TRC Environmental Solutions Inc.

# CELL 12 DESIGN REPORT

## VOLUME II

*Prepared for*

**US Ecology, Inc.**

Beatty, Nevada

*Prepared by*

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Project No. 95-284
March 1996

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EXHIBIT C

CALCULATIONS

C.1 Slope Stability Final Configuration
C.2 Slope Stability Interim Configuration
C.3 Surficial Soil Improvements
C.4 Settlement
C.5 Liner Stresses
C.6 Anchor Trench
C.7 Sump and Riser Design
C.8 LCRS Flow Capacity and Pump Sizing
C.9 LDS Flow Capacity and Pump Sizing
C.10 Surface Water Management
C.11 Erosion Control

# C.1 SLOPE STABILITY FINAL CONFIGURATION

# ENVIRONMENTAL SOLUTIONS, INC.

By _____ Date _____ Subject _____ Sheet No. ____ of ____

Chkd. By ____ Date _____ _____ Proj. No. _____

BEATTY

DEEP SEATED

SLOPE STABILITY

FINAL CONFIGURATION



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By WB Date 1/10/96 Subject SLOPE STABILITY EVALUATION Sheet No. 1 of 14
Chkd. By JM Date 2/18/96 FINAL CONFIGURATION Proj. No. 95-284

SLOPE STABILITY - FINAL CONFIGURATIONS

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# ENVIRONMENTAL SOLUTIONS, INC.

By WS Date 1/10/96 Subject SLOPE STABILITY EVALUATION Sheet No. 2 of 14
Chkd. By MH Date 1/12/96 FINAL CONFIGURATION Proj. No. 95-284

## PURPOSE

EVALUATE THE STABILITY OF THE FINAL CELL 12 COVER CONFIGURATION FOR STATIC & SEISMIC CONDITIONS.

## REFERENCES


1. GRANT ENVIRONMENTAL, JULY 14, 1994, "GEOTECHNICAL INVESTIGATION FOR CELL 12 AT THE U.S. ECOLOGY HAZARDOUS WASTE MANAGEMENT FACILITY, BEATTY, NEVADA".

2. PURDUE UNIVERSITY, RONALD A. SIEGAL, "STABL USERS MANUAL", 1976

3. ENVIRONMENTAL SOLUTIONS INC. INTERNAL DATABASE OF LANDFILL PROJECTS STRENGTH PARAMETERS.

4. FRANKLIN & CHANG, NOVEMBER 1977, "PERMANENT DISPLACEMENTS OF EARTH EMBANKMENTS BY NEWMARK SLIDING BLOCK ANALYSIS" EARTHQUAKE RESISTANCE OF EARTH & ROCKFILL DAMS, REPORT 5, U.S. ARMY ENGINEERS WATERWAYS EXPERIMENT STATION.

5. DONOVAN, JULY 1983, "A PRACTITIONERS VIEW OF SITE EFFECTS ON STRONG GROUND MOTION", PROCEEDINGS OF CONFERENCE XXII ON SITE SPECIFIC EFFECTS OF SOIL & ROCK ON GROUND MOTION & THE IMPLICATIONS OF EARTHQUAKE RESISTANT DESIGN, USGS, SANTA FE, NEW MEXICO.




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# ENVIRONMENTAL SOLUTIONS, INC.

By UB Date 1/10/96 Subject SLOPE STABILITY EVALUATION Sheet No. 3 of 14
Chkd. By MJ Date 1/12/96 FINAL CONFIGURATION Proj. No. 95-234

ASSUMPTIONS

1. A CONCEPTUAL PROFILE OF THE FINAL CONFIGURATION IS PRESENTED ON FIGURE 1

2. THE WASTE MATERIAL HAS A SHEAR STRENGTH OF 27° FRICTION & 300 psf COHESION BASED ON THE PROPERTIES USED IN THE STABILITY ANALYSIS OF OTHER SIMILAR SITES (REFERENCE 3).

3. SURFICIAL SOILS ARE IMPROVED TO PROVIDE AT LEAST 1000 psf COHESION, AS NOTED ON FIGURE 1.

4. THE VELOCITY TO ACCELERATION RATIO FOR THE SITE IS ESTIMATED TO BE 40 (REFERENCE 5).

5. THREE CONCEPTS WILL BE EVALUATED; 1.) 2.5:1 SIDE SLOPES 2.) 3:1 SIDE SLOPES & 3.) 3:1 SIDE SLOPES WITH 2:1 BUTTRESS SLOPES.

6 THE MAXIMUM FILL HEIGHT WILL BE ABOUT 10' & THE ENTIRE BOTTOM WILL BE FILLED PRIOR TO NEXT LIFT

7 THE BUTTRESS FILL MATERIAL WILL ACHIEVE A COHESION VALUE OF AT LEAST 50psf. THIS COHESION VALUE WILL BE A RESULT OF 1.) COMPACTION EQUIPMENT THAT WILL CRUSH PARTICLES THEREBY INCREASING THE FINES CONTENT & 2.) THE COMPACTION PROCESS THAT WILL DEVELOP NEGATIVE PORE PRESSURE IN THE FILL MATERIAL.



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CHECKED BY JM 2/18/96
Prep. by WB 2/18/96



TABLE B. MATERIAL PROPERTIES

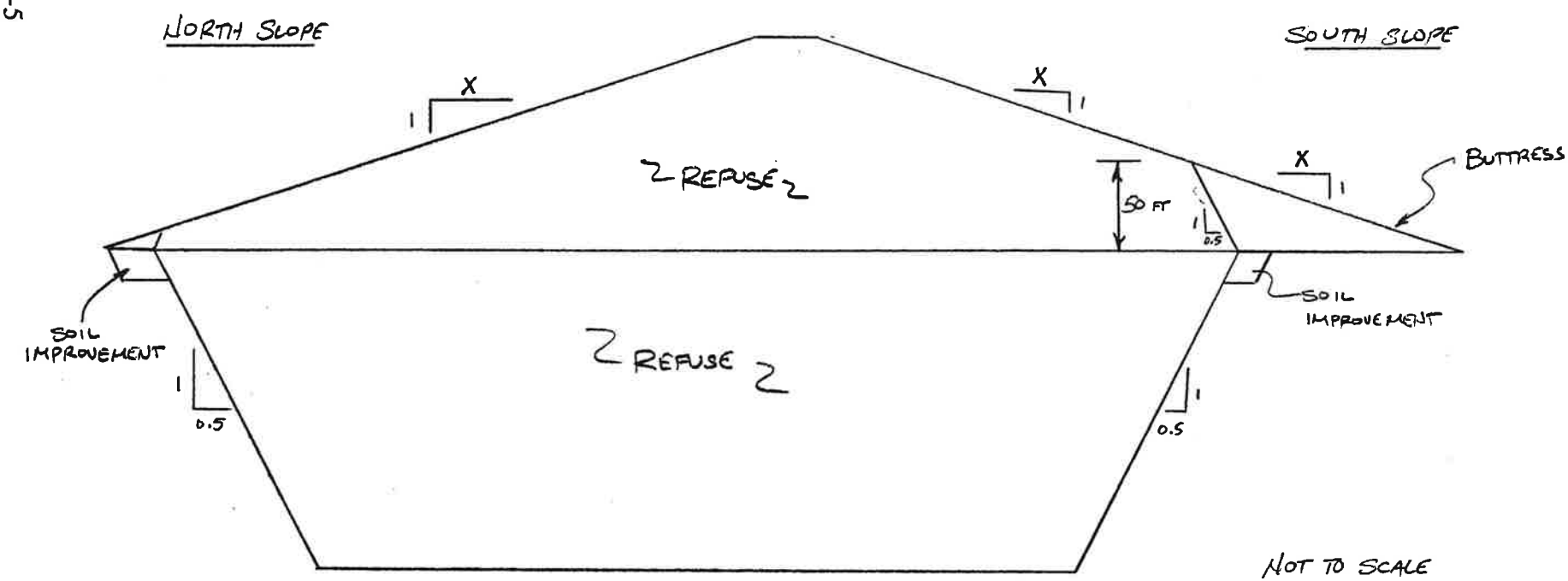
| MATERIAL | $\gamma$ (pcf) | c (psf) | $\phi$ |
|---|---|---|---|
| REFUSE | 115 | 300 | 27 |
| BUTTRESS | 120 | 50 | 35 |
| SOIL IMPROVEMENT | 120 | 1000 | 35 |
| NATIVE SOILS | 120 | 1000 | 35 |

FIGURE 1

FINAL CONFIGURATION

TABLE A. SLOPE STEEPNESS

| | CASE I | CASE II | CASE III |
|---|---|---|---|
| NORTH SLOPE | 2.5:1 | 3:1 | 3:1 |
| SOUTH SLOPE | 2.5:1 | 3:1 | 3:1 |
| BUTTRESS | 2.5:1 | 3:1 | 2:1 |



# ENVIRONMENTAL SOLUTIONS, INC.

By NB Date 1/10/96 Subject SLOPE STABILITY EVALUATION Sheet No. 5 of 14
Chkd. By JM Date 2/16/96 FINAL CONFIGURATION Proj. No. 95-284

## SEISMICITY

ATTACHMENT A PRESENTS THE PEAK HORIZONTAL ACCELERATION FOR THE BEATTY SITE. THE CONTROLLING FAULT IS FAULT A CAPABLE OF PRODUCING A 0.42g maximum ground ACCELERATION AT THE SITE. THEREFORE, FOR THE FINAL DESIGN ANALYSIS A .42g ACCELERATION IS THE DESIGN HORIZONTAL ACCELERATION.

## EVALUATION

TABLE 1 PRESENTS A SUMMARY OF THE STATIC FACTORS OF SAFETY FOR EACH OF THE FINAL DESIGN ALTERNATIVES (SEE FIGURE 1). THE COMPUTER OUTPUT & GRAPHICAL PLOT OF EACH OF THE CRITICAL FAILURE SURFACES IS PRESENTED IN ATTACHMENT B.

TABLE 2 PRESENTS A SUMMARY OF THE PSEUDOSTATIC FACTORS OF SAFETY FOR EACH OF THE DESIGN ALTERNATIVES. THE STABL5 COMPUTER OUTPUT & THE GRAPHICAL PLOT OF EACH OF THE CRITICAL FAILURE SURFACES IS PRESENTED IN ATTACHMENT C.

SINCE THE PSEUDOSTATIC FACTORS OF SAFETY IS LESS THAN 1.0 A PERMANENT DISPLACEMENT ANALYSIS NEEDS TO BE PERFORMED.



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TABLE 1. SUMMARY STATIC FACTORS OF SAFETY

| | SLOPE STEEPNESS | | |
|---|---|---|---|
| | 2.5:1 | 3:1 | 3:1 WITH 2:1 BUTTR. |
| NORTH SLOPE | 1.75 | 2.06 | - |
| SOUTH SLOPE | 1.82 | 2.22 | 1.71 |
| SOUTH SLOPE (buttress only) | 1.96 | 2.32 | 1.61 |

"Buttress only" indicates only the buttress was evaluated and not the entire slope.

TABLE 2. SUMMARY SEISMIC FACTORS OF SAFETY(1)

| | SLOPE STEEPNESS | | |
|---|---|---|---|
| | 2.5:1 | 3:1 | 3:1 WITH 2:1 BUTTR. |
| NORTH SLOPE | 0.73 | 0.81 | - |
| SOUTH SLOPE | 0.77 | 0.86 | 0.75 |
| SOUTH SLOPE (buttress only) | 0.82 | 0.90 | 0.72 |

"Buttress only" indicates only the buttress was evaluated and not the entire slope.

☐ - INDICATES SELECTED DESIGN CONFIGURATION

(1) A SEISMIC FACTOR OF SAFETY LESS THAN 1.0 INDICATES THAT A DISPLACEMENT ANALYSIS IS NECESSARY (SEE SHEETS 7 THROUGH 14).

## PERMANENT DISPLACEMENT

THE NEWMARK SLIDING BLOCK METHOD OF ESTIMATING SEISMICALLY INDUCED PERMANENT DISPLACEMENTS INVOLVES CALCULATING THE EARTHQUAKE INDUCED INERTIAL FORCE OF A POTENTIAL SLIDING MASS & COMPARING THIS FORCE WITH A RESISTING FORCE DUE TO THE SHEAR STRENGTH OF THE MATERIAL. A PERMANENT DISPLACEMENT OF THE SLIDING MASS OCCURS WHENEVER THE DRIVING FORCE EXCEEDS THE RESISTING FORCE.

THE DRIVING FORCE EQUALS THE SLIDING MASS TIMES THE ACCELERATION AT ANY POINT IN TIME. THEREFORE, IF THE AVERAGE ACCELERATION ACROSS A SLIDING MASS IS GREATER THAN THE YIELD ACCELERATION, MOVEMENT WILL OCCUR. MOVEMENT WILL STOP WHEN THE GROUND MOTION IS REVERSED & THE AVERAGE ACCELERATION BECOMES LESS THAN THE YIELD ACCELERATION.

THE YIELD ACCELERATION (OR SEISMIC COEFFICIENT) IS THE ACCELERATION AT WHICH THE POTENTIAL MASS WILL JUST BEGIN TO MOVE. (FS = 1.0). BY DOUBLE INTEGRATION OF THE AVERAGE ACCELERATION TIME HISTORY THAT EXCEEDS THE YIELD ACCELERATION, THE PERMANENT DISPLACEMENTS CAN BE ESTIMATED. THIS PROCEDURE IS PRESENTED AS FIGURE 2

THE STABLE5M COMPUTER OUTPUT WAS USED TO ESTIMATE THE SEISMIC COEFFICIENT ($K_y$) TO PRODUCE A FACTOR OF SAFETY OF 1.0 (YIELD ACCELERATION). THE YIELD ACCELERATIONS ARE PRESENTED IN TABLE 3 & THE STABL5 COMPUTER OUTPUTS FOR THE YIELD ACCELERATIONS ARE PRESENTED IN APPENDIX D.



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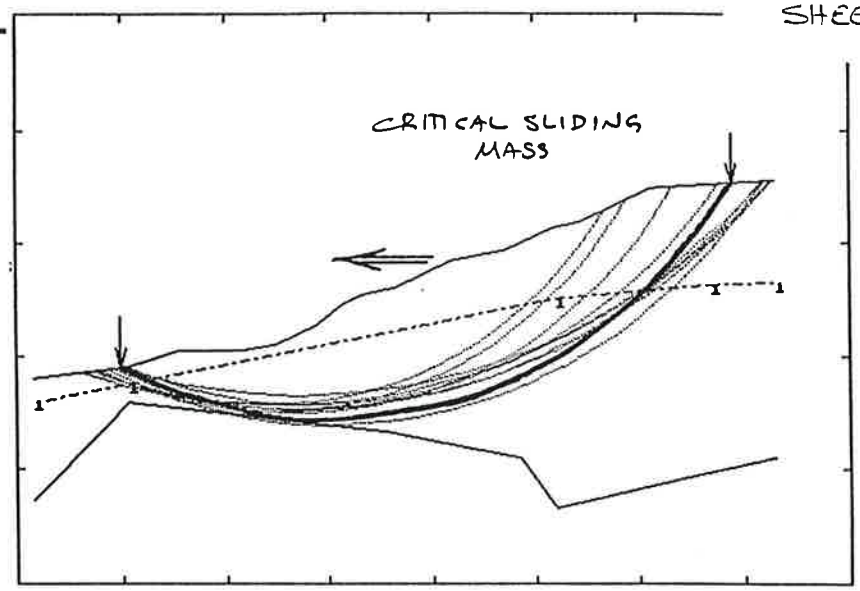
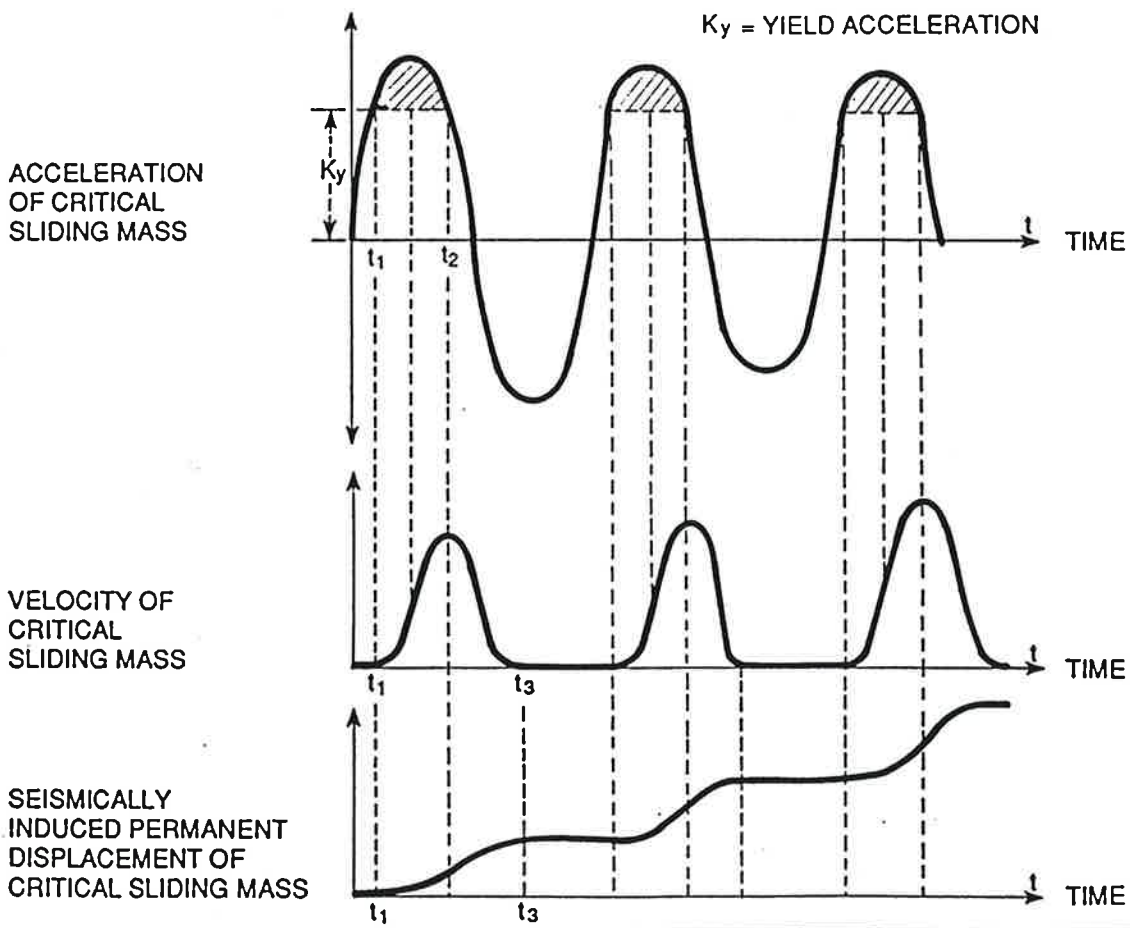



(EXAMPLE ONLY)




**SEISMIC DISPLACEMENT CALCULATION CONCEPT**

FIGURE 2

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FIGURE 3 PRESENTS THE RANGE OF VELOCITY TO ACCELERATION VALUES (V/A) FOR ROCK & SOIL SITES AS DEVELOPED BY DONOVAN (REFERENCE 5). AN ESTIMATE FOR THE V/A RATIO WITH A SOURCE 5-6 KILOMETERS FROM THE SITE IS 40 (SEE FIGURE 3).

THE RELATIONSHIP OF THE MAXIMUM GROUND ACCELERATION, MAXIMUM GROUND VELOCITY & STANDARDIZED MAXIMUM DISPLACEMENT TO THE UNSCALED PERMANENT DISPLACEMENT IS:

$$U_M = \frac{U_S V^2}{1800 A}$$

WHERE $U_M$ = UNSCALED DISPLACEMENT (IN)
$U_S$ = STANDARDIZED MAX. GROUND DISPLACEMENT (IN)
$A$ = MAX. GROUND ACCELERATION (g)
$V$ = MAX. GROUND VELOCITY (in/sec)

REFERENCE 4 HAS COMPILED A SERIES OF CURVES THAT RELATE THE RATIO OF YIELD ACCELERATION TO MAX. GROUND ACCELERATION TO A STANDARDIZED MAXIMUM DISPLACEMENT. THESE SERIES OF CURVES IS PRESENTED IN FIGURE 4.

FROM THE V/A RATIO (FIGURE 3) & THE STANDARDIZED MAX. GROUND DISPLACEMENT (FIGURE 4), AN UNSCALED PERMANENT DISPLACEMENT CAN BE OBTAINED (FROM THE ABOVE EQUATION). THIS UNSCALED PERMANENT DISPLACEMENT IS THE ESTIMATED MAXIMUM THE SLOPE SHOULD EXPERIENCE FROM A GROUND ACCELERATION OF VALUE "A". IN THIS CASE A=0.42g, WHICH IS THE MAXIMUM MEAN PEAK HORIZONTAL ACCELERATION THE SITE WILL EXPERIENCE.



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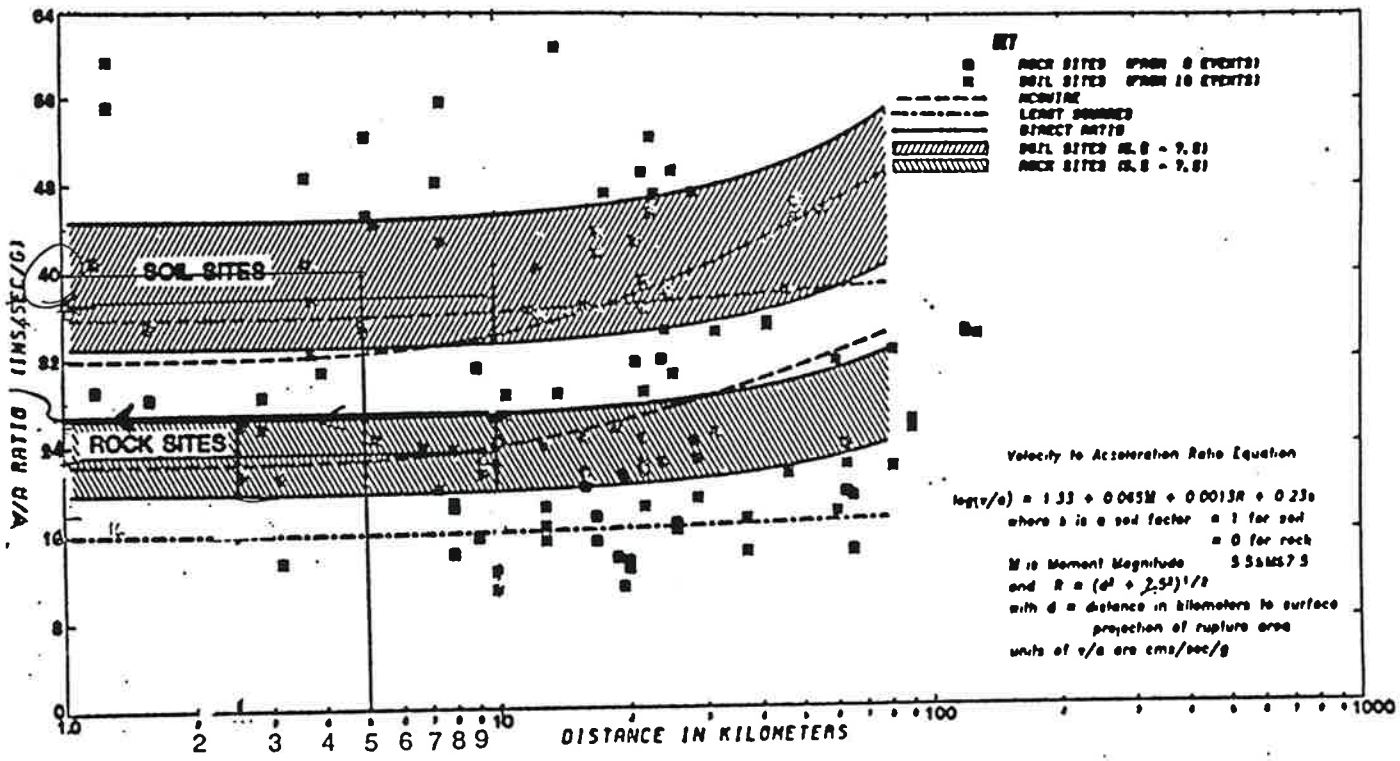


NOTE: SHADED AREAS SHOW RANGE OF VALUES OBTAINED USING EQUATION FOR ROCK AND SOIL SITES.

