By [illegible] Date 8/5/85

Liquid Limit Determination

Can no.	AT35	AT54				
Wt. of wet soil + can	20.52	18.48				
Wt. of dry soil + can	14.19	12.94				
Wt. of can	4.31	4.28				
Wt. of dry soil	9.88	8.66				
Wt. of moisture	6.33	5.54				
Water content, w%	64.1	64.0				
No. of blows, N	25	25				



Flow index F_1 = _____

Liquid limit = 64

Plastic limit = 29

Plasticity index I_P = 35

Plastic Limit Determination

Can no.	AT61	AT68		
Wt. of wet soil + can	14.98	15.29		
Wt. of dry soil + can	12.58	12.81		
Wt. of can	4.32	4.29		
Wt. of dry soil	8.26	8.52		
Wt. of moisture	2.40	2.48		
Water content, $w\% = w_P$	29.1	29.1		

ATTERBERG LIMITS DETERMINATION

Project Resource Mgmt Job No. 85B081-01

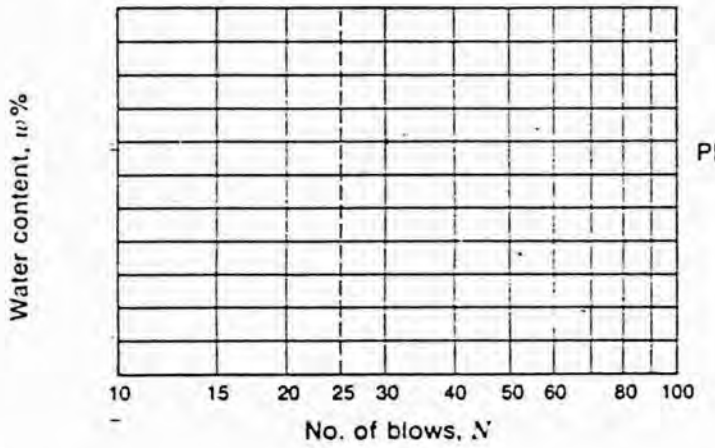
Location of Project Puerto Rico Boring No. TB-1 Sample No. S-18

Description of Soil

Depth of Sample 56½-57½ Tested By [illegible] Date 8/13/85

Liquid Limit Determination

Can no.	AT15	AT18				D17
Wt. of wet soil + can	12.99	12.93				209.2
Wt. of dry soil + can	9.79	9.73				170.6
Wt. of can	4.26	4.24				39.4
Wt. of dry soil	5.53	5.49				131.2
Wt. of moisture	3.20	3.20				38.6
Water content, w%	57.9	58.3				29.4 %
No. of blows, N	18	18				



Flow index F_i =

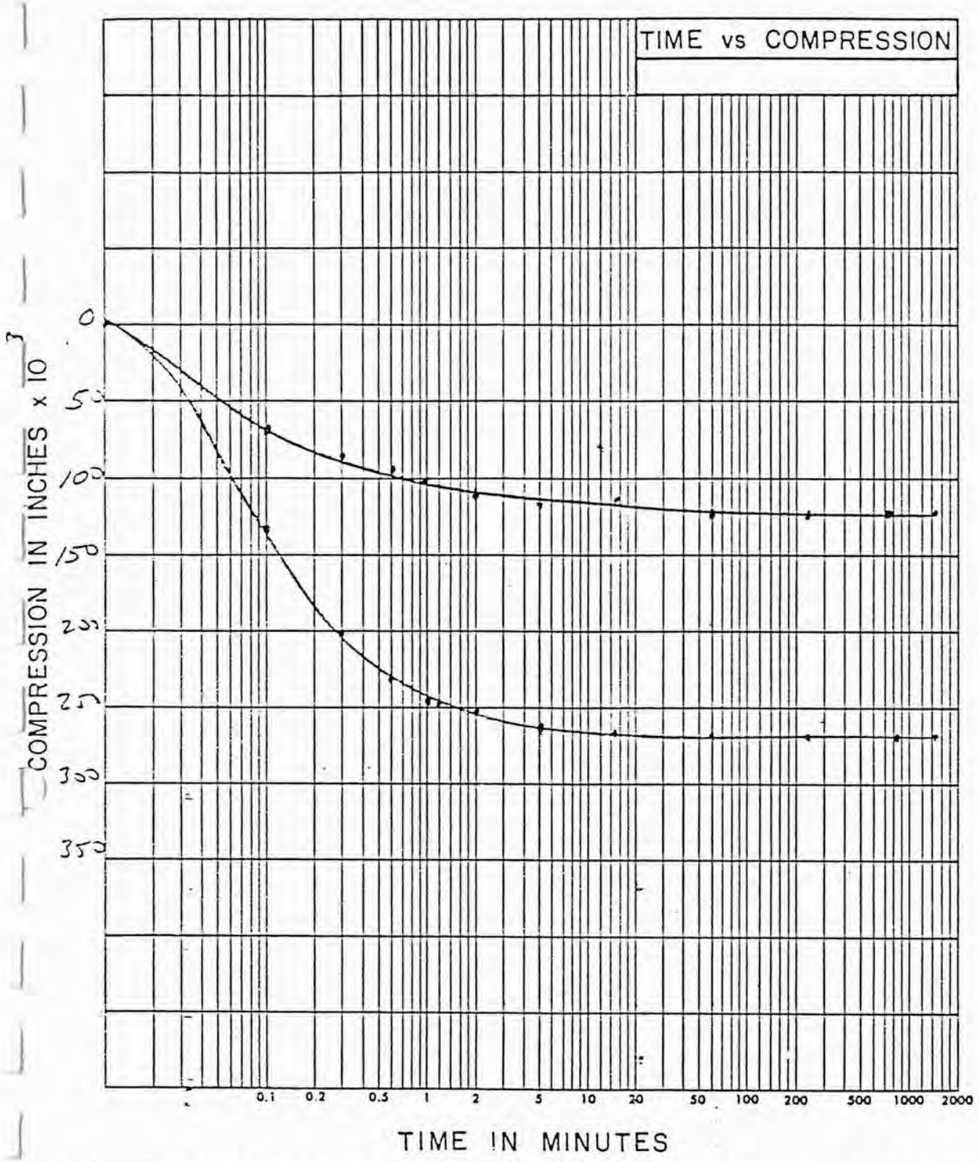
Liquid limit = 56

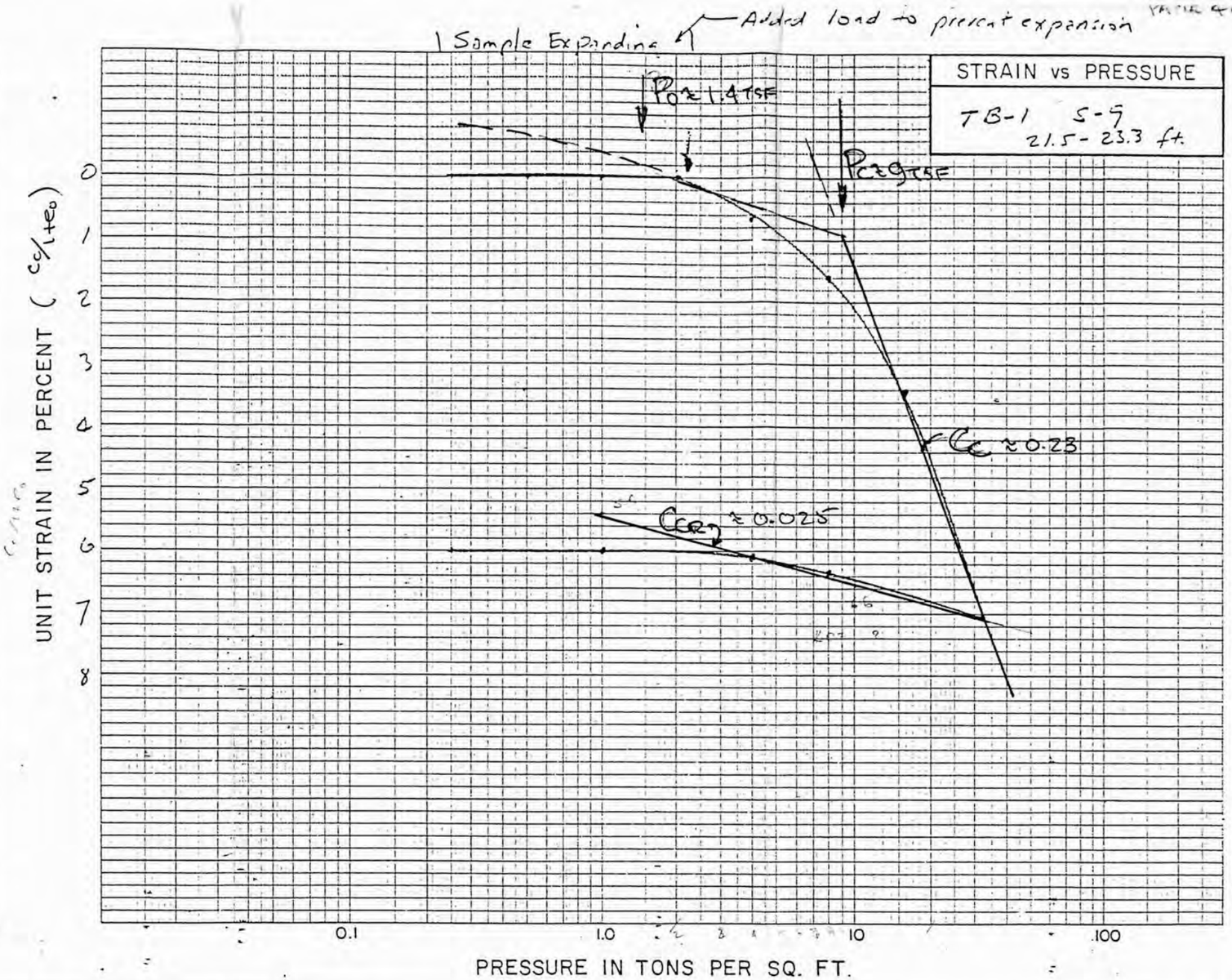
Plastic limit = 34

Plasticity index I_P = 22

Plastic Limit Determination

Can no.	AT36	AT48		
Wt. of wet soil + can	11.07	11.70		
Wt. of dry soil + can	9.31	9.79		
Wt. of can	4.26	4.26		
Wt. of dry soil	5.05	5.53		
Wt. of moisture	1.76	1.91		
Water content, $w\% = w_P$	34.8	34.5		





PRESSURE INCREMENTS OF TIME COMPRESSION CURVES

CURVE NUMBER	PRESSURE INCREMENT FROM (tsf)	PRESSURE INCREMENT TO (tsf)	COEFFICIENT OF CONS. (cm^2/sec)
(1)	8	16	
(2)	16	32	

DESCRIPTION OF SPECIMEN: Hard High plastic Overconsolidated clay (CH)

CONSOLIDATION PROPERTIES		TEST SPECIMEN PROPERTIES	INITIAL	FINAL
COMPRESSION INDEX (UNIT STRAIN)		WATER CONTENT, %	37.1	35.0
RECOMPRESSION INDEX (UNIT STRAIN)		VOID RATIO	1.1040	0.9785
SWELLING INDEX		SATURATION, %	93.2	99.8
PRECONSOLIDATION STRESS, tsf	8.9 TSF	SAMPLE HEIGHT, in.	0.75	0.7051
EXISTING OVERBURDEN STRESS, tsf		UNIT DRY WEIGHT, pcf	82.6	88.0
↳ 113x22 =	1.4 TSF	SAMPLE DIAMETER, in	2.50	
		LIQUID LIMIT, %	75	
		PLASTIC LIMIT, %	32	
FROM VOLUMETRIC STRAIN		SPECIFIC GRAVITY	2.79	

CONSOLIDATION TEST