FIGURE 3

**MEASURED VELOCITY/ACCELERATION (V/A) RATIOS AND DERIVED V/A EQUATIONS**

ENVIRONMENTAL SOLU ONS, INC.

REFERENCE: DONOVAN, NEVILLE. "A PRACTITIONER'S VIEW OF SITE EFFECTS ON ST G GROUND MOTION."

SHEET 10 OF 14

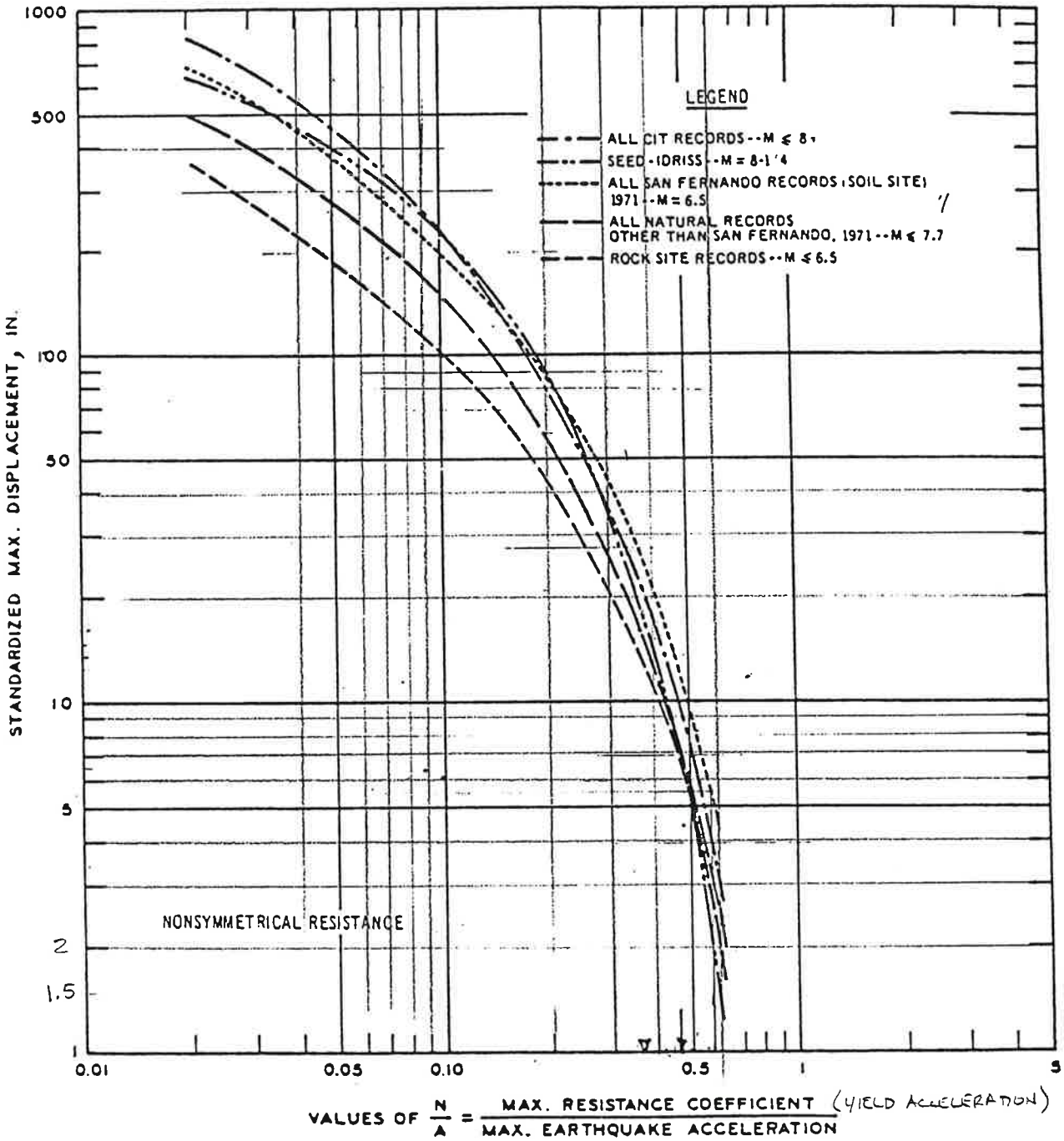


Figure 16. Upper bound envelope curves of permanent displacements for all natural and synthetic records analyzed

FIGURE 4
(REFERENCE 4)

# ENVIRONMENTAL SOLUTIONS, INC.

By WB Date 1/10/96 Subject SLOPE STABILITY EVALUATION Sheet No. 12 of 14
Chkd. By YM Date 2/18/96 FINAL CONFIGURATION Proj. No. 95-234

FROM TABLE 3, THE SUMMARY OF SEISMIC YIELD VALUES THE PERMANENT DISPLACEMENT CAN BE CALCULATED.

I) FOR 2.5:1 SLOPE THE MINIMUM YIELD ACCELERATION IS 0.24g

$$\frac{ky}{A} = \frac{0.24}{0.42} = 0.57$$ FROM FIGURE 4 $U_s = 6$ INCHES

$$U_m = \frac{U_s V^2}{1800 A}$$ FROM REFERENCE 4

WHERE $U_s$ = STANDARDIZED DISPLACEMENT (IN)
$U_m$ = UNSCALED PERMANENT DISPLACEMENT (IN)
$V$ = VELOCITY
$A$ = ACCELERATION

FROM REFERENCE 5, AN AVERAGE $V/A$ RATIO IS 40

$$\therefore \frac{V}{A} = 40 \qquad V = (40)(.42) = 16.8 \text{ IN/SEC}$$

$$U_m = \frac{(6)(16.8)^2}{(1800)(.42)} = 2.2 \text{ INCHES}$$

II) FOR A 3:1 SLOPE THE MINIMUM YIELD ACCELERATION IS 0.30g

$$\frac{ky}{A} = \frac{.30}{.42} = .71 \qquad U_s = 1.5 \text{ IN}$$

$$U_m = \frac{(1.5)(16.8)^2}{(1800)(.42)} = 0.6 \text{ IN.}$$



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# ENVIRONMENTAL SOLUTIONS, INC.

By WB Date 1/10/96 Subject SLOPE STABILITY EVALUATION Sheet No. 13 of 14
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III FOR THE 2:1 BUTTRESS SLOPE, THE MINIMUM YIELD ACCELERATION IS 0.23g.

$\frac{k_y}{A} = \frac{.23}{.42} = .55$ $U_s = 7$ IN. $U_m = \frac{(7)(16.8)^2}{1800(.42)} = 2.6$ IN.

TABLE 4 PRESENTS A SUMMARY OF PERMANENT DISPLACEMENTS ESTIMATED BY THE METHOD PREVIOUSLY DESCRIBED

## CONCLUSION

THE STATIC FACTORS OF SAFETY ARE ALL ABOVE THE ACCEPTED 1.5 MINIMUM FACTOR OF SAFETY. HOWEVER, THE SEISMIC ACCELERATIONS ARE ALL LESS THAN 1.0, WHICH INDICATES SOME MOVEMENT OR PERMANENT DISPLACEMENT WILL OCCUR FOR THE HORIZONTAL ACCELERATION OF 0.42g. IN EVALUATING THE PERMANENT DISPLACEMENTS (TABLE 4), THE LARGEST ESTIMATED DISPLACEMENT IS ONLY 2.6 INCHES. GENERALLY AN ESTIMATED DISPLACEMENT OF LESS THAN 12 INCHES INDICATES THE SLOPE SHOULD NOT FAIL DURING THE DESIGN EVENT. THE BUTTRESS FILL SOILS SHOULD BE TESTED PRIOR TO CONSTRUCTION TO VERIFY SOIL STRENGTHS.



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TABLE 3. SUMMARY SEISMIC YIELD VALUES

| | SLOPE STEEPNESS | | |
|---|---|---|---|
| | 2.5:1 | 3:1 | 3:1 WITH 2:1 BUTTR. |
| NORTH SLOPE | .24g | .30g | --- |
| SOUTH SLOPE | .27g | .34g | .24g |
| SOUTH SLOPE (buttress only) | .30g | .36g | .23g |

"Buttress only" indicates that only the buttress was evaluated and not the entire slope.

TABLE 4. SUMMARY PERMANENT DISPLACEMENTS (inches)

| | SLOPE STEEPNESS | | |
|---|---|---|---|
| | 2.5:1 | 3:1 | 3:1 WITH 2:1 BUTTR. |
| NORTH SLOPE | 2.2 | 0.6 | --- |
| SOUTH SLOPE | 1.1 | 0.5 | 2.2 |
| SOUTH SLOPE (buttress only) | 0.75 | 0.3 | 2.6 |

"Buttress only" indicates that only the buttress was evaluated and not the entire slope.

☐ - INDICATES SELECTED DESIGN CONFIGURATION

# ENVIRONMENTAL SOLUTIONS, INC.

By WB Date 1/10/96 Subject SLOPE STABILITY EVALUATION Sheet No. ____ of ____

Chkd. By JA Date 2/18/96 FINAL CONFIGURATION Proj. No. 95-234

ATTACHMENT A

PEAK HORIZONTAL ACCELERATION



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## 1.0 OBJECTIVE

The objective of this calculation is to estimate the peak horizontal ground accelerations (PHGA) at the Beatty Landfill site in southwestern Nevada (see Figure 1). The Beatty Landfill is a hazardous waste Class I facility owned and operated by American Ecology Corporation. The ground motions resulting from this calculation will be used as design parameters for landfill cell expansions.

## 2.0 ASSUMPTIONS

Fault Identification:
Use Jennings, *Fault Activity Map of California and Adjacent Areas,* DMG Geologic Map No. 6, 1994
Cross check with Maulchin and Jones, *Peak Acceleration from Maximum Credible Earthquakes in California,* DMG Open-File Report 92-1, 1992.

Maximum Credible Earthquakes:
Use Wells and Coppersmith, *New Empirical Relationships among Magnitude, Rupture Length, Rupture Width, Rupture Area, and Surface Displacement,* Geomatrix Consultants, Inc., 1993.

PHGA:
Use Maulchin and Jones, *Peak Acceleration from Maximum Credible Earthquakes in California,* DMG Open-File Report 92-1, 1992.

## 3.0 METHODOLOGY

The calculation of PHGA for the Beatty Landfill was completed using a deterministic approach. The calculation steps are outlined below:

I. Fault Identification - use fault maps to identify capable faults within a 100-km radius of the site.
II. Assign Maximum Credible Earthquakes (MCE) - use magnitude - surface rupture relationships to estimate MCE.
III. Fault Screening - screen the identified faults for further analysis by 1) grouping them by site to fault distance and 2) fault MCE.
IV. Estimate PHGA - use attenuation curves to determine ground motions from screened faults.

## 4.0 CALCULATION

### I. Fault Identification

Figures 2a and 2b show the identified capable faults within 100-km of the site. Table 1 lists the faults and fault parameters. Fault lengths and fault - site distance are measured from the surface traces shown on Figures 2a and 2b.




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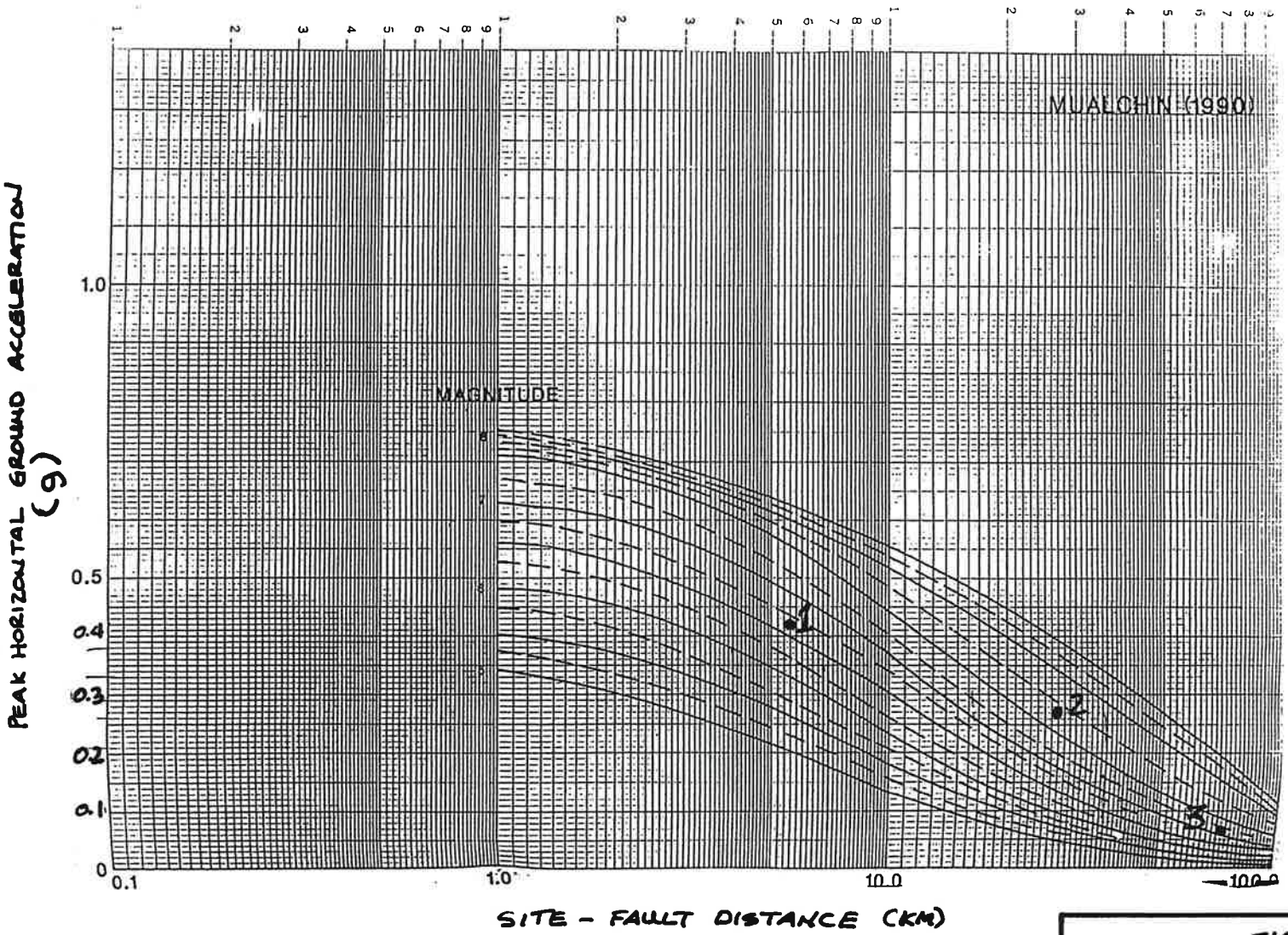



BY: RP 2-29-96

CK: JWD 3-6-96

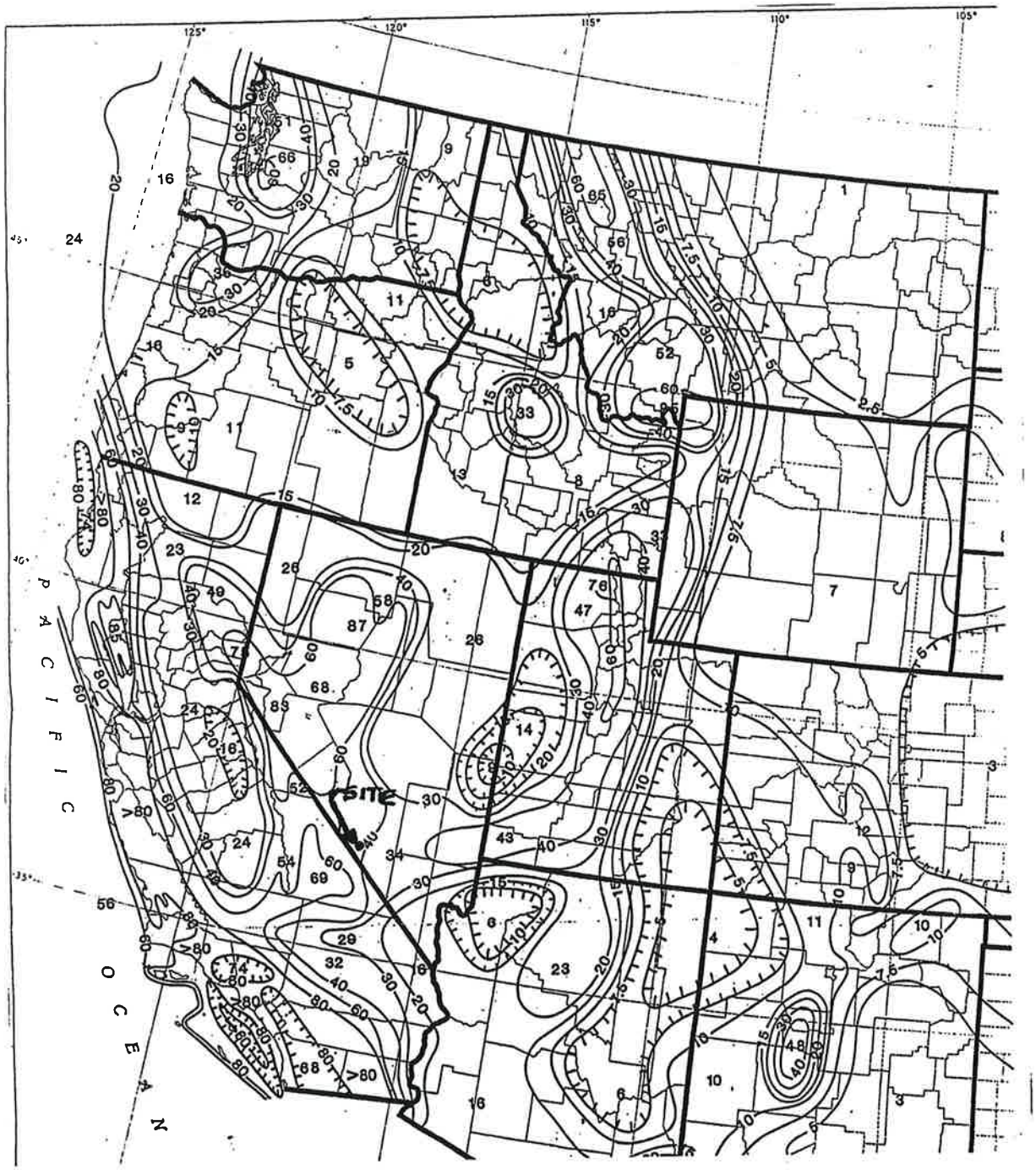
FIGURE 3

ATTENUATION CURVES (MUALCHIN & JONES, 1992)

BEATTY LANDFILL

ENVIRONMENTAL SOLUTIONS, INC.

REF: MAULCHIN & JONES, Peak Accelerations from MCE in California, DMG OPEN-FILE REPORT 92-1, 1992. CURVES ORIGINALLY PUBLISHED BY MUALCHIN, 1990.




BY: RP 2-29-96

CK: AWB 3-6-96

FIGURE 4

PROBABILISTIC EARTHQUAKE ACCELERATION CONTOURS (250-YR RECURRANCE)

BENTTY LANDFILL

REF: ALGERMISSEN, et.al., PROBABILISTIC EARTHQUAKE ACCELERATION AND VELOCITY MAPS...., USGS MISC. FIELD MAP MF2120, 1990.

ENVIRONMENTAL SOLUTIONS, INC.