$C_c/(1+e_o) \approx 11\% \approx 0.110$

$C_c \cong 0.110 \div (1+1.104)$

$\cong 0.23$

$C_{\varepsilon r} = 0.012\,(2.1040) = 0.025$ ✓

[redacted]

TESTED BY: [illegible] | CHKD BY:

DATE: 8/11/85 | JOB. No. B 5111

Sheet 1 of ___

CONSOLIDATION TEST

Project: Resource Mom — Job No. 81 B021-01

Location: Puerto Rico — Boring No. TB-1 — Sample No. S-9

Description of Soil: Highly Plastic Clay — Depth of Sample: 21.5 - 23.3

Tested By: JJ — Date of Testing: 8/7/85

Consolidometer type: Floating — Ring no. 3

Multiplication ratio of load device: —

Ring dimensions: Diam. 2.50 in / 6.35 cm — Area, A 31.6692 cm² — Ht. 0.75 in / 1.905 cm

Initial ht. of soil, H_i: 1.905 cm = 0.75 in

Specific gravity of soil, G_s = 2.79

Wt. of ring + specimen at beginning of test = 716.5 gr

Wt. of ring = 607.0 gr

Wt. of wet soil, W_t = 109.5 gr
γ_{wet} = 113.2 pcf

Computed dry weight of soil, W_s' = 79.9 gr
γ_d = 82.6 pcf

Oven dry wt. of soil,[a] W_s = 80.0 gr

Water content determination

Wt. of can + wet soil	= 177.5
Wt. of can + dry soil	= 140.2
Wt. of can D22	= 39.7
Wt. of water	= 100.5
Wt. of dry soil	= 37.3
Initial water content w_i	= 37.1