# ENVIRONMENTAL SOLUTIONS, INC.

By WB Date 1/10/96 Subject SLOPE STABILITY EVALUATION Sheet No. ____ of ____

Chkd. By MH Date 1/12/96 FINAL CONFIGURATION Proj. No. 95-284

ATTACHMENT B

STABLSM COMPUTER OUTPUT

STATIC CASE



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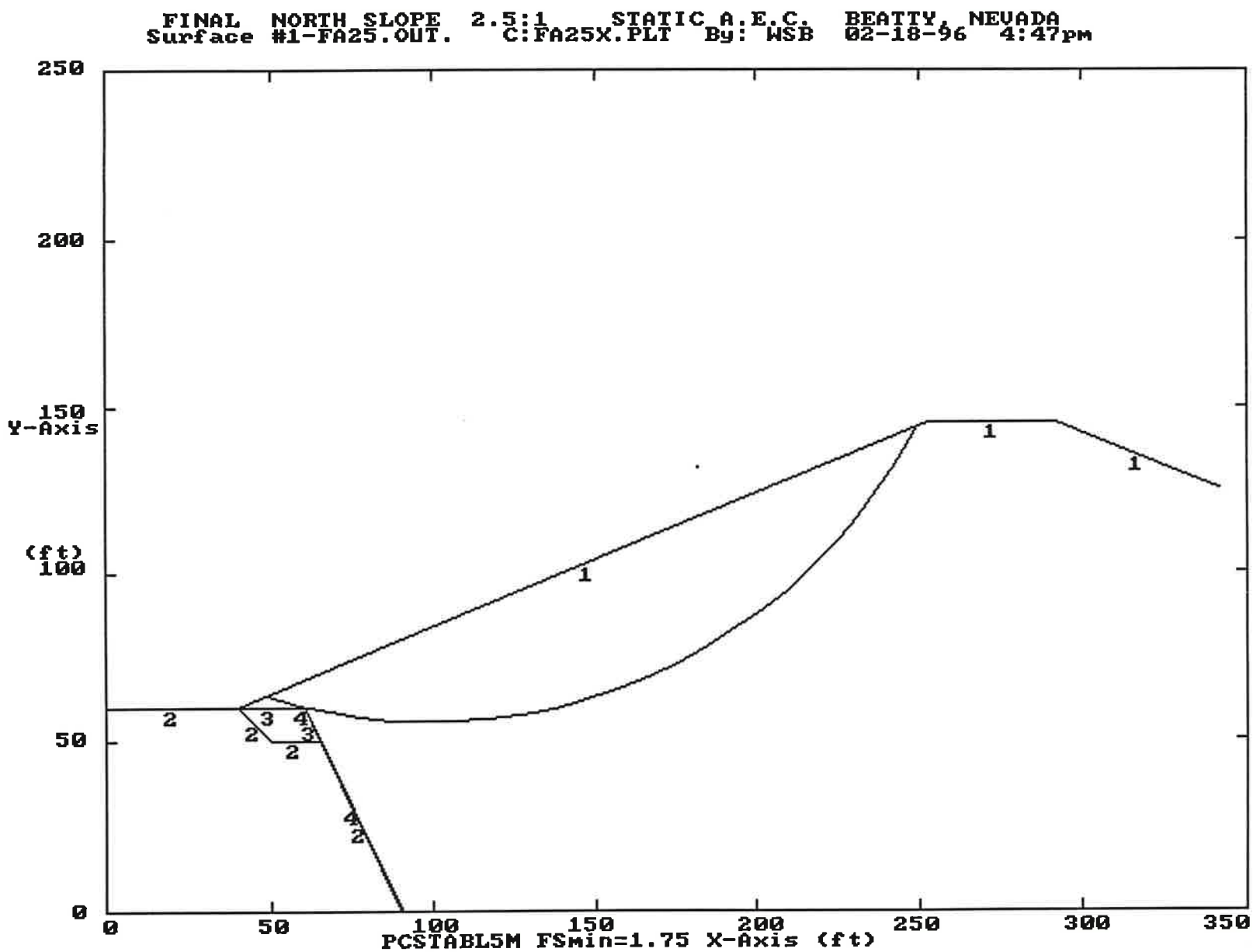

FINAL NORTH SLOPE 2.5:1 STATIC A.E.C. BEATTY, NEVADA
Surface #1-FA25.OUT. C:FA25X.PLT By: WSB 02-18-96 4:47pm
250
200
150
Y-Axis
(ft)
100
50
0
0
50
100
150
200
250
300
350
PCSTABL5M FSmin=1.75 X-Axis (ft)
1
1
1
2
2
2
2
3
3
4
4

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 02-18-96 |
| Time of Run: | 4:47pm |
| Run By: | WSB |
| Input Data Filename: | C:FA25X |
| Output Filename: | C:FA25X.OUT |
| Plotted Output Filename: | C:FA25X.PLT |

PROBLEM DESCRIPTION FINAL NORTH SLOPE 2.5:1 STATIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

4 Top Boundaries
11 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 2 |
| 2 | 40.00 | 60.00 | 252.50 | 145.00 | 1 |
| 3 | 252.50 | 145.00 | 292.50 | 145.00 | 1 |
| 4 | 292.50 | 145.00 | 342.50 | 125.00 | 1 |
| 5 | 40.00 | 60.00 | 60.00 | 60.00 | 3 |
| 6 | 60.00 | 60.00 | 60.20 | 60.00 | 4 |
| 7 | 40.00 | 60.00 | 50.00 | 50.00 | 2 |
| 8 | 60.20 | 60.00 | 90.20 | .00 | 4 |
| 9 | 60.00 | 60.00 | 65.00 | 50.00 | 3 |
| 10 | 50.00 | 50.00 | 65.00 | 50.00 | 2 |
| 11 | 65.00 | 50.00 | 90.00 | .00 | 2 |

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |

1

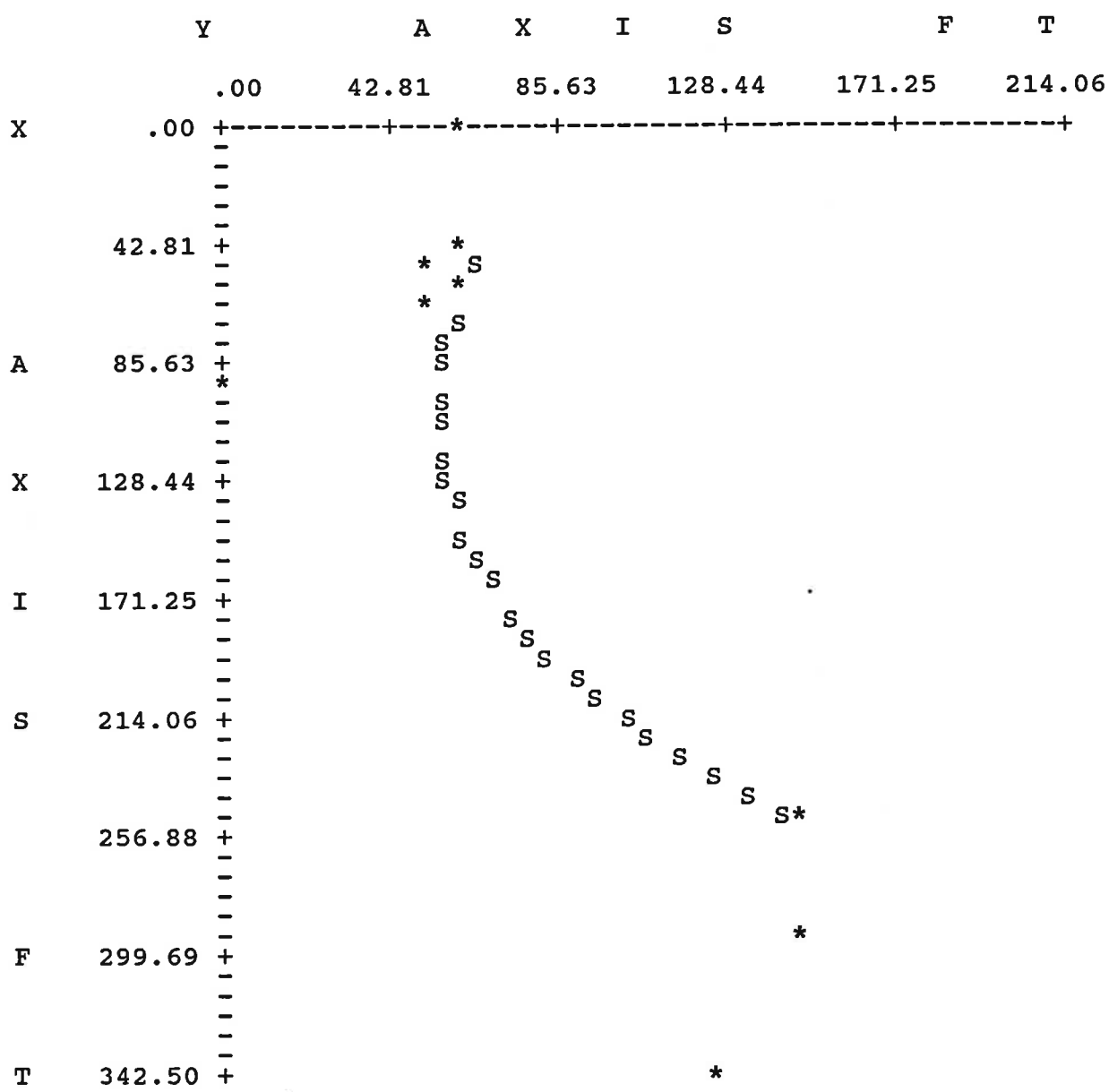
Trial Failure Surface Specified By 25 Coordinate Points

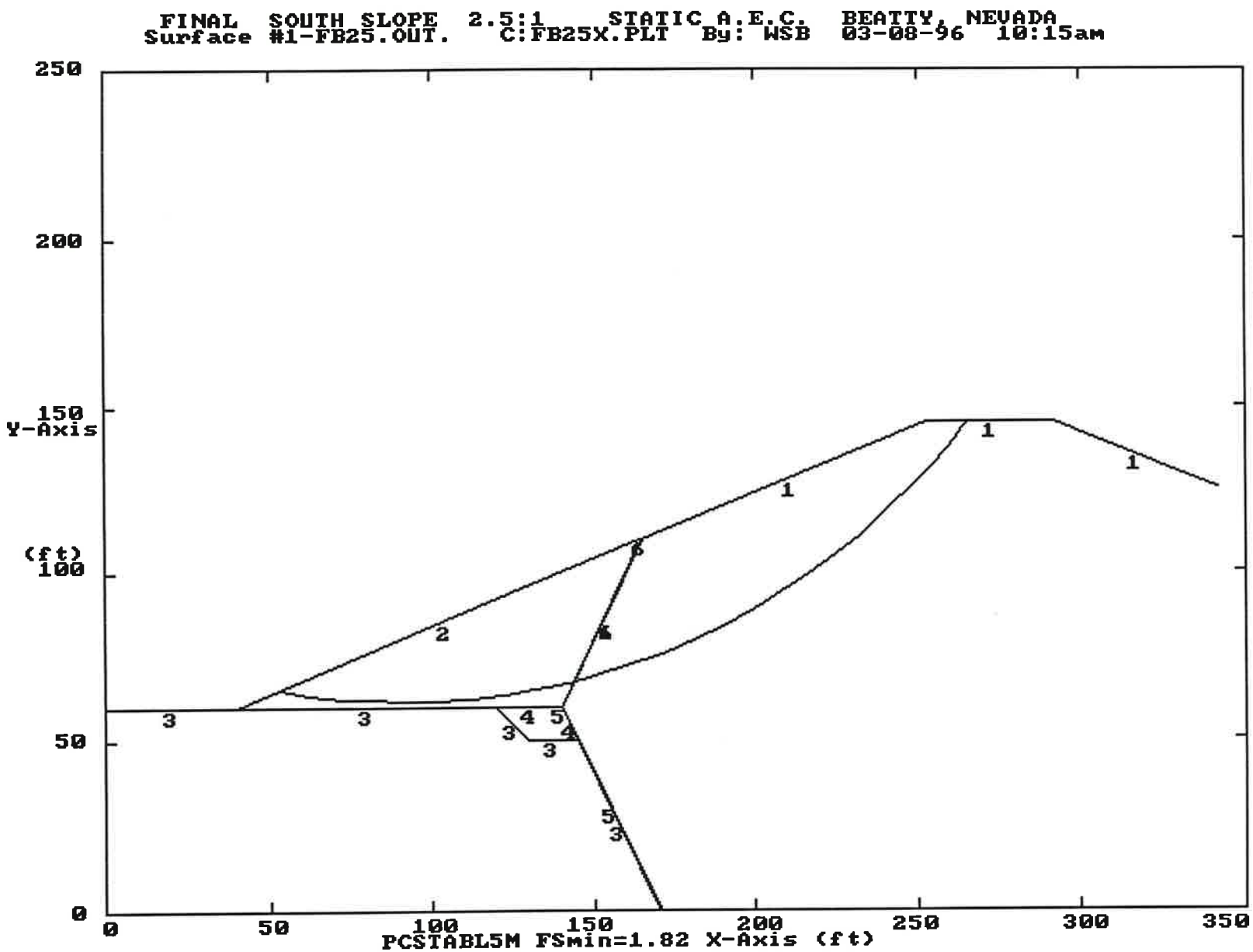
| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 49.47 | 63.79 |
| 2 | 59.06 | 60.96 |
| 3 | 68.81 | 58.70 |
| 4 | 78.67 | 57.04 |
| 5 | 88.61 | 55.96 |
| 6 | 98.60 | 55.49 |
| 7 | 108.60 | 55.61 |
| 8 | 118.57 | 56.33 |
| 9 | 128.48 | 57.64 |
| 10 | 138.30 | 59.55 |
| 11 | 147.99 | 62.04 |
| 12 | 157.51 | 65.10 |

| | | |
|---|---|---|
| 13 | 166.82 | 68.73 |
| 14 | 175.91 | 72.91 |
| 15 | 184.73 | 77.63 |
| 16 | 193.25 | 82.86 |
| 17 | 201.44 | 88.60 |
| 18 | 209.27 | 94.82 |
| 19 | 216.72 | 101.49 |
| 20 | 223.75 | 108.60 |
| 21 | 230.35 | 116.11 |
| 22 | 236.49 | 124.01 |
| 23 | 242.14 | 132.26 |
| 24 | 247.29 | 140.83 |
| 25 | 248.66 | 143.46 |

Factor Of Safety For The Preceding Specified Surface = 1.745

1




FINAL SOUTH SLOPE 2.5:1 STATIC A.E.C. BEATTY, NEVADA
Surface #1-FB25.OUT. C:FB25X.PLT By: WSB 03-08-96 10:15am
Y-Axis
(ft)
250
200
150
100
50
0
0
50
100
150
200
250
300
350
PCSTABL5M FSmin=1.82 X-Axis (ft)

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:15am |
| Run By: | WSB |
| Input Data Filename: | C:FB25X |
| Output Filename: | C:FB25X.OUT |
| Plotted Output Filename: | C:FB25X.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 2.5:1 STATIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 3 |
| 2 | 40.00 | 60.00 | 165.00 | 110.00 | 2 |
| 3 | 165.00 | 110.00 | 165.20 | 110.10 | 6 |
| 4 | 165.20 | 110.10 | 252.50 | 145.00 | 1 |
| 5 | 252.50 | 145.00 | 292.50 | 145.00 | 1 |
| 6 | 292.50 | 145.00 | 342.50 | 125.00 | 1 |
| 7 | 140.00 | 60.00 | 165.00 | 110.00 | 6 |
| 8 | 140.20 | 60.00 | 165.20 | 110.10 | 1 |
| 9 | 40.00 | 60.00 | 120.00 | 60.00 | 3 |
| 10 | 120.00 | 60.00 | 140.00 | 60.00 | 4 |
| 11 | 140.00 | 60.00 | 140.20 | 60.00 | 5 |
| 12 | 120.00 | 60.00 | 130.00 | 50.00 | 3 |
| 13 | 140.20 | 60.00 | 170.20 | .00 | 5 |
| 14 | 140.00 | 60.00 | 145.00 | 50.00 | 4 |
| 15 | 130.00 | 50.00 | 145.00 | 50.00 | 3 |
| 16 | 145.00 | 50.00 | 170.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | .5 | .5 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | .5 | .5 | .0 | 14.0 | .00 | .0 | 0 |

1

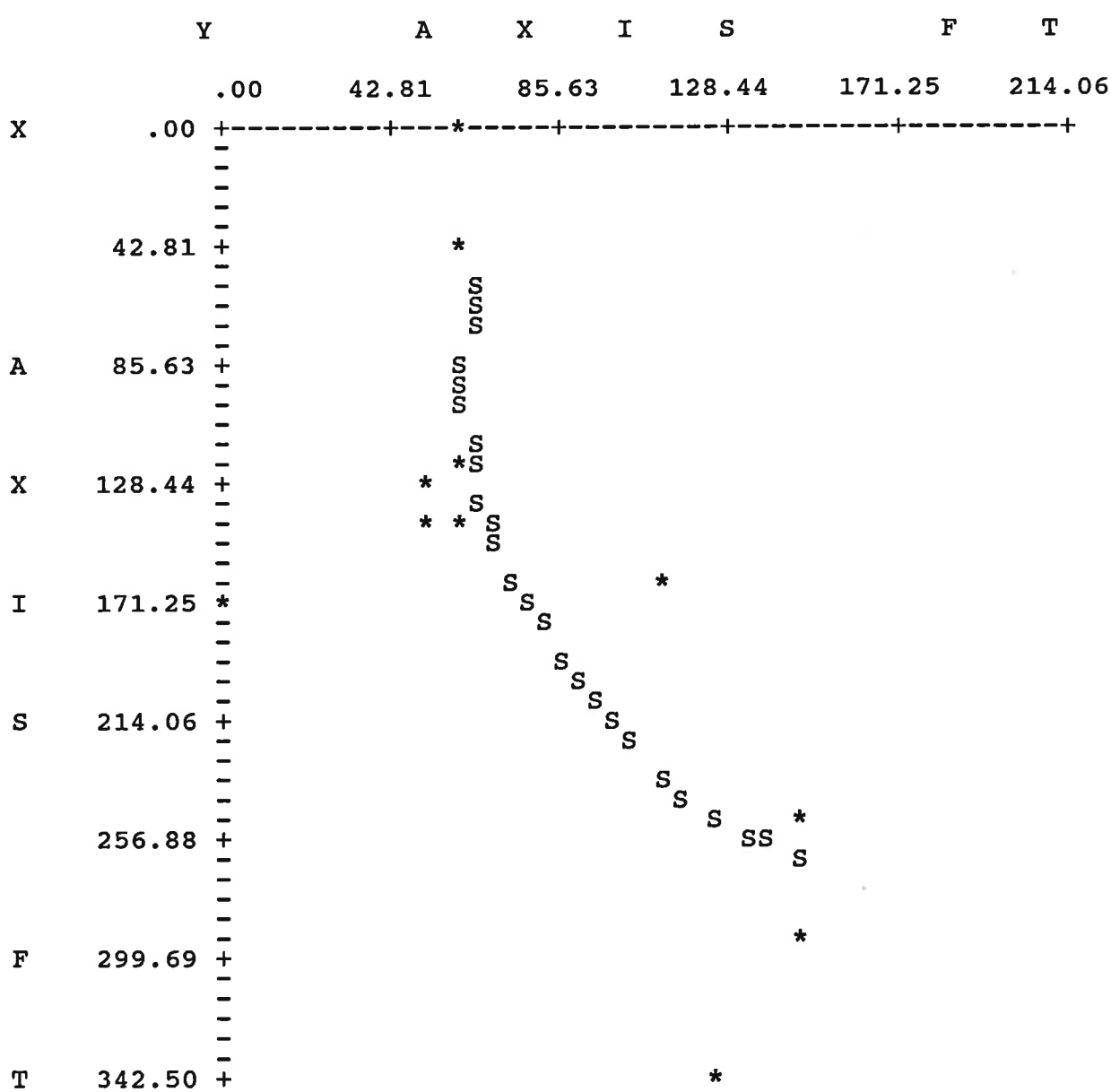
Trial Failure Surface Specified By 25 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 53.68 | 65.47 |
| 2 | 63.55 | 63.83 |
| 3 | 73.48 | 62.64 |
| 4 | 83.45 | 61.91 |
| 5 | 93.45 | 61.64 |

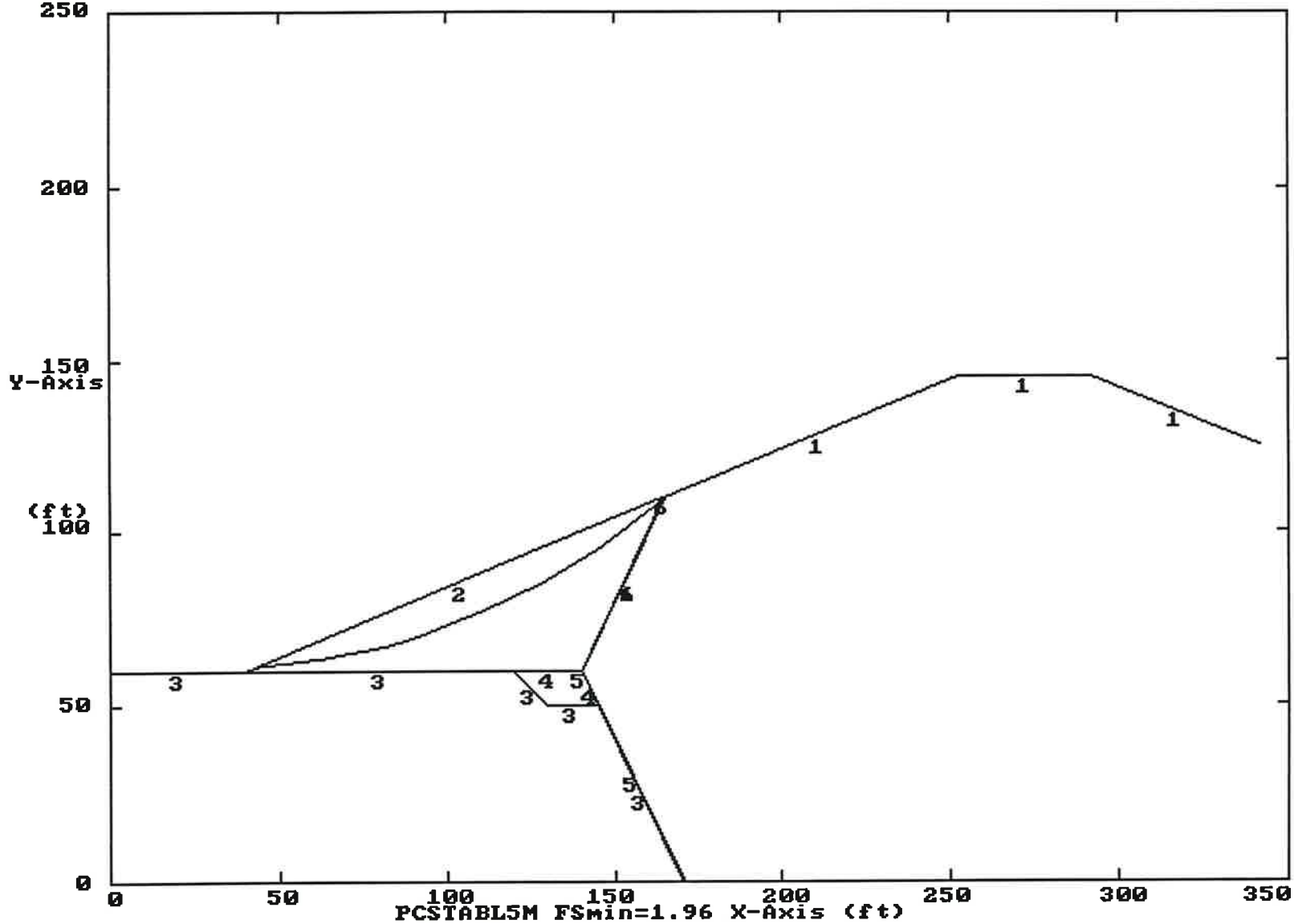
| | | |
|---|---|---|
| 6 | 103.45 | 61.84 |
| 7 | 113.42 | 62.49 |
| 8 | 123.36 | 63.61 |
| 9 | 133.24 | 65.18 |
| 10 | 143.03 | 67.20 |
| 11 | 152.72 | 69.68 |
| 12 | 162.28 | 72.60 |
| 13 | 171.70 | 75.96 |
| 14 | 180.96 | 79.75 |
| 15 | 190.03 | 83.96 |
| 16 | 198.89 | 88.58 |
| 17 | 207.53 | 93.61 |
| 18 | 215.94 | 99.04 |
| 19 | 224.08 | 104.84 |
| 20 | 231.94 | 111.02 |
| 21 | 239.52 | 117.55 |
| 22 | 246.78 | 124.42 |
| 23 | 253.72 | 131.63 |
| 24 | 260.32 | 139.14 |
| 25 | 265.00 | 145.00 |

Factor Of Safety For The Preceding Specified Surface = 1.823

1



FINAL SOUTH SLOPE 2.5:1 STATIC A.E.C. BEATTY, NEVADA
Surface #1-FB25A.OUT. C:FB25AX.PLT By: WSB 03-08-96 10:16am



** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:16am |
| Run By: | WSB |
| Input Data Filename: | C:FB25AX |
| Output Filename: | C:FB25AX.OUT |
| Plotted Output Filename: | C:FB25AX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 2.5:1 STATIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 3 |
| 2 | 40.00 | 60.00 | 165.00 | 110.00 | 2 |
| 3 | 165.00 | 110.00 | 165.20 | 110.10 | 6 |
| 4 | 165.20 | 110.10 | 252.50 | 145.00 | 1 |
| 5 | 252.50 | 145.00 | 292.50 | 145.00 | 1 |
| 6 | 292.50 | 145.00 | 342.50 | 125.00 | 1 |
| 7 | 140.00 | 60.00 | 165.00 | 110.00 | 6 |
| 8 | 140.20 | 60.00 | 165.20 | 110.10 | 1 |
| 9 | 40.00 | 60.00 | 120.00 | 60.00 | 3 |
| 10 | 120.00 | 60.00 | 140.00 | 60.00 | 4 |
| 11 | 140.00 | 60.00 | 140.20 | 60.00 | 5 |
| 12 | 120.00 | 60.00 | 130.00 | 50.00 | 3 |
| 13 | 140.20 | 60.00 | 170.20 | .00 | 5 |
| 14 | 140.00 | 60.00 | 145.00 | 50.00 | 4 |
| 15 | 130.00 | 50.00 | 145.00 | 50.00 | 3 |
| 16 | 145.00 | 50.00 | 170.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

1

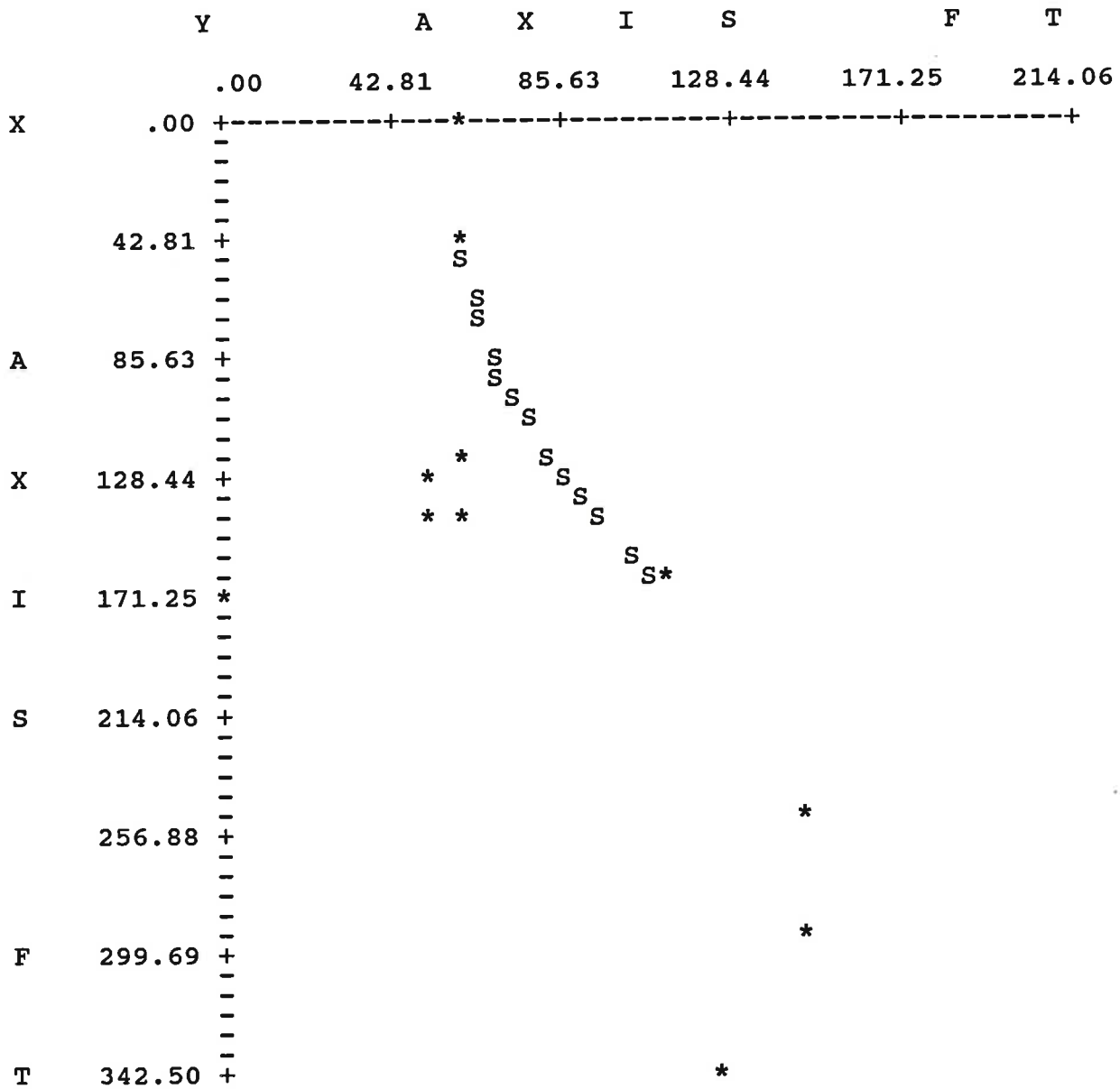
Trial Failure Surface Specified By 15 Coordinate Points

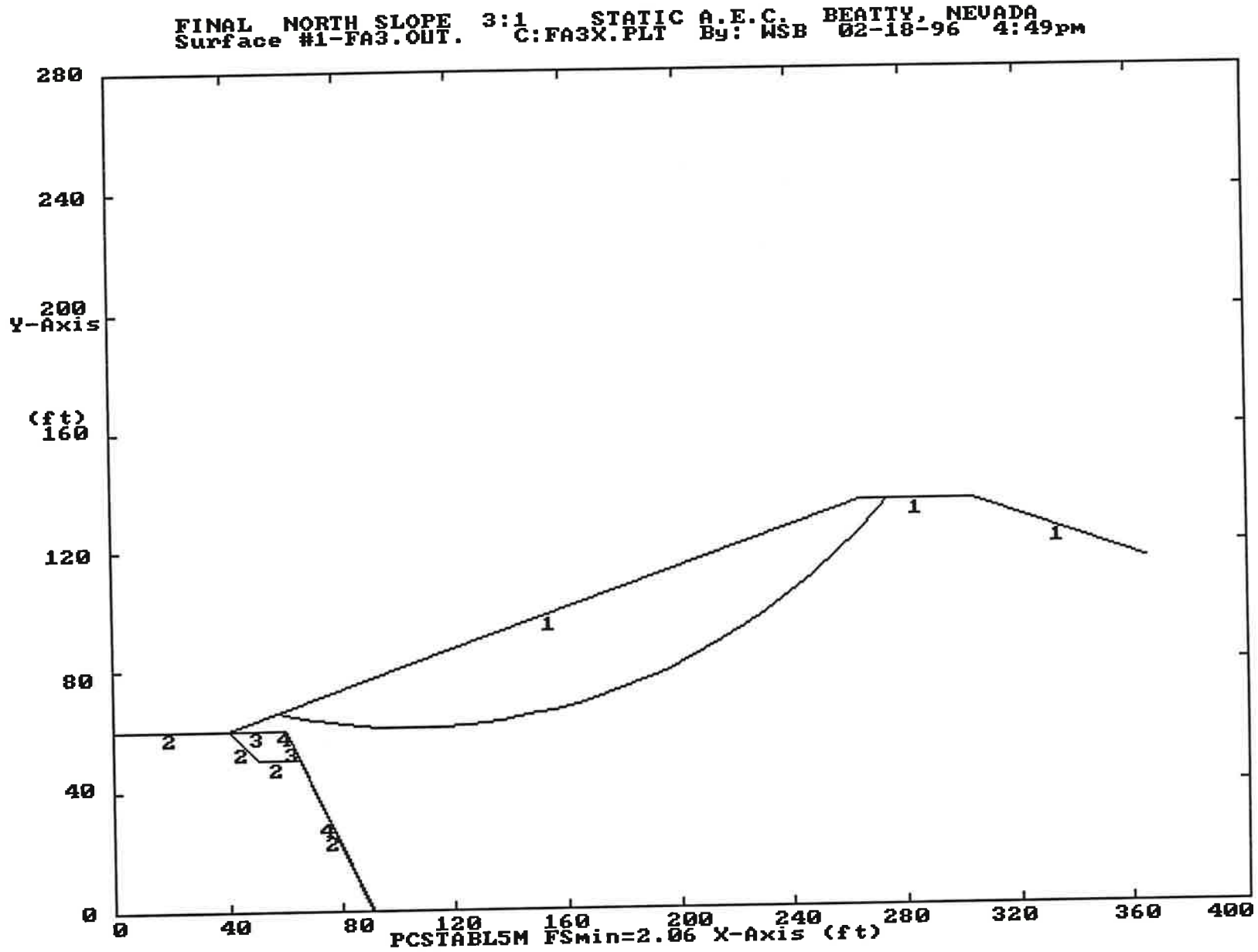
| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 42.63 | 61.05 |
| 2 | 52.59 | 61.93 |
| 3 | 62.50 | 63.28 |
| 4 | 72.34 | 65.09 |
| 5 | 82.08 | 67.36 |

| | | |
|---|---|---|
| 6 | 91.70 | 70.09 |
| 7 | 101.18 | 73.26 |
| 8 | 110.50 | 76.88 |
| 9 | 119.64 | 80.93 |
| 10 | 128.58 | 85.41 |
| 11 | 137.30 | 90.31 |
| 12 | 145.79 | 95.61 |
| 13 | 154.01 | 101.30 |
| 14 | 161.95 | 107.37 |
| 15 | 165.20 | 110.10 |

Factor Of Safety For The Preceding Specified Surface = 1.957

1




FINAL NORTH SLOPE 3:1 STATIC A.E.C. BEATTY, NEVADA
Surface #1-FA3.OUT. C:FA3X.PLT By: WSB 02-18-96 4:49pm
Y-Axis (ft)
X-Axis (ft)
PCSTABL5M FSmin=2.06
280
240
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
360
400
1
2
3
4

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 02-18-96 |
| Time of Run: | 4:49pm |
| Run By: | WSB |
| Input Data Filename: | C:FA3X |
| Output Filename: | C:FA3X.OUT |
| Plotted Output Filename: | C:FA3X.PLT |

PROBLEM DESCRIPTION FINAL NORTH SLOPE 3:1 STATIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

4 Top Boundaries
11 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 2 |
| 2 | 40.00 | 60.00 | 265.00 | 135.00 | 1 |
| 3 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 4 | 305.00 | 135.00 | 365.00 | 115.00 | 1 |
| 5 | 40.00 | 60.00 | 60.00 | 60.00 | 3 |
| 6 | 60.00 | 60.00 | 60.20 | 60.00 | 4 |
| 7 | 40.00 | 60.00 | 50.00 | 50.00 | 2 |
| 8 | 60.20 | 60.00 | 90.20 | .00 | 4 |
| 9 | 60.00 | 60.00 | 65.00 | 50.00 | 3 |
| 10 | 50.00 | 50.00 | 65.00 | 50.00 | 2 |
| 11 | 65.00 | 50.00 | 90.00 | .00 | 2 |

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |

1

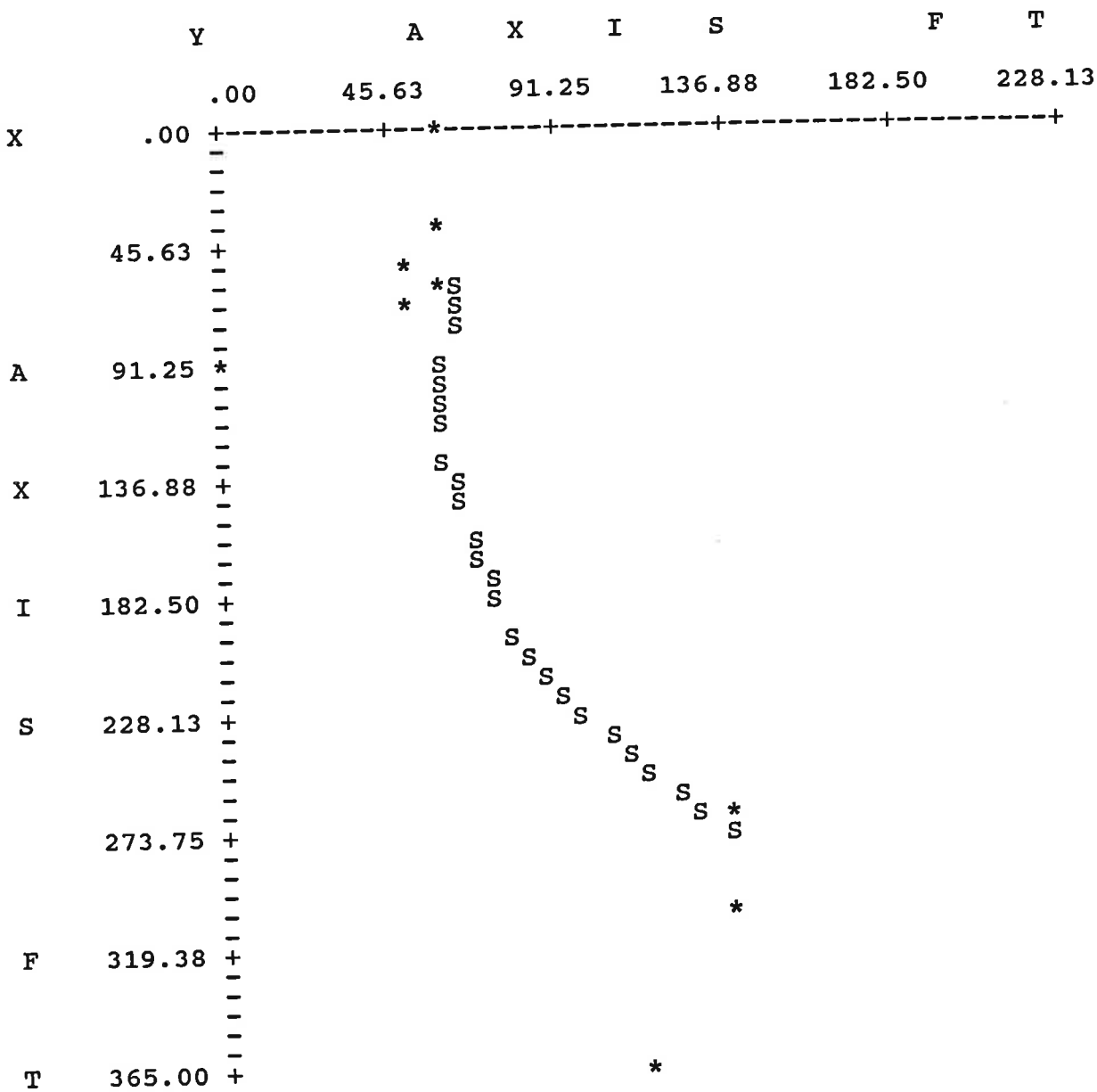
Trial Failure Surface Specified By 25 Coordinate Points

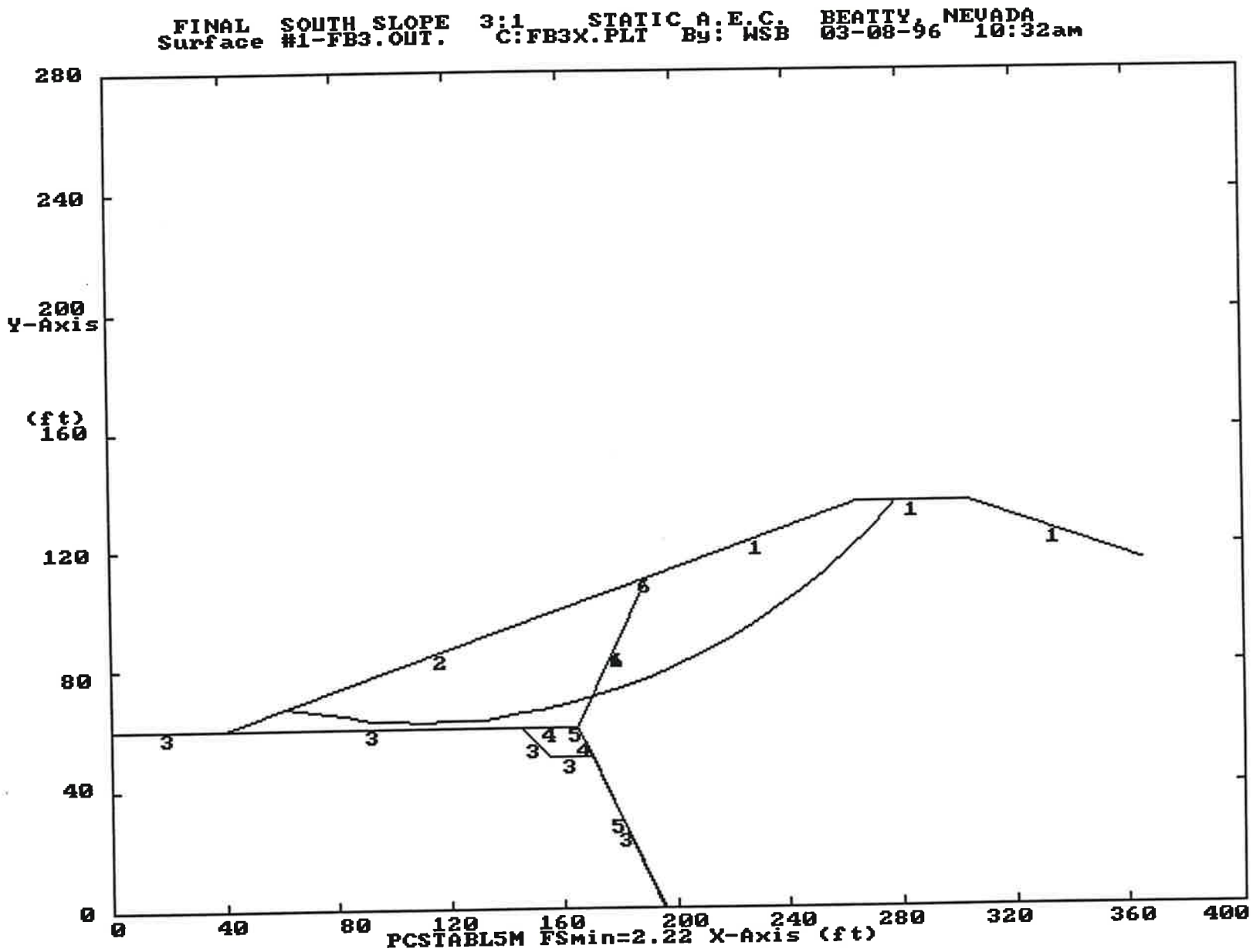
| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 57.90 | 65.97 |
| 2 | 67.70 | 64.00 |
| 3 | 77.58 | 62.47 |
| 4 | 87.52 | 61.38 |
| 5 | 97.50 | 60.74 |
| 6 | 107.50 | 60.54 |
| 7 | 117.50 | 60.79 |
| 8 | 127.47 | 61.48 |
| 9 | 137.41 | 62.61 |
| 10 | 147.28 | 64.18 |
| 11 | 157.08 | 66.20 |
| 12 | 166.78 | 68.64 |

| | | |
|---|---|---|
| 13 | 176.35 | 71.52 |
| 14 | 185.80 | 74.81 |
| 15 | 195.08 | 78.53 |
| 16 | 204.19 | 82.65 |
| 17 | 213.11 | 87.18 |
| 18 | 221.82 | 92.09 |
| 19 | 230.30 | 97.39 |
| 20 | 238.53 | 103.06 |
| 21 | 246.51 | 109.10 |
| 22 | 254.21 | 115.48 |
| 23 | 261.62 | 122.19 |
| 24 | 268.72 | 129.23 |
| 25 | 274.04 | 135.00 |

Factor Of Safety For The Preceding Specified Surface = 2.059

1




FINAL SOUTH SLOPE 3:1 STATIC A.E.C. BEATTY, NEVADA
Surface #1-FB3.OUT. C:FB3X.PLT By: WSB 03-08-96 10:32am
Y-Axis
(ft)
280
240
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
360
400
PCSTABL5M FSmin=2.22 X-Axis (ft)

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:32am |
| Run By: | WSB |
| Input Data Filename: | C:FB3X |
| Output Filename: | C:FB3X.OUT |
| Plotted Output Filename: | C:FB3X.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 3:1 STATIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 3 |
| 2 | 40.00 | 60.00 | 190.00 | 110.00 | 2 |
| 3 | 190.00 | 110.00 | 190.20 | 110.10 | 6 |
| 4 | 190.20 | 110.10 | 265.00 | 135.00 | 1 |
| 5 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 6 | 305.00 | 135.00 | 365.00 | 115.00 | 1 |
| 7 | 165.00 | 60.00 | 190.00 | 110.00 | 6 |
| 8 | 165.20 | 60.00 | 190.20 | 110.10 | 1 |
| 9 | 40.00 | 60.00 | 145.00 | 60.00 | 3 |
| 10 | 145.00 | 60.00 | 165.00 | 60.00 | 4 |
| 11 | 165.00 | 60.00 | 165.20 | 60.00 | 5 |
| 12 | 145.00 | 60.00 | 155.00 | 50.00 | 3 |
| 13 | 165.20 | 60.00 | 195.20 | .00 | 5 |
| 14 | 165.00 | 60.00 | 170.00 | 50.00 | 4 |
| 15 | 155.00 | 50.00 | 170.00 | 50.00 | 3 |
| 16 | 170.00 | 50.00 | 195.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

1

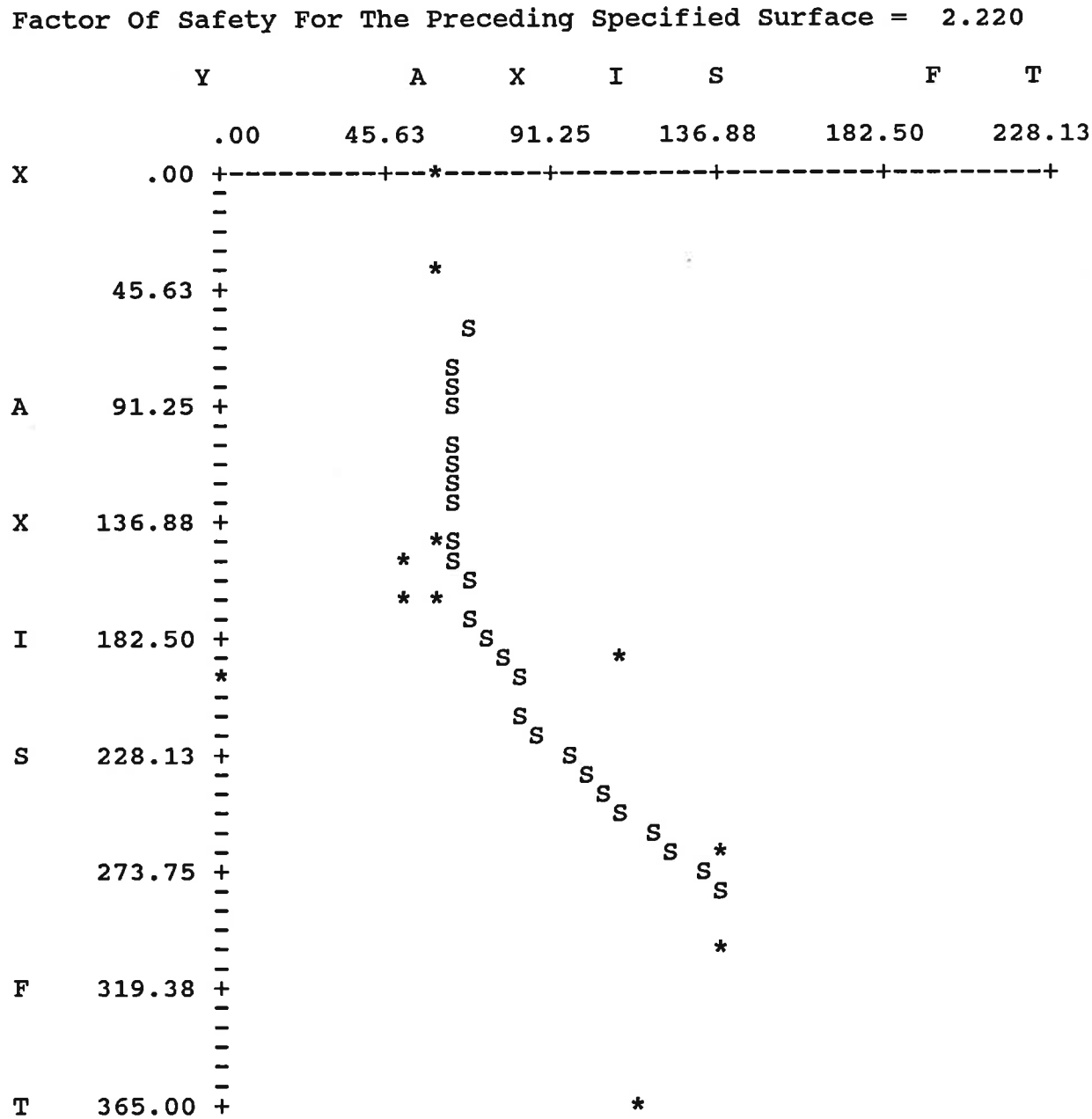
Trial Failure Surface Specified By 25 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 63.16 | 67.72 |
| 2 | 72.96 | 65.75 |
| 3 | 82.85 | 64.22 |
| 4 | 92.79 | 63.13 |
| 5 | 102.76 | 62.48 |