Computed Ht. of solids[b] $H_0 = W_s/G_sA$ = 80.0 / 2.79 (31.6692) = 0.9054

Initial Ht. of voids, $H_v = H_i - H_0$ = 1.905 - 0.9054 = 0.9996

Initial degree of saturation, $S_i = (W_t - W_s)/(H_i - H_0)A$ = 29.5 / (0.9996)(31.6692) = 93.2

Initial void ratio $e_o = H_v/H_0$ = 0.9996 / 0.9054 = 1.1040

Final Test Data (obtained at end of load testing)

Initial dial reading: 0.0000

Final dial reading: 0.0449

Change in sample ht. Δ = 0.0449

Final ht. of voids, H_{vf}: ___

Final void ratio, $e_f = H_{vf}/H_0$: 0.9785

Final water content determination

Final wet Wt. + ring[c]	715.0
Final dry wt. + ring	687.0
ring = 607 gr	
Oven dry wt. of soil, W_s	80.0 gr
Water gr =	28.0
Final water content, w_f	35.0
Final degree of sat. S	100 %

[a]Obtained from Final Test Data below.
[b]If it appears that any soil is lost from sample, use W_s'
[c]Be sure to include any soil extruded from ring which is in consolidometer

[redacted]

Sheet 1 of ___

B5 [illegible]

CONSOLIDATION TEST

Project: Resource Mgmt — Job No. 85B081-01

Location: Puerto Rico — Boring No. TB-2 Sample No. S-11

Description of Soil: Mottled Silty Clay — Depth of Sample: 36.5 - 38.3

Tested By: [signature] — Date of Testing: 8/7/85

Consolidometer type: Floating — Ring no. 1

Multiplication ratio of load device: —

Ring dimensions: Diam. 2.50 Area, A ___ Ht. 0.75

Initial ht. of soil, H_i: 0.75 — Tube out of round

Specific gravity of soil, G_s = 2.79

Wt. of ring + specimen at beginning of test = ___

Wt. of ring = ___

Wt. of wet soil, W_t = Disturbed

Computed dry weight of soil, W_s' = ___

Oven dry wt. of soil,[a] W_s = ___

Water content determination (Trimmings)	
Wt. of can + wet soil	= 200.2
Wt. of can + dry soil	= 161.5
Wt. of can D 11	= 39.0
Wt. of water	= 38.7
Wt. of dry soil	= 122.5
Initial water content w_i	= 31.6

Computed Ht. of solids[b] $H_0 = W_s/G_sA$ = ___

Initial Ht. of voids, $H_v = H_i - H_0$ = Sample Disturbed

Initial degree of saturation, $S_i = (W_t - W_s)/(H_i - H_0)A$ = ___

Initial void ratio $e_o = H_v/H_0$ = ___

Final Test Data (obtained at end of load testing)

Initial dial reading ___

Final dial reading ___

Change in sample ht. ___

Final ht. of voids, H_{vf} ___

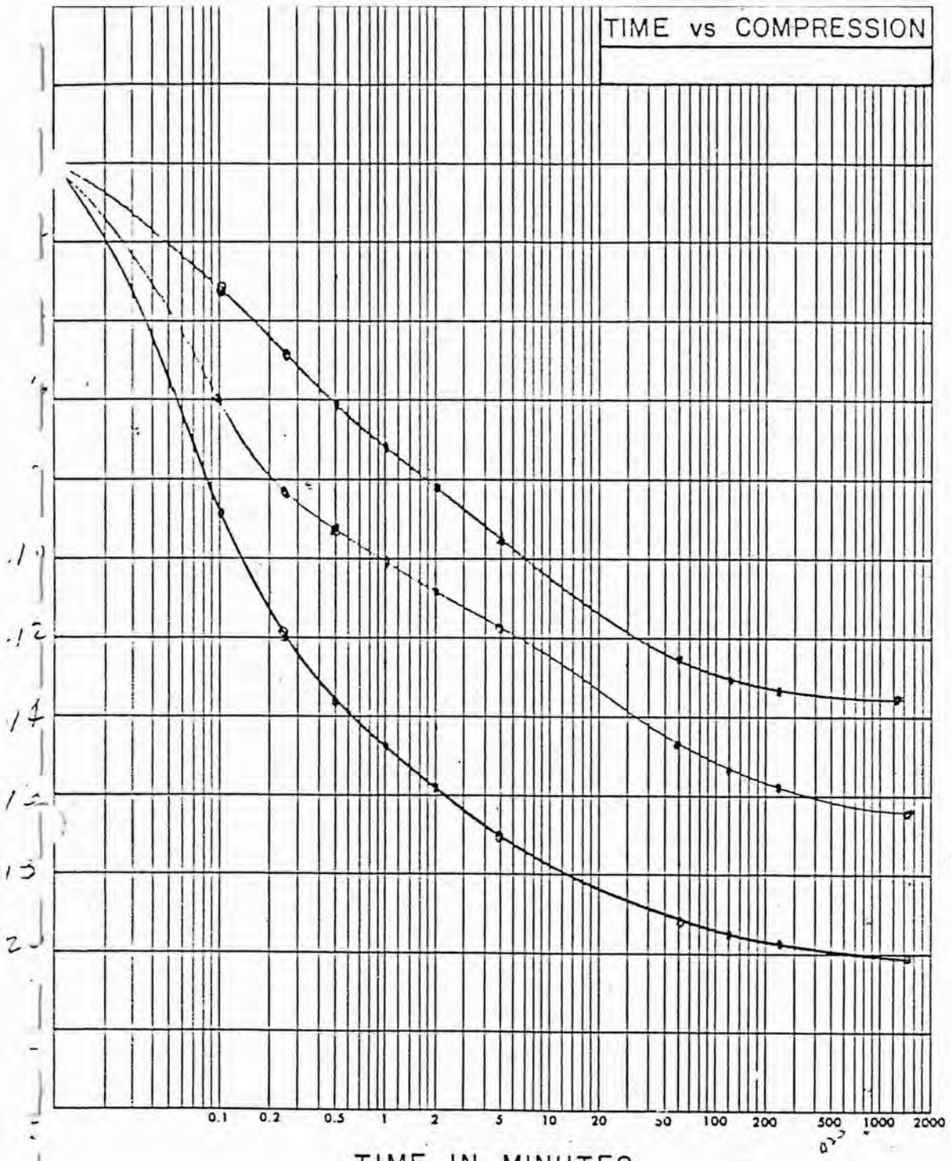
Final void ratio, $e_f = H_{vf}/H_0$ ___

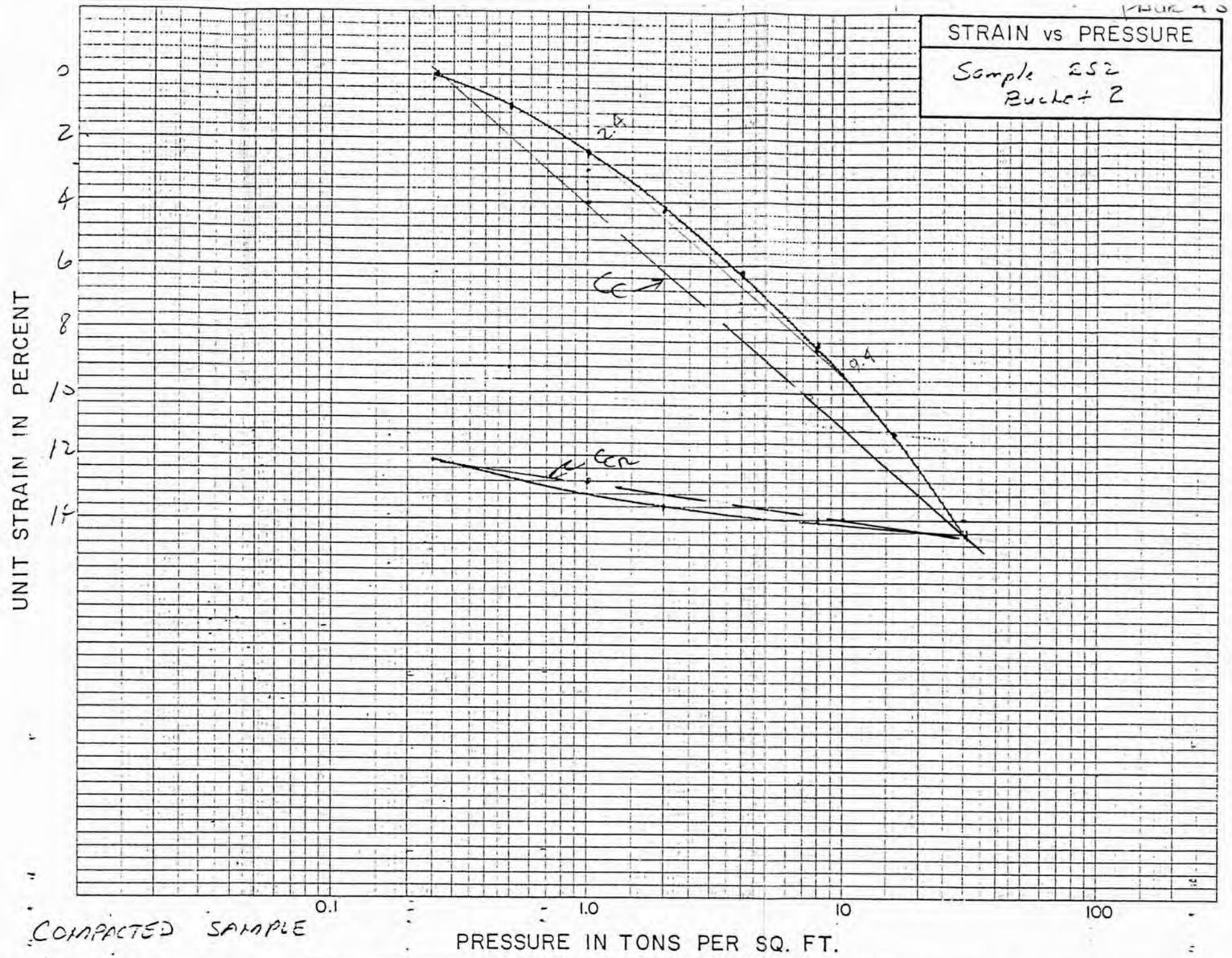
Final water content determination	
Final wet Wt. + ring[c]	___
Final dry wt. + ring	___
Oven dry wt. of soil, W_s	___
Final water content, w_f	___
Final degree of sat. S	___ %