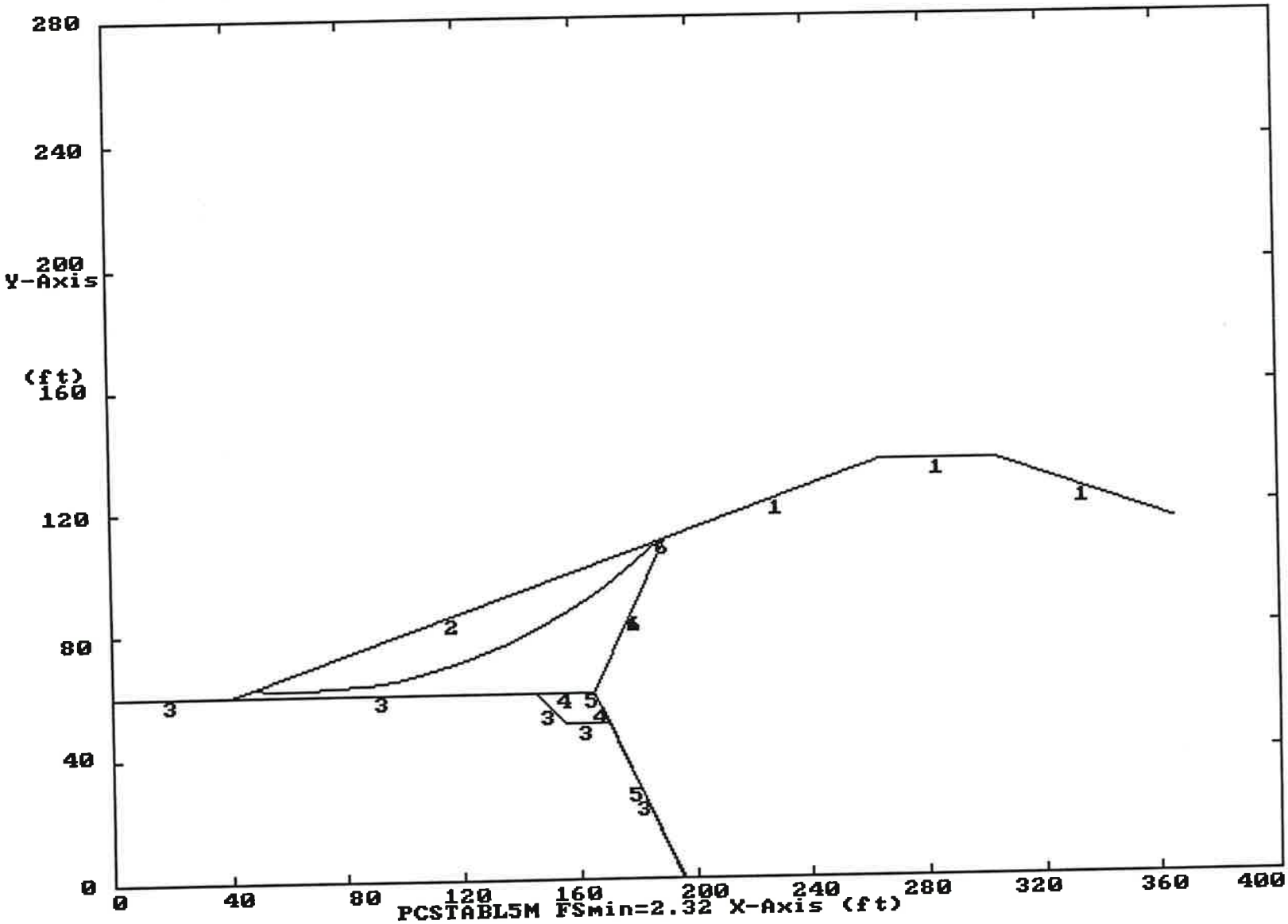
| | | |
|---|---|---|
| 6 | 112.76 | 62.27 |
| 7 | 122.76 | 62.51 |
| 8 | 132.74 | 63.18 |
| 9 | 142.67 | 64.30 |
| 10 | 152.55 | 65.86 |
| 11 | 162.35 | 67.85 |
| 12 | 172.05 | 70.27 |
| 13 | 181.64 | 73.12 |
| 14 | 191.09 | 76.39 |
| 15 | 200.38 | 80.08 |
| 16 | 209.51 | 84.17 |
| 17 | 218.44 | 88.67 |
| 18 | 227.17 | 93.55 |
| 19 | 235.67 | 98.82 |
| 20 | 243.93 | 104.46 |
| 21 | 251.93 | 110.45 |
| 22 | 259.66 | 116.80 |
| 23 | 267.10 | 123.48 |
| 24 | 274.24 | 130.48 |
| 25 | 278.45 | 135.00 |

Factor Of Safety For The Preceding Specified Surface = 2.220

1



FINAL SOUTH SLOPE 3:1 STATIC A.E.C. BEATTY, NEVADA
Surface #1-FB3A.OUT. C:FB3AX.PLT By: WSB 03-08-96 10:43am



** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:43am |
| Run By: | WSB |
| Input Data Filename: | C:FB3AX |
| Output Filename: | C:FB3AX.OUT |
| Plotted Output Filename: | C:FB3AX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 3:1 STATIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 3 |
| 2 | 40.00 | 60.00 | 190.00 | 110.00 | 2 |
| 3 | 190.00 | 110.00 | 190.20 | 110.10 | 6 |
| 4 | 190.20 | 110.10 | 265.00 | 135.00 | 1 |
| 5 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 6 | 305.00 | 135.00 | 365.00 | 115.00 | 1 |
| 7 | 165.00 | 60.00 | 190.00 | 110.00 | 6 |
| 8 | 165.20 | 60.00 | 190.20 | 110.10 | 1 |
| 9 | 40.00 | 60.00 | 145.00 | 60.00 | 3 |
| 10 | 145.00 | 60.00 | 165.00 | 60.00 | 4 |
| 11 | 165.00 | 60.00 | 165.20 | 60.00 | 5 |
| 12 | 145.00 | 60.00 | 155.00 | 50.00 | 3 |
| 13 | 165.20 | 60.00 | 195.20 | .00 | 5 |
| 14 | 165.00 | 60.00 | 170.00 | 50.00 | 4 |
| 15 | 155.00 | 50.00 | 170.00 | 50.00 | 3 |
| 16 | 170.00 | 50.00 | 195.00 | .00 | 3 |

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

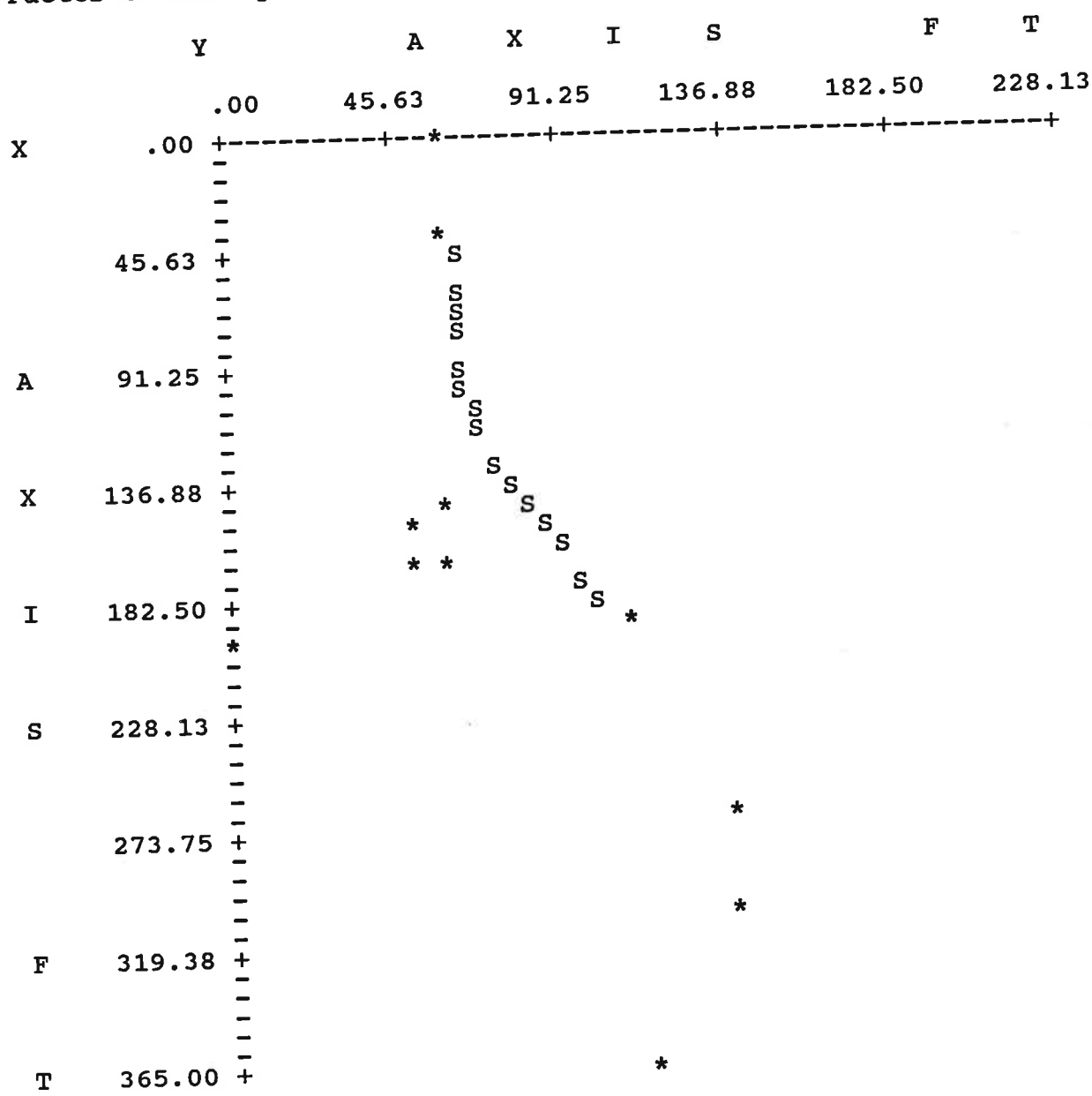
Trial Failure Surface Specified By 17 Coordinate Points

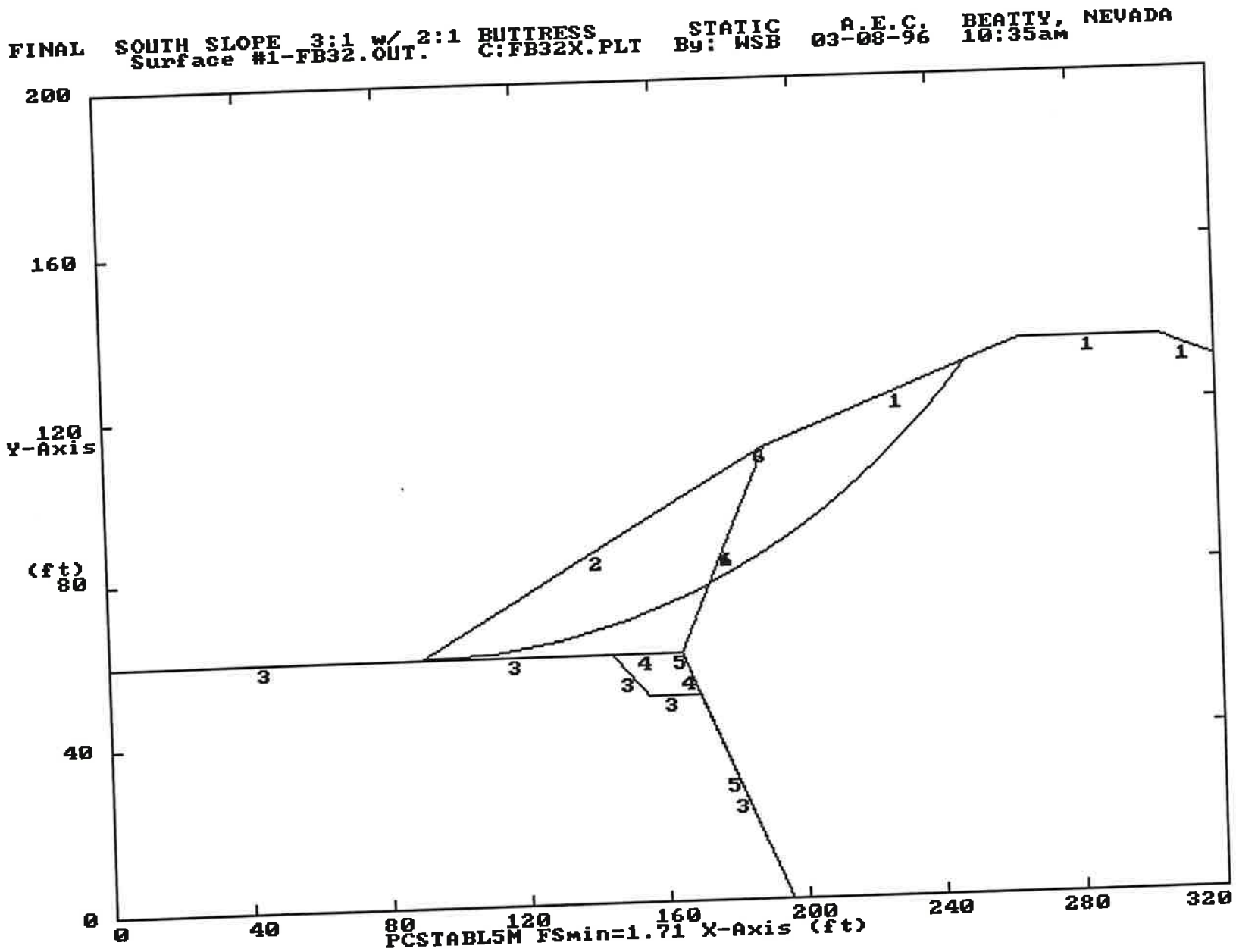
| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 48.42 | 62.81 |
| 2 | 58.40 | 62.16 |
| 3 | 68.40 | 62.06 |
| 4 | 78.39 | 62.50 |
| 5 | 88.34 | 63.48 |

| | | |
|---|---|---|
| 6 | 98.22 | 65.01 |
| 7 | 108.01 | 67.07 |
| 8 | 117.67 | 69.66 |
| 9 | 127.17 | 72.78 |
| 10 | 136.49 | 76.41 |
| 11 | 145.59 | 80.54 |
| 12 | 154.46 | 85.16 |
| 13 | 163.07 | 90.26 |
| 14 | 171.38 | 95.82 |
| 15 | 179.38 | 101.82 |
| 16 | 187.03 | 108.25 |
| 17 | 188.29 | 109.43 |

Factor Of Safety For The Preceding Specified Surface = 2.318

1




FINAL SOUTH SLOPE 3:1 w/ 2:1 BUTTRESS STATIC A.E.C. BEATTY, NEVADA
Surface #1-FB32.OUT. C:FB32X.PLT By: WSB 03-08-96 10:35am
Y-Axis (ft)
X-Axis (ft)
PCSTABL5M FSmin=1.71
0
40
80
120
160
200
240
280
320

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:35am |
| Run By: | WSB |
| Input Data Filename: | C:FB32X |
| Output Filename: | C:FB32X.OUT |
| Plotted Output Filename: | C:FB32X.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 3:1 w/ 2:1 BUTTRESS
STATIC A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 90.00 | 60.00 | 3 |
| 2 | 90.00 | 60.00 | 190.00 | 110.00 | 2 |
| 3 | 190.00 | 110.00 | 190.20 | 110.10 | 6 |
| 4 | 190.20 | 110.10 | 265.00 | 135.00 | 1 |
| 5 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 6 | 305.00 | 135.00 | 320.00 | 130.00 | 1 |
| 7 | 165.00 | 60.00 | 190.00 | 110.00 | 6 |
| 8 | 165.20 | 60.00 | 190.20 | 110.10 | 1 |
| 9 | 90.00 | 60.00 | 145.00 | 60.00 | 3 |
| 10 | 145.00 | 60.00 | 165.00 | 60.00 | 4 |
| 11 | 165.00 | 60.00 | 165.20 | 60.00 | 5 |
| 12 | 145.00 | 60.00 | 155.00 | 50.00 | 3 |
| 13 | 165.20 | 60.00 | 195.20 | .00 | 5 |
| 14 | 165.00 | 60.00 | 170.00 | 50.00 | 4 |
| 15 | 155.00 | 50.00 | 170.00 | 50.00 | 3 |
| 16 | 170.00 | 50.00 | 195.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

1

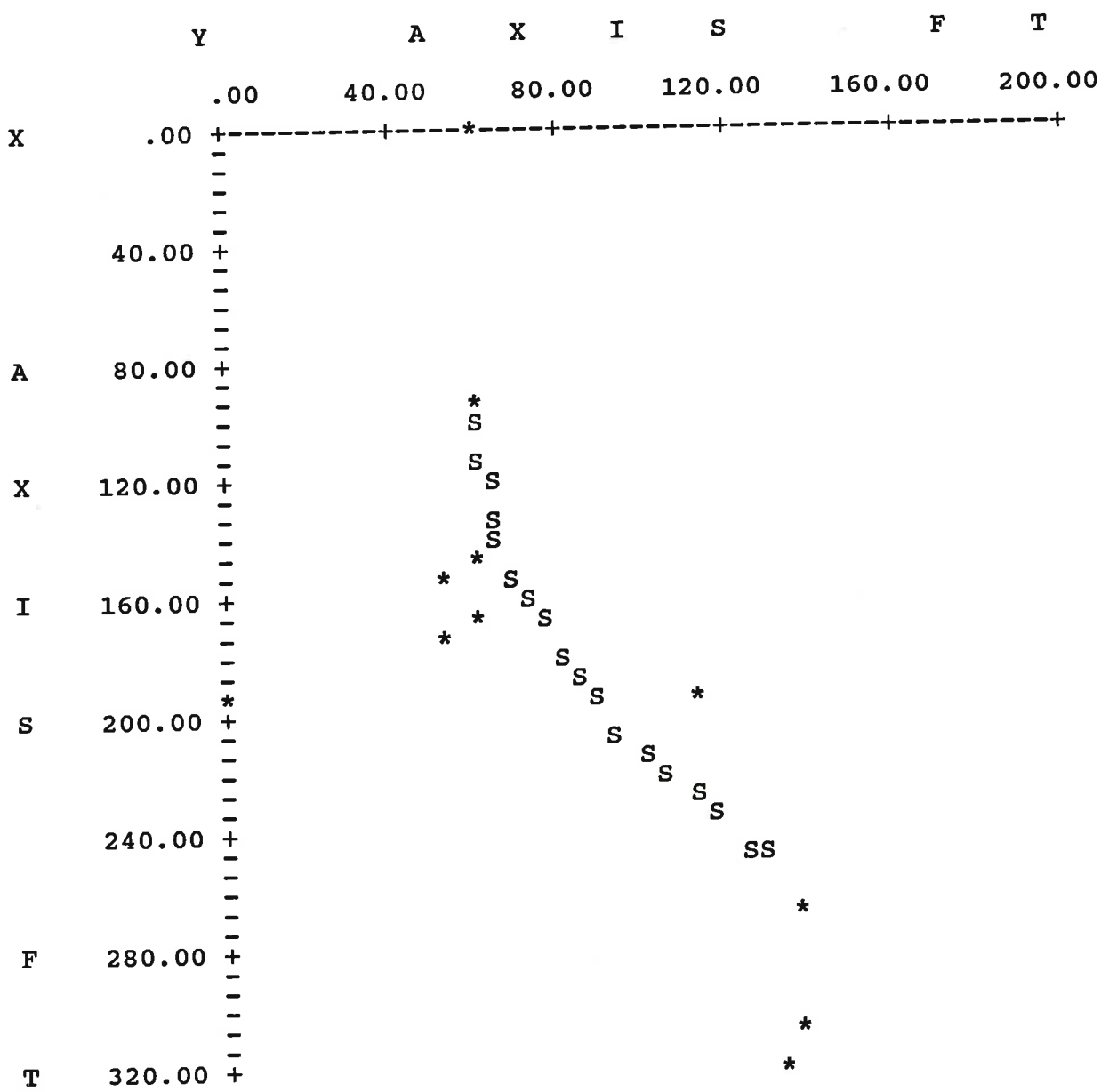
Trial Failure Surface Specified By 19 Coordinate Points