[a]Obtained from Final Test Data below.
[b]If it appears that any soil is lost from sample, use W_s'.
[c]Be sure to include any soil extruded from ring which is in consolidometer.

- Sample expanded as it was extruded
- Tube out of round - Very hard material

[redacted]





COMPACTED SAMPLE

CURVE NUMBER	PRESSURE INCREMENT FROM (tsf)	TO (tsf)	COEFFICIENT OF CONS. (cm^2/sec)
(1)	1	2	
(2)	4	8	
(3)	8	16	

DESCRIPTION OF SPECIMEN: Highly Plastic CL (CL/CH)

CONSOLIDATION PROPERTIES		TEST SPECIMEN PROPERTIES	INITIAL	FINAL
COMPRESSION INDEX (UNIT STRAIN)	N/A	WATER CONTENT, %	20.6	17.9
RECOMPRESSION INDEX (UNIT STRAIN)	N/A	VOID RATIO	0.7607	.49941
SWELLING INDEX	N/A	SATURATION, %	81.5	100.0
PRECONSOLIDATION STRESS, tsf	N/A	SAMPLE HEIGHT, in.	0.75	.6592±
EXISTING OVERBURDEN STRESS, tsf	N/A	UNIT DRY WEIGHT, pcf	102.0	116.0
		SAMPLE DIAMETER, in	2.50	
		LIQUID LIMIT, %	48	
		PLASTIC LIMIT, %	17	
FROM VOLUMETRIC STRAIN		SPECIFIC GRAVITY	2.74	

CONSOLIDATION TEST

$C_c/1+e_0 = 0.07$ $C_c = 0.07 \times 1.71 \approx 0.12$

$C_{cr}/1+e_0 = 0.012$ $C_{cr} = 0.021$

[redacted]

TESTED BY: [illegible] CHKD BY:

DATE: 1/10/71 JOB. No. 85C081-01

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Sheet 1 of ___

CONSOLIDATION TEST

Project: Resource Miami — Job No. 85B081-01

Location: Puerto Rico — Boring No. #2 — Sample No. Bucket

Description of Soil: Highly Plastic Clay — Depth of Sample: 252

Tested By: [illegible] — Date of Testing: 8/6/85

Consolidometer type: Floating — Ring no. 3

Multiplication ratio of load device: —

Ring dimensions: Diam. 2.50 in (6.35 cm) Area, A = 31.6692 cm² Ht. 1.905 cm (0.75 in)

Initial ht. of soil, H_i = 1.905 cm

Specific gravity of soil, G_s = 2.79

Wt. of ring + specimen at beginning of test = 725.5 gr

Wt. of ring = 606.5 gr

Wt. of wet soil, W_t = 119.0 gr

Computed dry weight of soil, W_s' = 98.66 gr

γ_d = 102.0 pcf

Oven dry wt. of soil,[c] W_s = 98.7 gr

γ_{wet} = 123.0 pcf

Water content determination (Trimming)

Wt. of can + wet soil	207.4
Wt. of can + dry soil	172.5
Wt. of can D/2	38.2
Wt. of water	28.9
Wt. of dry soil	140.3
Initial water content w_i	20.6

Computed Ht. of solids[a] $H_s = W_s/G_sA$ = 98.66/2.79(31.6692) = 1.1166

Initial Ht. of voids, $H_v = H_i - H_s$ = 1.905 − 1.1166 = 0.7884

Initial degree of saturation, $S_i = (W_t - W_s)/(H_i - H_s)A$ = (119.0 − 98.66)/(0.7884)(31.6692) = 81.5

Initial void ratio $e_o = H_v/H_s$ = 0.7884 / 1.1166 = 0.70607

Final Test Data (obtained at end of load testing)

Initial dial reading: 0.0000

Final dial reading: 0.0907[5]

Change in sample ht.: 0.0907[5]

Final ht. of voids, H_{vf}: 0.5576[5]

Final void ratio, $e_f = H_{vf}/H_s$: 0.49941

Final water content determination

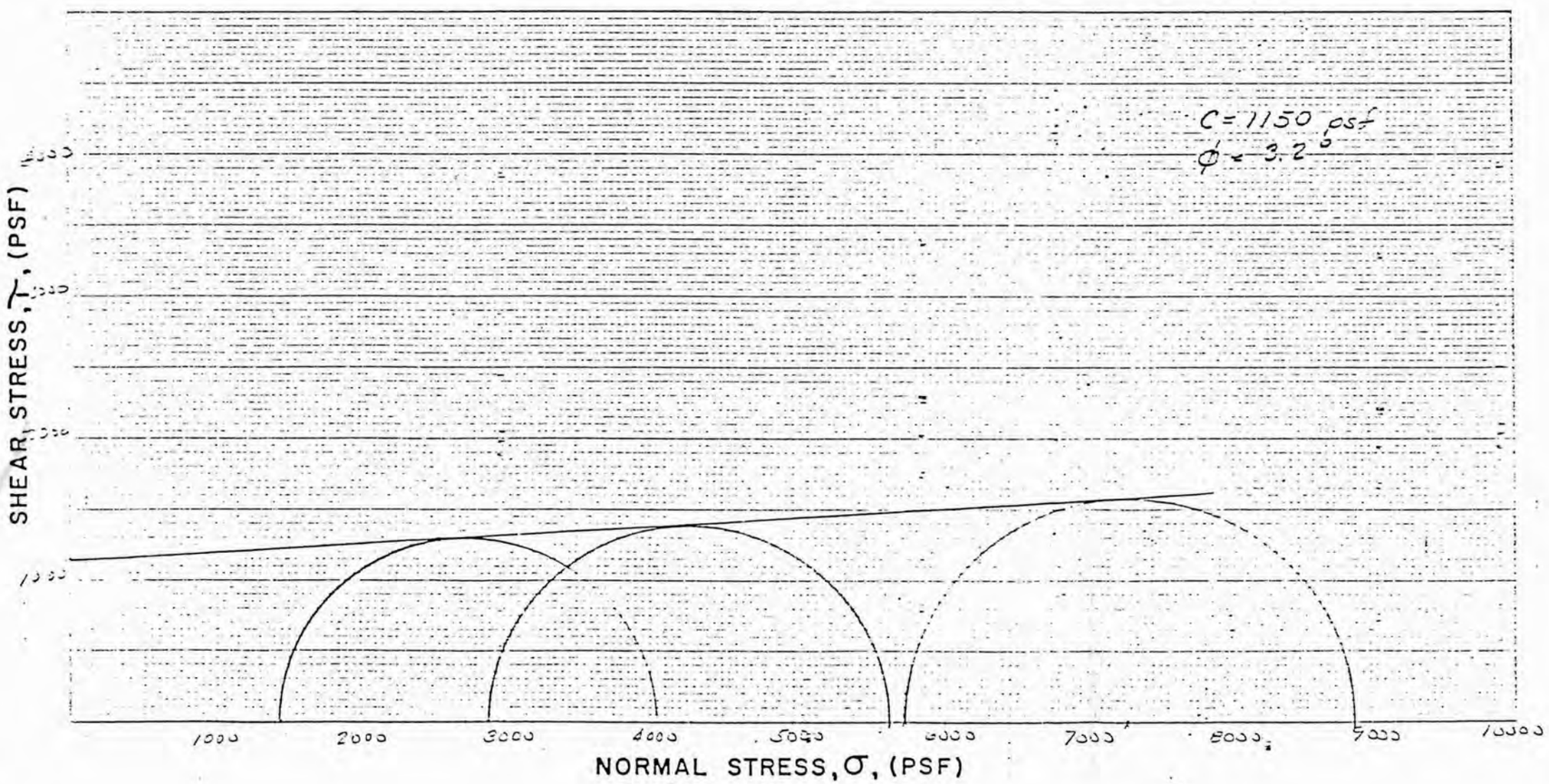
Final wet Wt. + ring[c]	722.9
Final dry wt. + ring	705.2 gr
ring	606.5 gr
Oven dry wt. of soil, W_s	98.7 gr
water	17.7 gr
Final water content, w_f	17.9 %
M/C =	
Final degree of sat. S	100 %