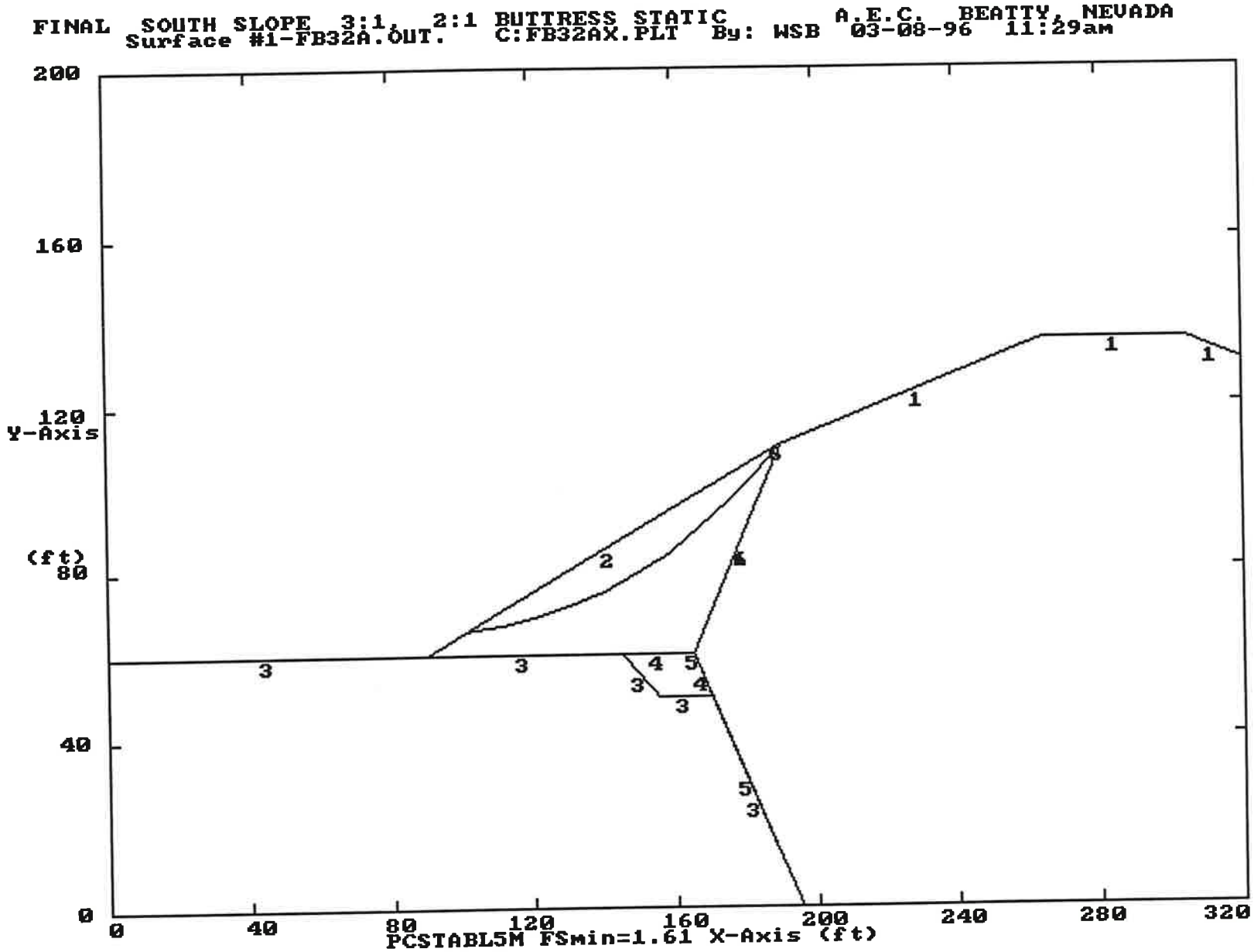
| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 91.05 | 60.53 |
| 2 | 101.05 | 60.67 |
| 3 | 111.03 | 61.29 |
| 4 | 120.97 | 62.38 |
| 5 | 130.85 | 63.95 |

| | | |
|---|---|---|
| 6 | 140.64 | 65.99 |
| 7 | 150.32 | 68.50 |
| 8 | 159.87 | 71.47 |
| 9 | 169.27 | 74.89 |
| 10 | 178.49 | 78.75 |
| 11 | 187.51 | 83.06 |
| 12 | 196.32 | 87.79 |
| 13 | 204.90 | 92.93 |
| 14 | 213.22 | 98.48 |
| 15 | 221.26 | 104.43 |
| 16 | 229.01 | 110.75 |
| 17 | 236.45 | 117.43 |
| 18 | 243.56 | 124.46 |
| 19 | 248.05 | 129.36 |

Factor Of Safety For The Preceding Specified Surface = 1.705

1




FINAL SOUTH SLOPE 3:1, 2:1 BUTTRESS STATIC A.E.C. BEATTY, NEVADA
Surface #1-FB32A.OUT. C:FB32AX.PLT By: WSB 03-08-96 11:29am
200
160
120
Y-Axis
(ft)
80
40
0
0
40
80
120
160
200
240
280
320
PCSTABL5M FSmin=1.61 X-Axis (ft)

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 11:29am |
| Run By: | WSB |
| Input Data Filename: | C:FB32AX |
| Output Filename: | C:FB32AX.OUT |
| Plotted Output Filename: | C:FB32AX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 3:1, 2:1 BUTTRESS
STATIC A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 90.00 | 60.00 | 3 |
| 2 | 90.00 | 60.00 | 190.00 | 110.00 | 2 |
| 3 | 190.00 | 110.00 | 190.20 | 110.10 | 6 |
| 4 | 190.20 | 110.10 | 265.00 | 135.00 | 1 |
| 5 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 6 | 305.00 | 135.00 | 320.00 | 130.00 | 1 |
| 7 | 165.00 | 60.00 | 190.00 | 110.00 | 6 |
| 8 | 165.20 | 60.00 | 190.20 | 110.10 | 1 |
| 9 | 90.00 | 60.00 | 145.00 | 60.00 | 3 |
| 10 | 145.00 | 60.00 | 165.00 | 60.00 | 4 |
| 11 | 165.00 | 60.00 | 165.20 | 60.00 | 5 |
| 12 | 145.00 | 60.00 | 155.00 | 50.00 | 3 |
| 13 | 165.20 | 60.00 | 195.20 | .00 | 5 |
| 14 | 165.00 | 60.00 | 170.00 | 50.00 | 4 |
| 15 | 155.00 | 50.00 | 170.00 | 50.00 | 3 |
| 16 | 170.00 | 50.00 | 195.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

1

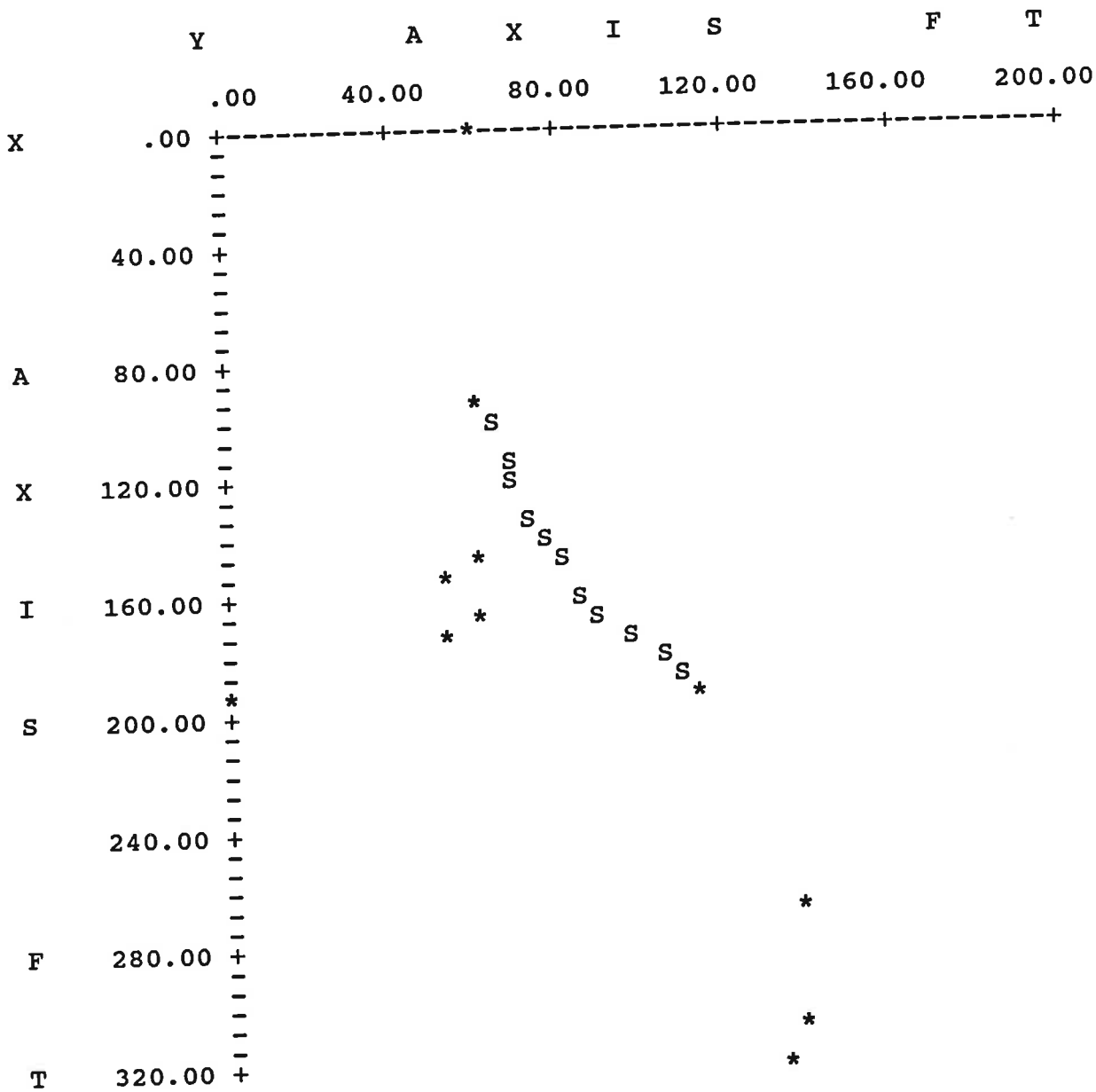
Trial Failure Surface Specified By 12 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 101.58 | 65.79 |
| 2 | 111.50 | 67.03 |
| 3 | 121.30 | 69.02 |
| 4 | 130.93 | 71.73 |
| 5 | 140.32 | 75.16 |

| | | |
|---|---|---|
| 6 | 149.43 | 79.28 |
| 7 | 158.21 | 84.08 |
| 8 | 166.60 | 89.52 |
| 9 | 174.56 | 95.57 |
| 10 | 182.04 | 102.20 |
| 11 | 189.01 | 109.38 |
| 12 | 189.19 | 109.60 |

Factor Of Safety For The Preceding Specified Surface = 1.609

1



# ENVIRONMENTAL SOLUTIONS, INC.

By UG Date 1/10/96 Subject SLOPE STABILITY EVALUATION Sheet No. ____ of ____

Chkd. By ____ Date ____ FINAL CONFIGURATION Proj. No. 95-284

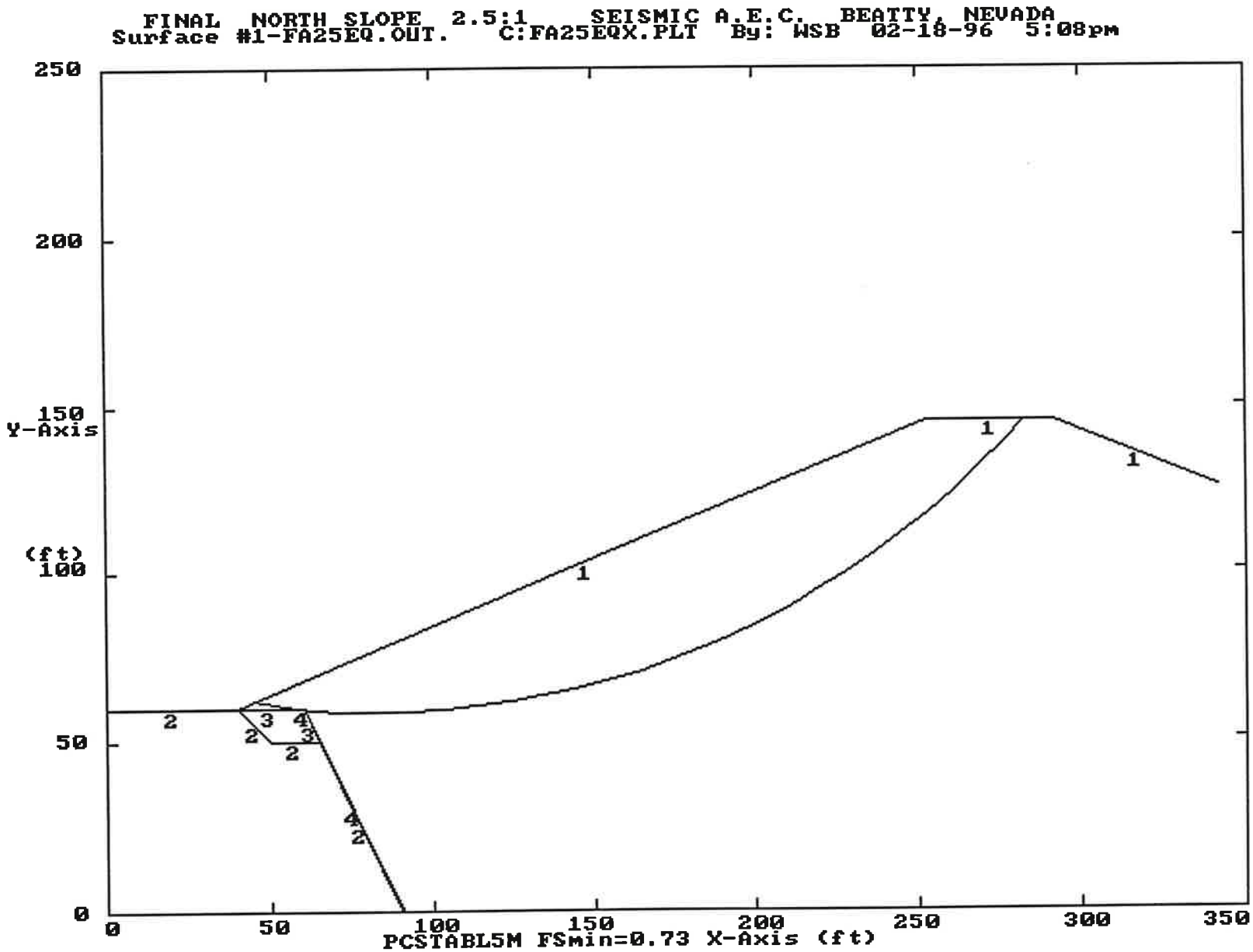
ATTACHMENT C

STABL5M COMPUTER OUTPUT

PSEUDOSTATIC CASE



Recyclable and made from recycled paper.


FINAL NORTH SLOPE 2.5:1 SEISMIC A.E.C. BEATTY, NEVADA
Surface #1-FA25EQ.OUT. C:FA25EQX.PLT By: WSB 02-18-96 5:08pm
250
200
150
Y-Axis
(ft)
100
50
0
0
50
100
150
200
250
300
350
PCSTABL5M FSmin=0.73 X-Axis (ft)
1
1
1
2
2
2
2
3
3
4
4
4

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 02-18-96 |
| Time of Run: | 5:08pm |
| Run By: | WSB |
| Input Data Filename: | C:FA25EQX |
| Output Filename: | C:FA25EQX.OUT |
| Plotted Output Filename: | C:FA25EQX.PLT |

PROBLEM DESCRIPTION FINAL NORTH SLOPE 2.5:1 SEISMIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

4 Top Boundaries
11 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 2 |
| 2 | 40.00 | 60.00 | 252.50 | 145.00 | 1 |
| 3 | 252.50 | 145.00 | 292.50 | 145.00 | 1 |
| 4 | 292.50 | 145.00 | 342.50 | 125.00 | 1 |
| 5 | 40.00 | 60.00 | 60.00 | 60.00 | 3 |
| 6 | 60.00 | 60.00 | 60.20 | 60.00 | 4 |
| 7 | 40.00 | 60.00 | 50.00 | 50.00 | 2 |
| 8 | 60.20 | 60.00 | 90.20 | .00 | 4 |
| 9 | 60.00 | 60.00 | 65.00 | 50.00 | 3 |
| 10 | 50.00 | 50.00 | 65.00 | 50.00 | 2 |
| 11 | 65.00 | 50.00 | 90.00 | .00 | 2 |

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .420 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

1

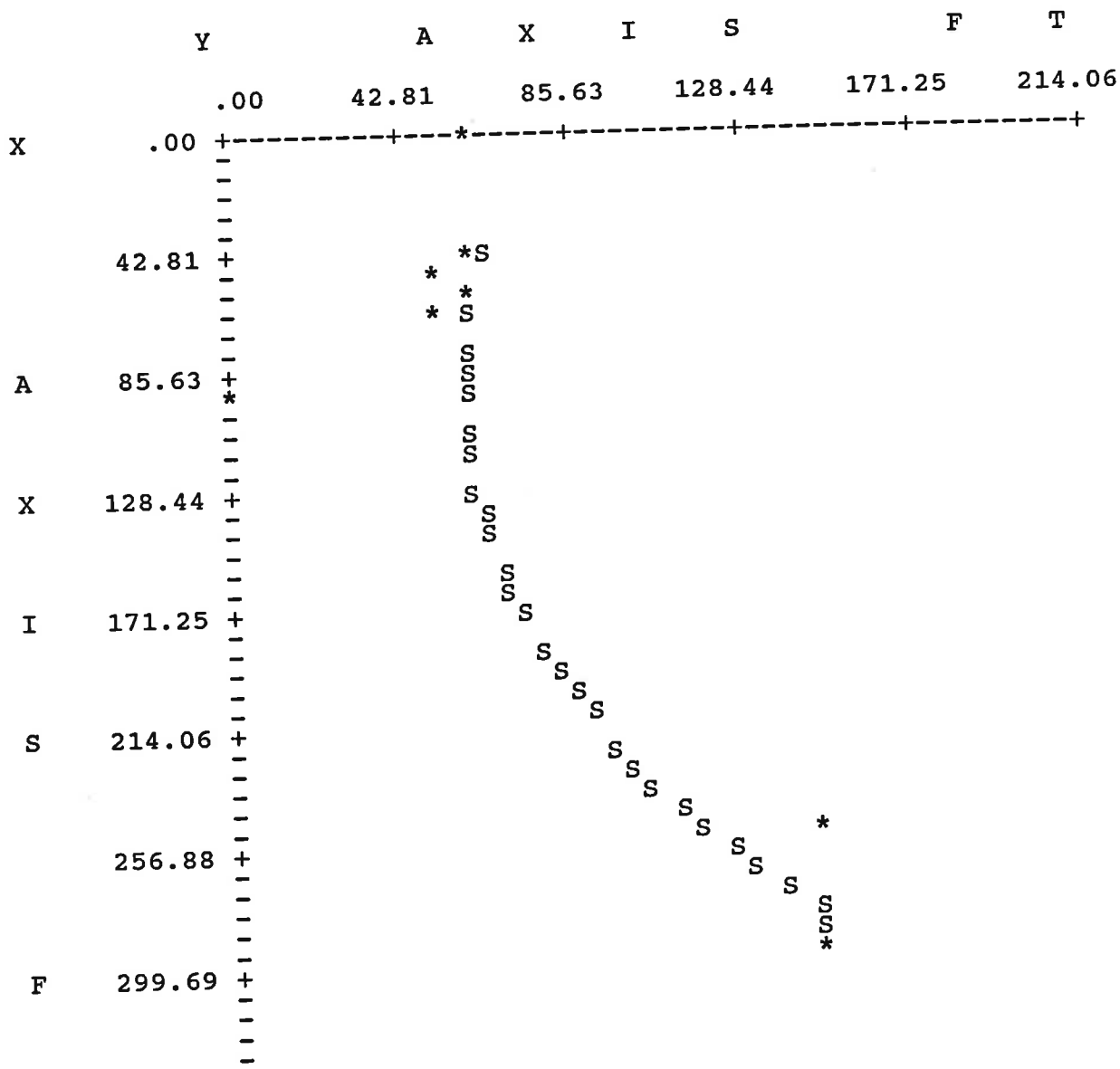
Trial Failure Surface Specified By 28 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 45.26 | 62.10 |
| 2 | 55.17 | 60.72 |

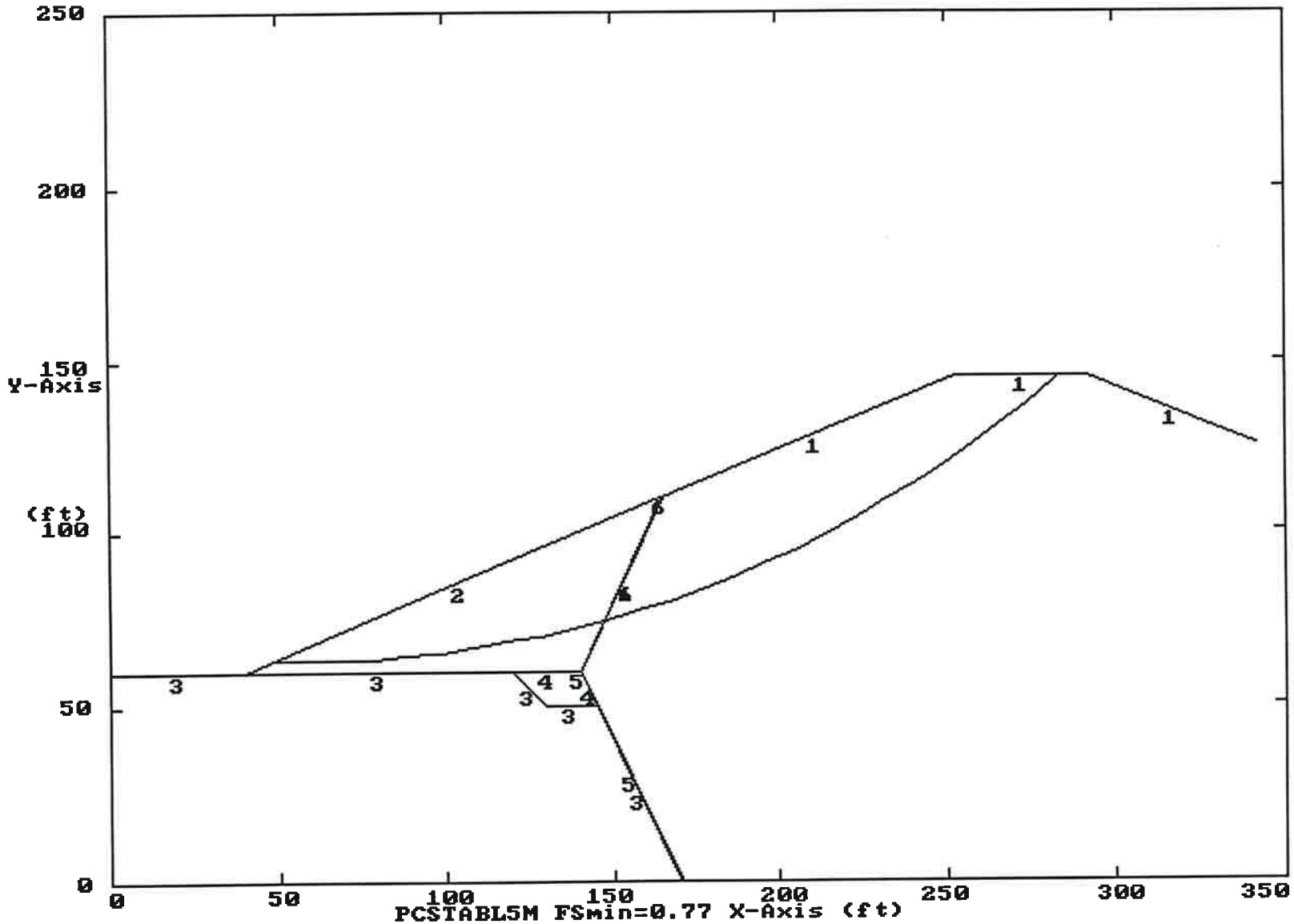
| | | |
|---|---|---|
| 3 | 65.12 | 59.72 |
| 4 | 75.10 | 59.09 |
| 5 | 85.09 | 58.83 |
| 6 | 95.09 | 58.95 |
| 7 | 105.08 | 59.45 |
| 8 | 115.04 | 60.33 |
| 9 | 124.96 | 61.58 |
| 10 | 134.83 | 63.21 |
| 11 | 144.63 | 65.20 |
| 12 | 154.35 | 67.57 |
| 13 | 163.97 | 70.29 |
| 14 | 173.48 | 73.38 |
| 15 | 182.86 | 76.83 |
| 16 | 192.12 | 80.63 |
| 17 | 201.22 | 84.77 |
| 18 | 210.15 | 89.26 |
| 19 | 218.92 | 94.07 |
| 20 | 227.49 | 99.22 |
| 21 | 235.87 | 104.68 |
| 22 | 244.03 | 110.46 |
| 23 | 251.97 | 116.54 |
| 24 | 259.67 | 122.92 |
| 25 | 267.13 | 129.58 |
| 26 | 274.33 | 136.52 |
| 27 | 281.27 | 143.72 |
| 28 | 282.40 | 145.00 |