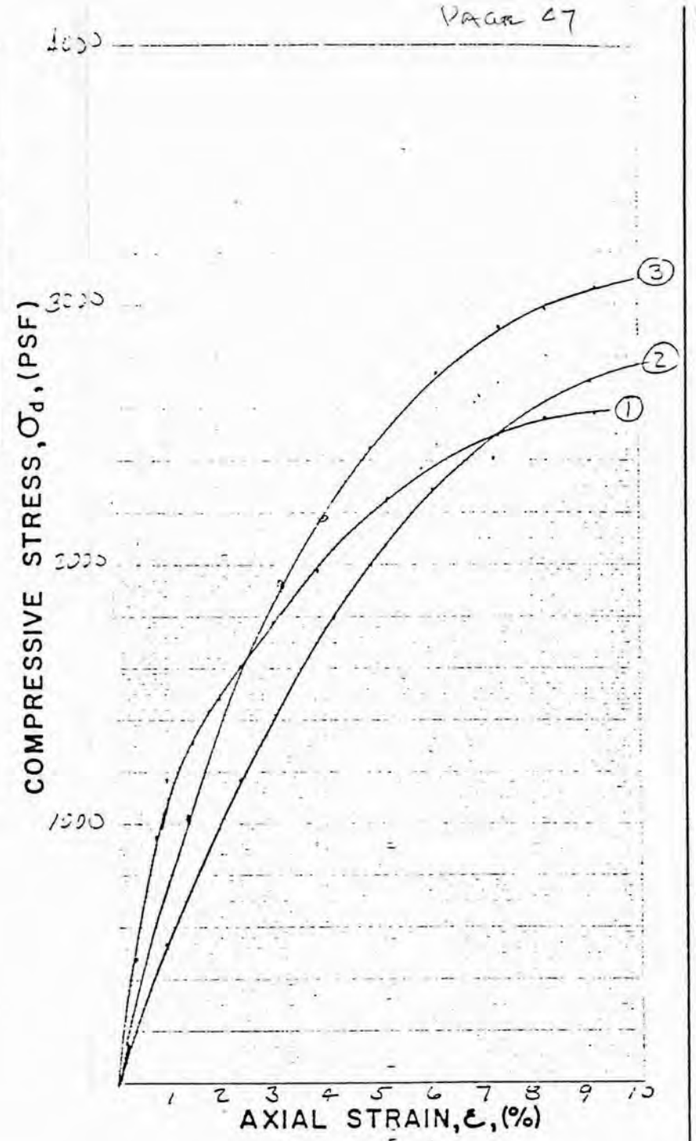
[a]Obtained from Final Test Data below.
[b]If it appears that any soil is lost from sample, use W_s'.
[c]Be sure to include any soil extruded from ring which is in consolidometer.

CURVE NO.	SAMPLE NO.	ELEV. OR DEPTH	CLASSIFICATION	WATER CONTENT (%) BEFORE	WATER CONTENT (%) AFTER	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTIC LIMIT	SPECIFIC GRAVITY	DRY DENSITY (PCF) BEFORE	DRY DENSITY (PCF) AFTER	STRESS (PSF)	STRAIN (%)	CELL PRESSURE (PSF)	C (PSF)	ϕ DEGREES	$\bar{C}$ (PSF)	$\bar{\phi}$ DEGREES
1	IMB-S2	Surface		19.6	—	"	"	—	100.5	—			1440	—	—	—	—
2	"	"		21.6	—	48	17	—	99.7	—			2880	1150	3.2	—	—
3	"	"		17.4	—	"	"	—	103.6	—			5760	—	—	—	—

All samples compacted at Modified Proctor Energy at natural moisture content

	% Max.
1	95.7
2	94.9
3	98.7





TRIAXIAL COMPRESSION TEST REPORT	
TYPE OF TEST Unconsolidated Undrained	
PROJECT Resource Management	
Puerto Rico	
B511K	
DATE Aug 19, 1985	JOB # 85B081-01

HART

Fred C. Hart Associates, Inc.