Factor Of Safety For The Preceding Specified Surface = .731

1



FINAL SOUTH SLOPE 2.5:1 SEISMIC A.E.C. BEATTY, NEVADA
Surface #1-FB25EQ.OUT. C:FB25EQX.PLT By: WSB 03-08-96 10:21am



** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:21am |
| Run By: | WSB |
| Input Data Filename: | C:FB25EQX |
| Output Filename: | C:FB25EQX.OUT |
| Plotted Output Filename: | C:FB25EQX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 2.5:1 SEISMIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 3 |
| 2 | 40.00 | 60.00 | 165.00 | 110.00 | 2 |
| 3 | 165.00 | 110.00 | 165.20 | 110.10 | 6 |
| 4 | 165.20 | 110.10 | 252.50 | 145.00 | 1 |
| 5 | 252.50 | 145.00 | 292.50 | 145.00 | 1 |
| 6 | 292.50 | 145.00 | 342.50 | 125.00 | 1 |
| 7 | 140.00 | 60.00 | 165.00 | 110.00 | 6 |
| 8 | 140.20 | 60.00 | 165.20 | 110.10 | 1 |
| 9 | 40.00 | 60.00 | 120.00 | 60.00 | 3 |
| 10 | 120.00 | 60.00 | 140.00 | 60.00 | 4 |
| 11 | 140.00 | 60.00 | 140.20 | 60.00 | 5 |
| 12 | 120.00 | 60.00 | 130.00 | 50.00 | 3 |
| 13 | 140.20 | 60.00 | 170.20 | .00 | 5 |
| 14 | 140.00 | 60.00 | 145.00 | 50.00 | 4 |
| 15 | 130.00 | 50.00 | 145.00 | 50.00 | 3 |
| 16 | 145.00 | 50.00 | 170.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | .5 | .5 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | .5 | .5 | .0 | 14.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .420 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

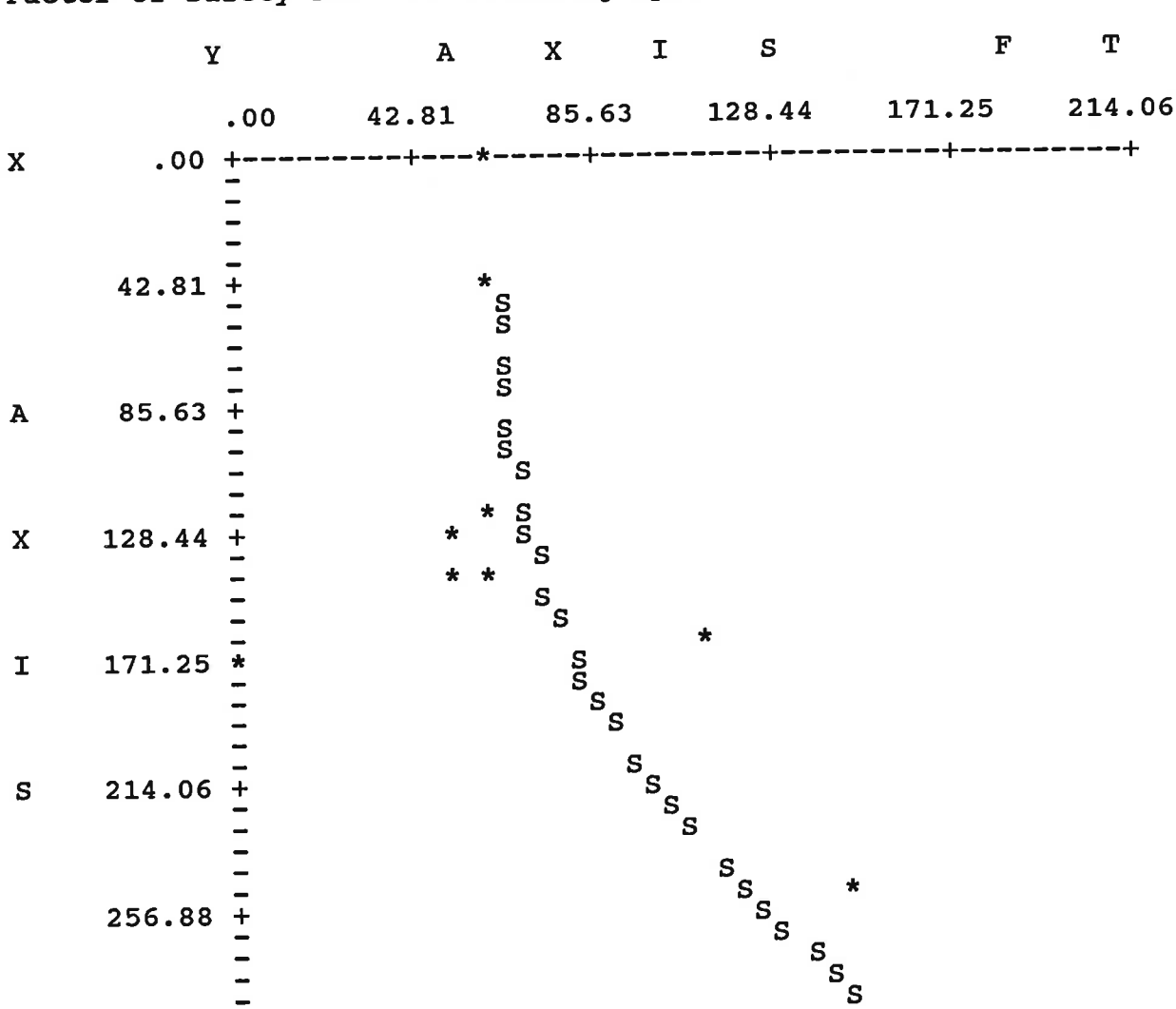
1

Trial Failure Surface Specified By 27 Coordinate Points

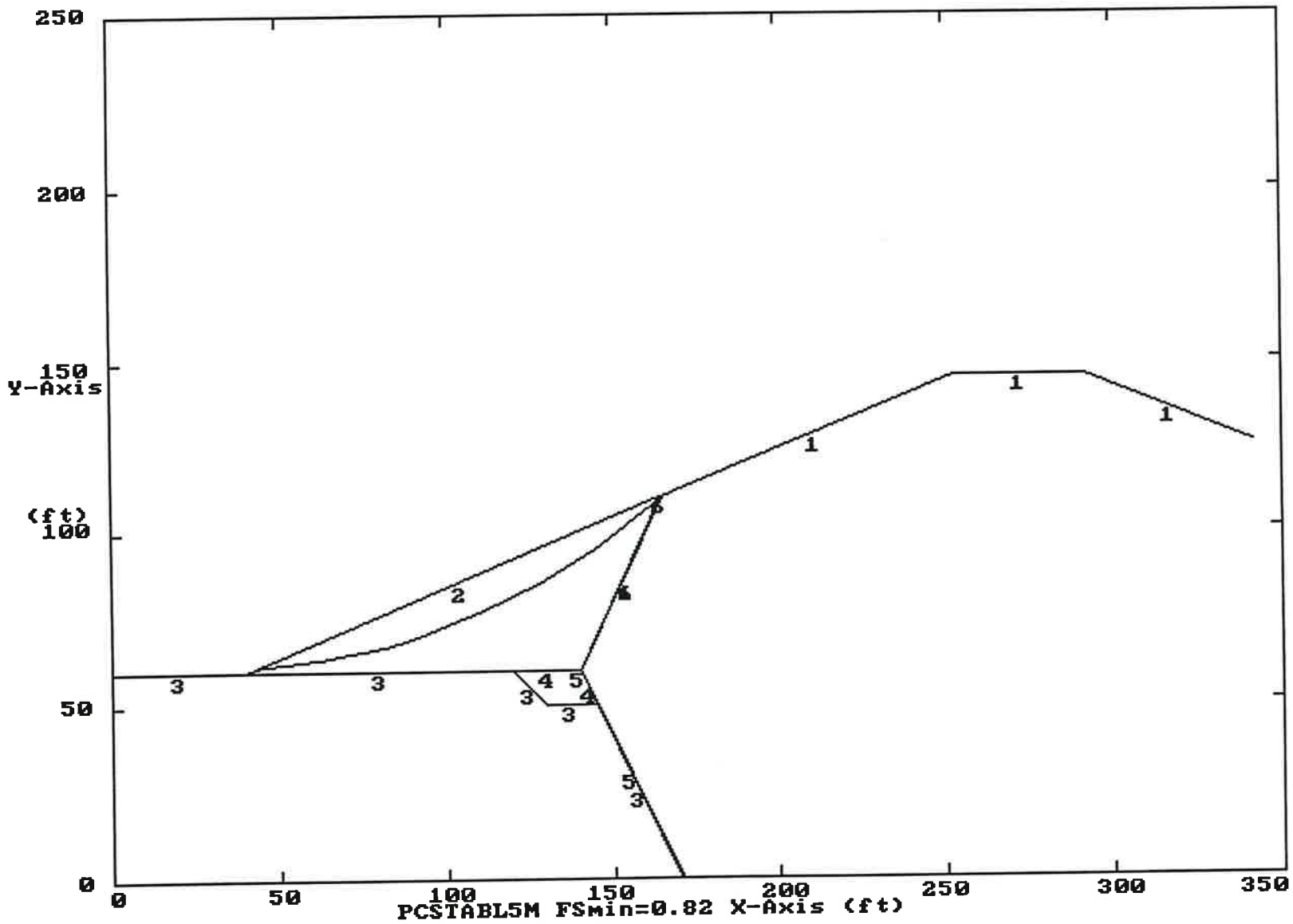
| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 49.47 | 63.79 |
| 2 | 59.47 | 63.56 |
| 3 | 69.47 | 63.62 |
| 4 | 79.47 | 63.97 |
| 5 | 89.44 | 64.62 |
| 6 | 99.40 | 65.56 |
| 7 | 109.32 | 66.79 |
| 8 | 119.21 | 68.31 |
| 9 | 129.04 | 70.12 |
| 10 | 138.82 | 72.23 |
| 11 | 148.53 | 74.61 |
| 12 | 158.17 | 77.28 |
| 13 | 167.72 | 80.24 |
| 14 | 177.18 | 83.47 |
| 15 | 186.55 | 86.98 |
| 16 | 195.80 | 90.76 |
| 17 | 204.95 | 94.82 |
| 18 | 213.96 | 99.14 |
| 19 | 222.85 | 103.72 |
| 20 | 231.60 | 108.56 |
| 21 | 240.21 | 113.66 |
| 22 | 248.66 | 119.00 |
| 23 | 256.94 | 124.60 |
| 24 | 265.07 | 130.43 |
| 25 | 273.01 | 136.50 |
| 26 | 280.78 | 142.80 |
| 27 | 283.33 | 145.00 |

Factor Of Safety For The Preceding Specified Surface = .768

1



FINAL SOUTH SLOPE 2.5:1 SEISMIC A.E.C. BEATTY, NEVADA
Surface #1-FB25AEQ.OUT. C:FB25AEQX.PLT By: WSB 03-08-96 10:17am



** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:17am |
| Run By: | WSB |
| Input Data Filename: | C:FB25AEQX |
| Output Filename: | C:FB25AEQX.OUT |
| Plotted Output Filename: | C:FB25AEQX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 2.5:1 SEISMIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 3 |
| 2 | 40.00 | 60.00 | 165.00 | 110.00 | 2 |
| 3 | 165.00 | 110.00 | 165.20 | 110.10 | 6 |
| 4 | 165.20 | 110.10 | 252.50 | 145.00 | 1 |
| 5 | 252.50 | 145.00 | 292.50 | 145.00 | 1 |
| 6 | 292.50 | 145.00 | 342.50 | 125.00 | 1 |
| 7 | 140.00 | 60.00 | 165.00 | 110.00 | 6 |
| 8 | 140.20 | 60.00 | 165.20 | 110.10 | 1 |
| 9 | 40.00 | 60.00 | 120.00 | 60.00 | 3 |
| 10 | 120.00 | 60.00 | 140.00 | 60.00 | 4 |
| 11 | 140.00 | 60.00 | 140.20 | 60.00 | 5 |
| 12 | 120.00 | 60.00 | 130.00 | 50.00 | 3 |
| 13 | 140.20 | 60.00 | 170.20 | .00 | 5 |
| 14 | 140.00 | 60.00 | 145.00 | 50.00 | 4 |
| 15 | 130.00 | 50.00 | 145.00 | 50.00 | 3 |
| 16 | 145.00 | 50.00 | 170.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .420 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

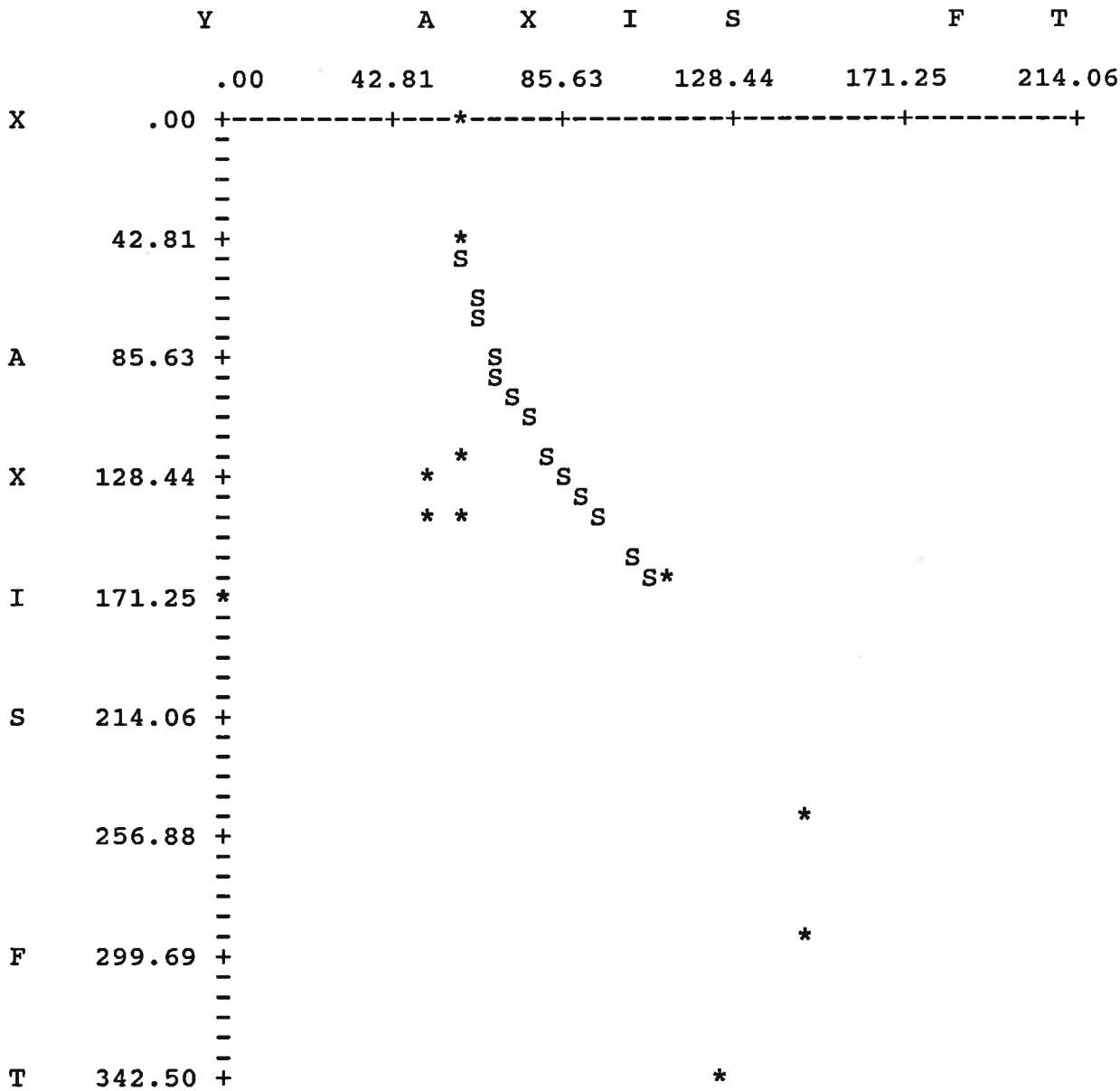
1

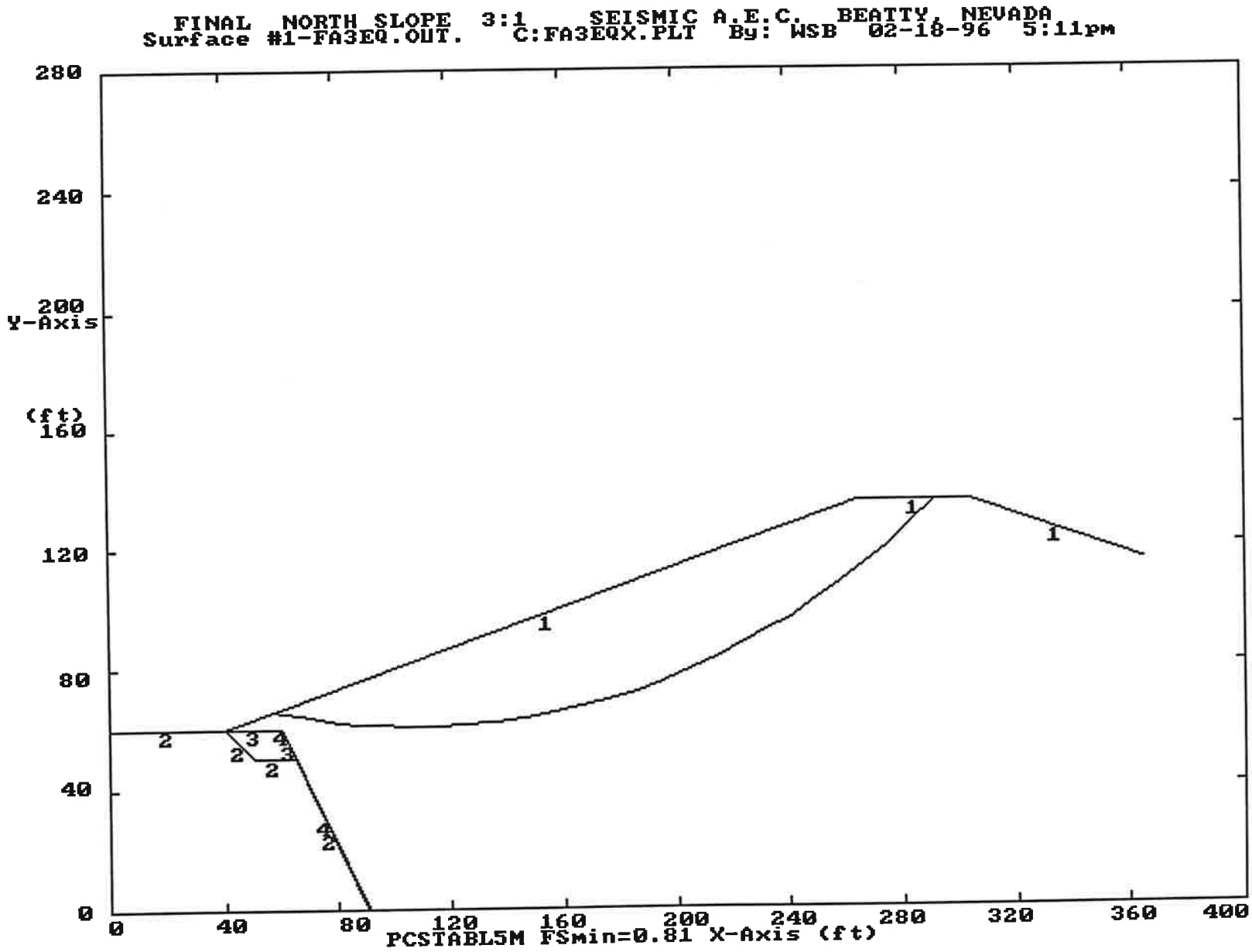
Trial Failure Surface Specified By 15 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 42.63 | 61.05 |
| 2 | 52.59 | 61.93 |
| 3 | 62.50 | 63.28 |
| 4 | 72.34 | 65.09 |
| 5 | 82.08 | 67.36 |
| 6 | 91.70 | 70.09 |
| 7 | 101.18 | 73.26 |
| 8 | 110.50 | 76.88 |
| 9 | 119.64 | 80.93 |
| 10 | 128.58 | 85.41 |
| 11 | 137.30 | 90.31 |
| 12 | 145.79 | 95.61 |
| 13 | 154.01 | 101.30 |
| 14 | 161.95 | 107.37 |
| 15 | 165.20 | 110.10 |

Factor Of Safety For The Preceding Specified Surface = .815

1




FINAL NORTH SLOPE 3:1 SEISMIC A.E.C. BEATTY, NEVADA
Surface #1-FA3EQ.OUT. C:FA3EQX.PLT By: WSB 02-18-96 5:11pm
Y-Axis
(ft)
280
240
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
360
400
PCSTABL5M FSmin=0.81 X-Axis (ft)

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 02-18-96 |
| Time of Run: | 5:11pm |
| Run By: | WSB |
| Input Data Filename: | C:FA3EQX |
| Output Filename: | C:FA3EQX.OUT |
| Plotted Output Filename: | C:FA3EQX.PLT |

PROBLEM DESCRIPTION FINAL NORTH SLOPE 3:1 SEISMIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

4 Top Boundaries
11 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 2 |
| 2 | 40.00 | 60.00 | 265.00 | 135.00 | 1 |
| 3 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 4 | 305.00 | 135.00 | 365.00 | 115.00 | 1 |
| 5 | 40.00 | 60.00 | 60.00 | 60.00 | 3 |
| 6 | 60.00 | 60.00 | 60.20 | 60.00 | 4 |
| 7 | 40.00 | 60.00 | 50.00 | 50.00 | 2 |
| 8 | 60.20 | 60.00 | 90.20 | .00 | 4 |
| 9 | 60.00 | 60.00 | 65.00 | 50.00 | 3 |
| 10 | 50.00 | 50.00 | 65.00 | 50.00 | 2 |
| 11 | 65.00 | 50.00 | 90.00 | .00 | 2 |

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .420 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

1

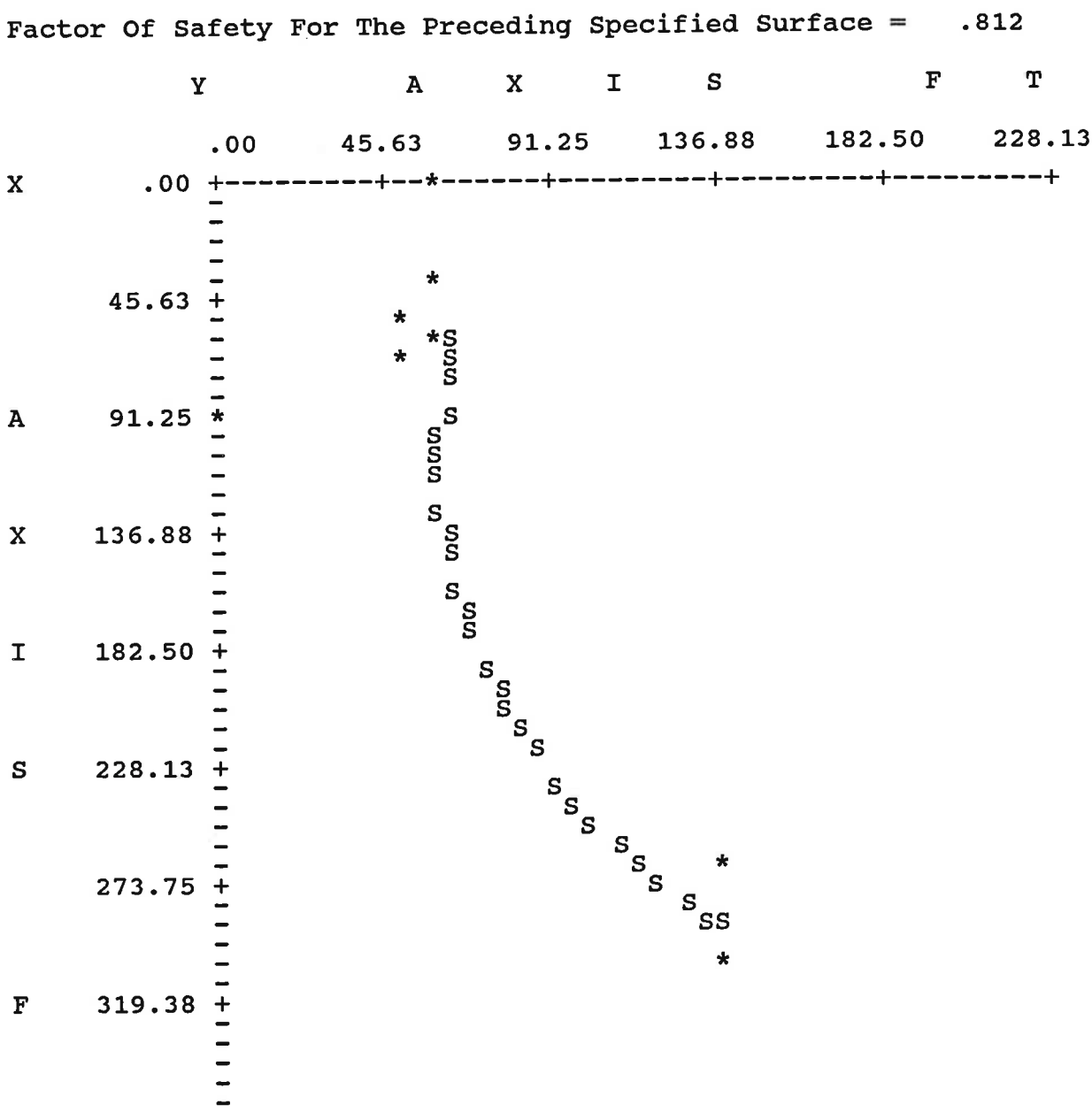
Trial Failure Surface Specified By 27 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 57.90 | 65.97 |
| 2 | 67.73 | 64.14 |

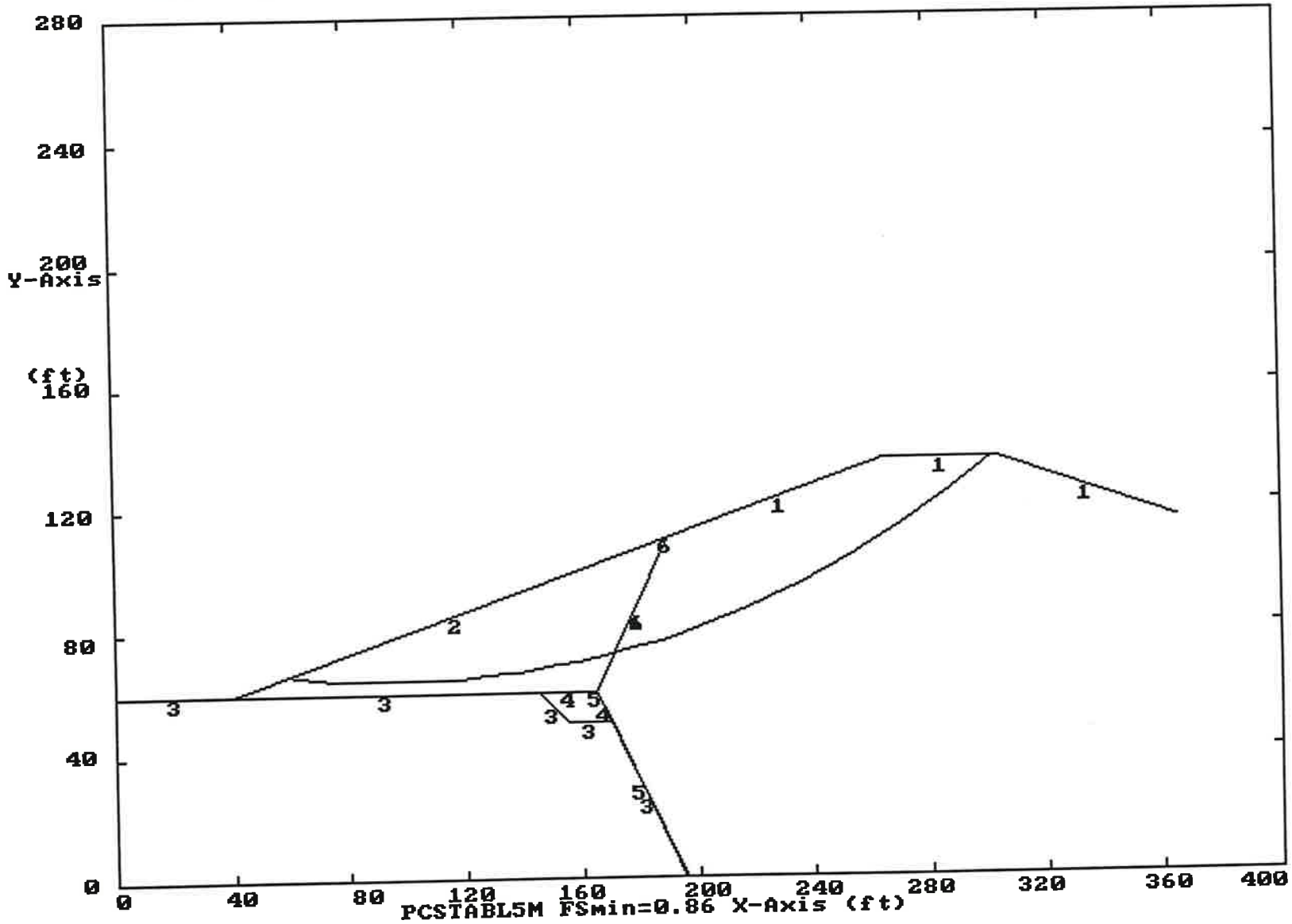
| | | |
|---|---|---|
| 3 | 77.62 | 62.70 |
| 4 | 87.57 | 61.64 |
| 5 | 97.54 | 60.97 |
| 6 | 107.54 | 60.68 |
| 7 | 117.54 | 60.78 |
| 8 | 127.53 | 61.26 |
| 9 | 137.49 | 62.14 |
| 10 | 147.41 | 63.39 |
| 11 | 157.27 | 65.03 |
| 12 | 167.07 | 67.05 |
| 13 | 176.78 | 69.45 |
| 14 | 186.39 | 72.22 |
| 15 | 195.88 | 75.36 |
| 16 | 205.25 | 78.87 |
| 17 | 214.47 | 82.73 |
| 18 | 223.54 | 86.95 |
| 19 | 232.43 | 91.51 |
| 20 | 241.15 | 96.42 |
| 21 | 249.66 | 101.66 |
| 22 | 257.97 | 107.23 |
| 23 | 266.06 | 113.11 |
| 24 | 273.91 | 119.30 |
| 25 | 281.52 | 125.79 |
| 26 | 288.87 | 132.57 |
| 27 | 291.31 | 135.00 |

Factor Of Safety For The Preceding Specified Surface = .812

1



FINAL SOUTH SLOPE 3:1 SEISMIC A.E.C. BEATTY, NEVADA
Surface #1-FB3EQ.OUT. C:FB3EQX.PLT By: WSB 03-08-96 10:53am



** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:53am |
| Run By: | WSB |
| Input Data Filename: | C:FB3EQX |
| Output Filename: | C:FB3EQX.OUT |
| Plotted Output Filename: | C:FB3EQX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 3:1 SEISMIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 3 |
| 2 | 40.00 | 60.00 | 190.00 | 110.00 | 2 |
| 3 | 190.00 | 110.00 | 190.20 | 110.10 | 6 |
| 4 | 190.20 | 110.10 | 265.00 | 135.00 | 1 |
| 5 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 6 | 305.00 | 135.00 | 365.00 | 115.00 | 1 |
| 7 | 165.00 | 60.00 | 190.00 | 110.00 | 6 |
| 8 | 165.20 | 60.00 | 190.20 | 110.10 | 1 |
| 9 | 40.00 | 60.00 | 145.00 | 60.00 | 3 |
| 10 | 145.00 | 60.00 | 165.00 | 60.00 | 4 |
| 11 | 165.00 | 60.00 | 165.20 | 60.00 | 5 |
| 12 | 145.00 | 60.00 | 155.00 | 50.00 | 3 |
| 13 | 165.20 | 60.00 | 195.20 | .00 | 5 |
| 14 | 165.00 | 60.00 | 170.00 | 50.00 | 4 |
| 15 | 155.00 | 50.00 | 170.00 | 50.00 | 3 |
| 16 | 170.00 | 50.00 | 195.00 | .00 | 3 |

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .420 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

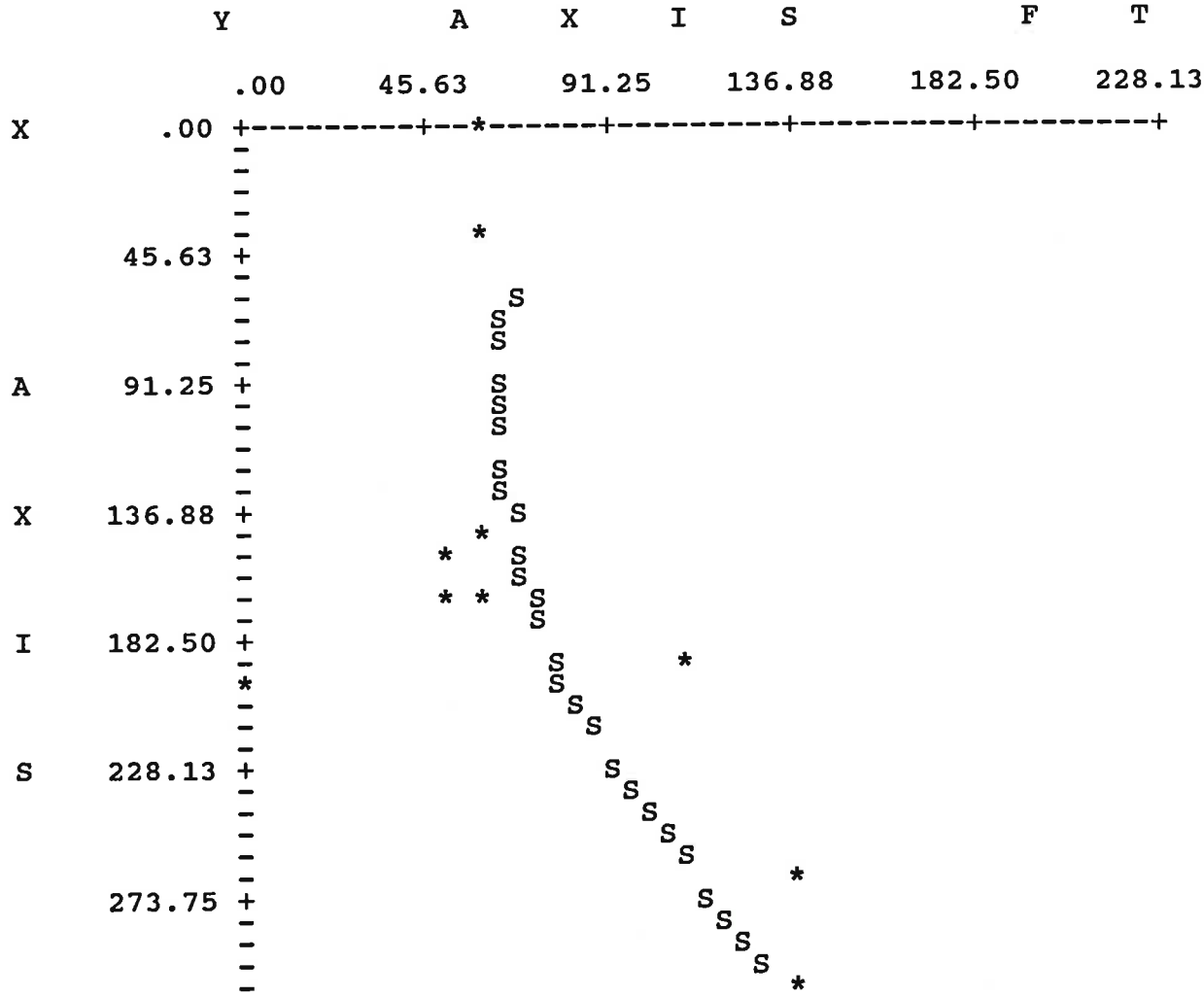
Cavitation Pressure = .0 psf

Trial Failure Surface Specified By 27 Coordinate Points

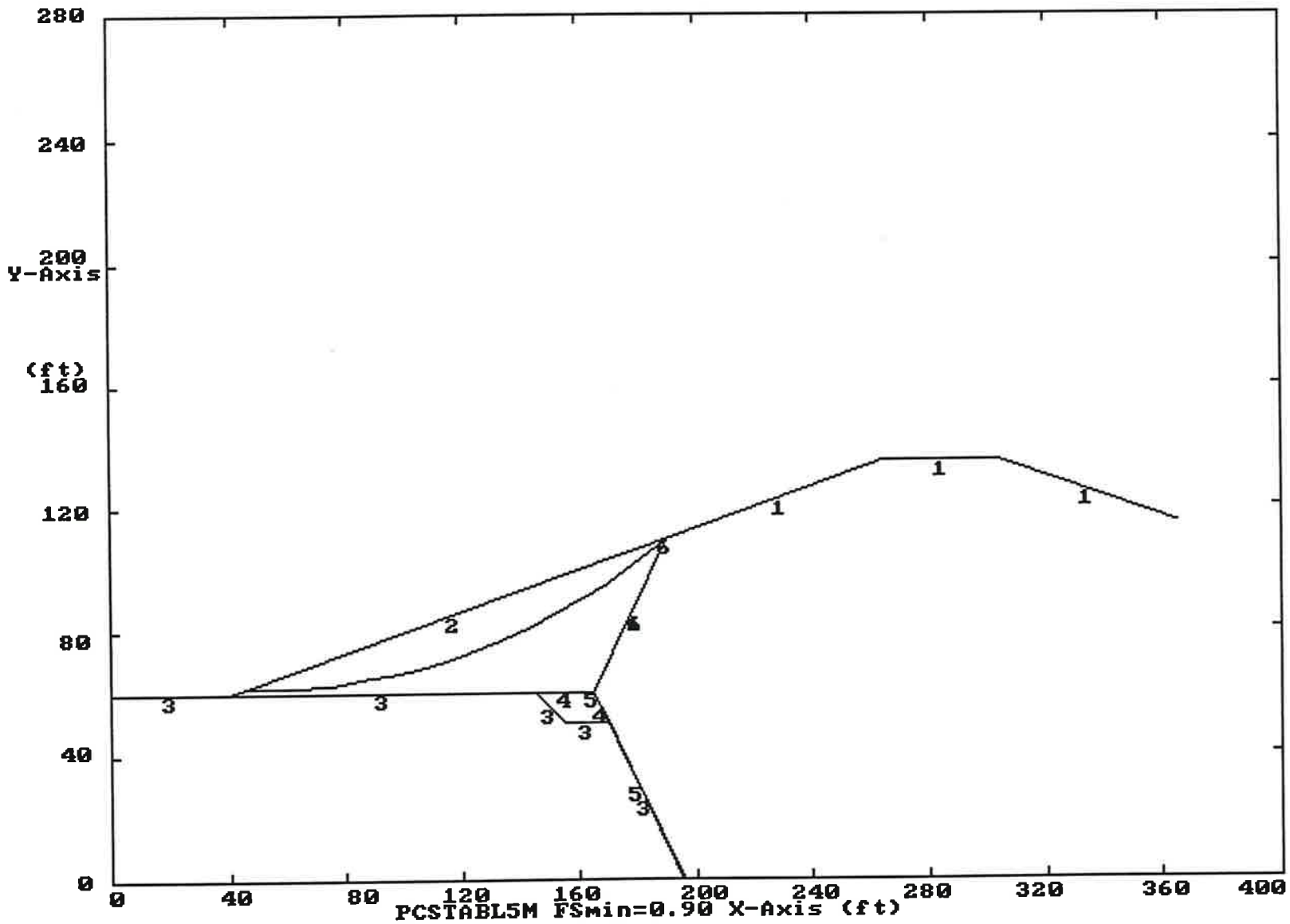
| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 59.47 | 66.49 |
| 2 | 69.42 | 65.46 |
| 3 | 79.40 | 64.74 |
| 4 | 89.39 | 64.32 |
| 5 | 99.39 | 64.21 |
| 6 | 109.38 | 64.40 |
| 7 | 119.37 | 64.90 |
| 8 | 129.34 | 65.70 |
| 9 | 139.28 | 66.80 |
| 10 | 149.18 | 68.21 |
| 11 | 159.03 | 69.92 |
| 12 | 168.83 | 71.93 |
| 13 | 178.56 | 74.23 |
| 14 | 188.21 | 76.83 |
| 15 | 197.79 | 79.73 |
| 16 | 207.26 | 82.92 |
| 17 | 216.64 | 86.39 |
| 18 | 225.91 | 90.15 |
| 19 | 235.06 | 94.19 |
| 20 | 244.08 | 98.50 |
| 21 | 252.96 | 103.09 |
| 22 | 261.70 | 107.95 |
| 23 | 270.29 | 113.07 |
| 24 | 278.72 | 118.45 |
| 25 | 286.98 | 124.09 |
| 26 | 295.06 | 129.98 |
| 27 | 301.54 | 135.00 |

Factor Of Safety For The Preceding Specified Surface = .864

1



FINAL SOUTH SLOPE 3:1 SEISMIC A.E.C. BEATTY, NEVADA
Surface #1-FB3AEQ.OUT. C:FB3AEQX.PLT By: WSB 03-08-96 10:45am



** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:45am |
| Run By: | WSB |
| Input Data Filename: | C:FB3AEQX |
| Output Filename: | C:FB3AEQX.OUT |
| Plotted Output Filename: | C:FB3AEQX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 3:1 SEISMIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 3 |
| 2 | 40.00 | 60.00 | 190.00 | 110.00 | 2 |
| 3 | 190.00 | 110.00 | 190.20 | 110.10 | 6 |
| 4 | 190.20 | 110.10 | 265.00 | 135.00 | 1 |
| 5 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 6 | 305.00 | 135.00 | 365.00 | 115.00 | 1 |
| 7 | 165.00 | 60.00 | 190.00 | 110.00 | 6 |
| 8 | 165.20 | 60.00 | 190.20 | 110.10 | 1 |
| 9 | 40.00 | 60.00 | 145.00 | 60.00 | 3 |
| 10 | 145.00 | 60.00 | 165.00 | 60.00 | 4 |
| 11 | 165.00 | 60.00 | 165.20 | 60.00 | 5 |
| 12 | 145.00 | 60.00 | 155.00 | 50.00 | 3 |
| 13 | 165.20 | 60.00 | 195.20 | .00 | 5 |
| 14 | 165.00 | 60.00 | 170.00 | 50.00 | 4 |
| 15 | 155.00 | 50.00 | 170.00 | 50.00 | 3 |
| 16 | 170.00 | 50.00 | 195.00 | .00 | 3 |

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .420 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

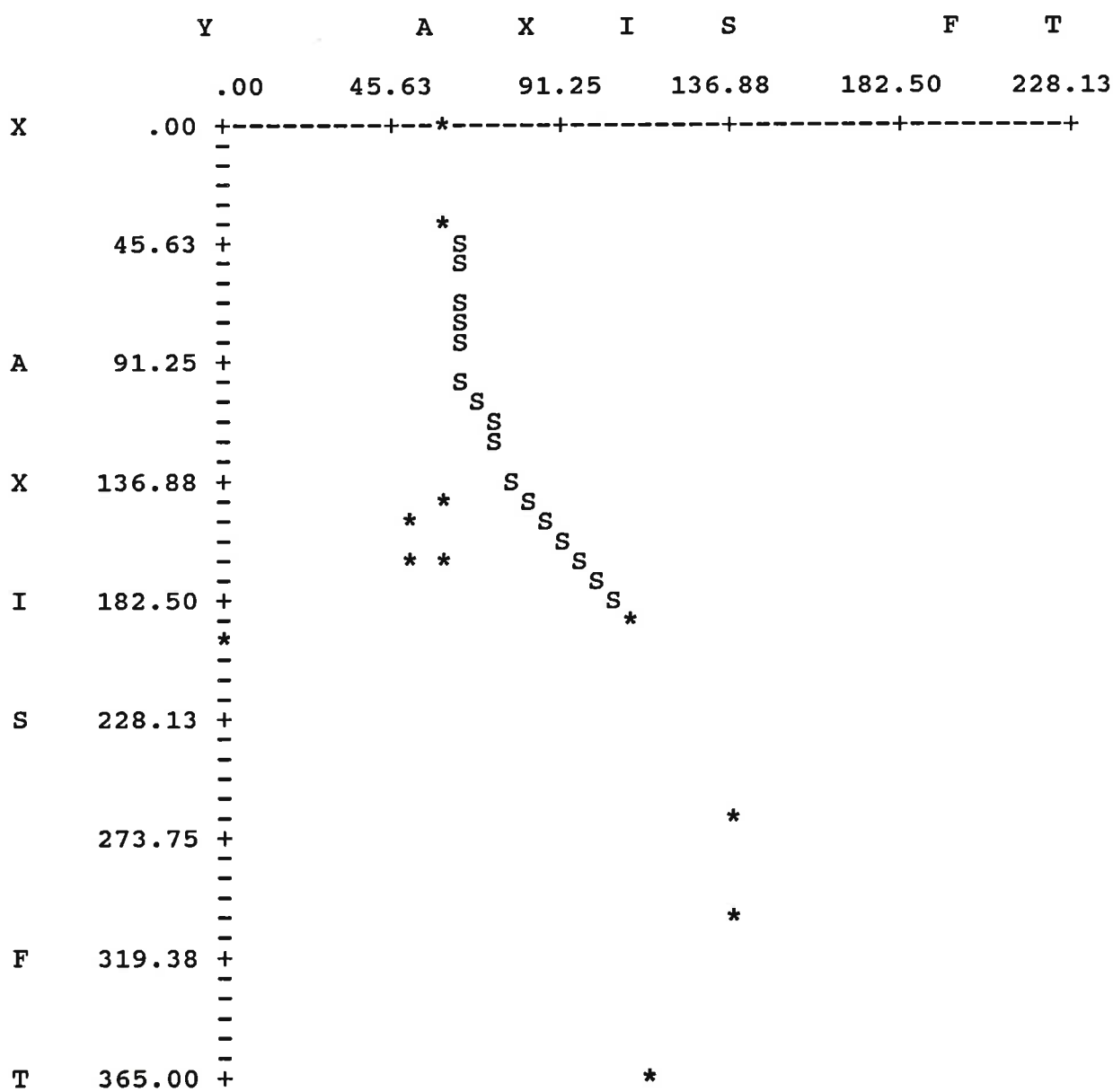
Cavitation Pressure = .0 psf

Trial Failure Surface Specified By 17 Coordinate Points

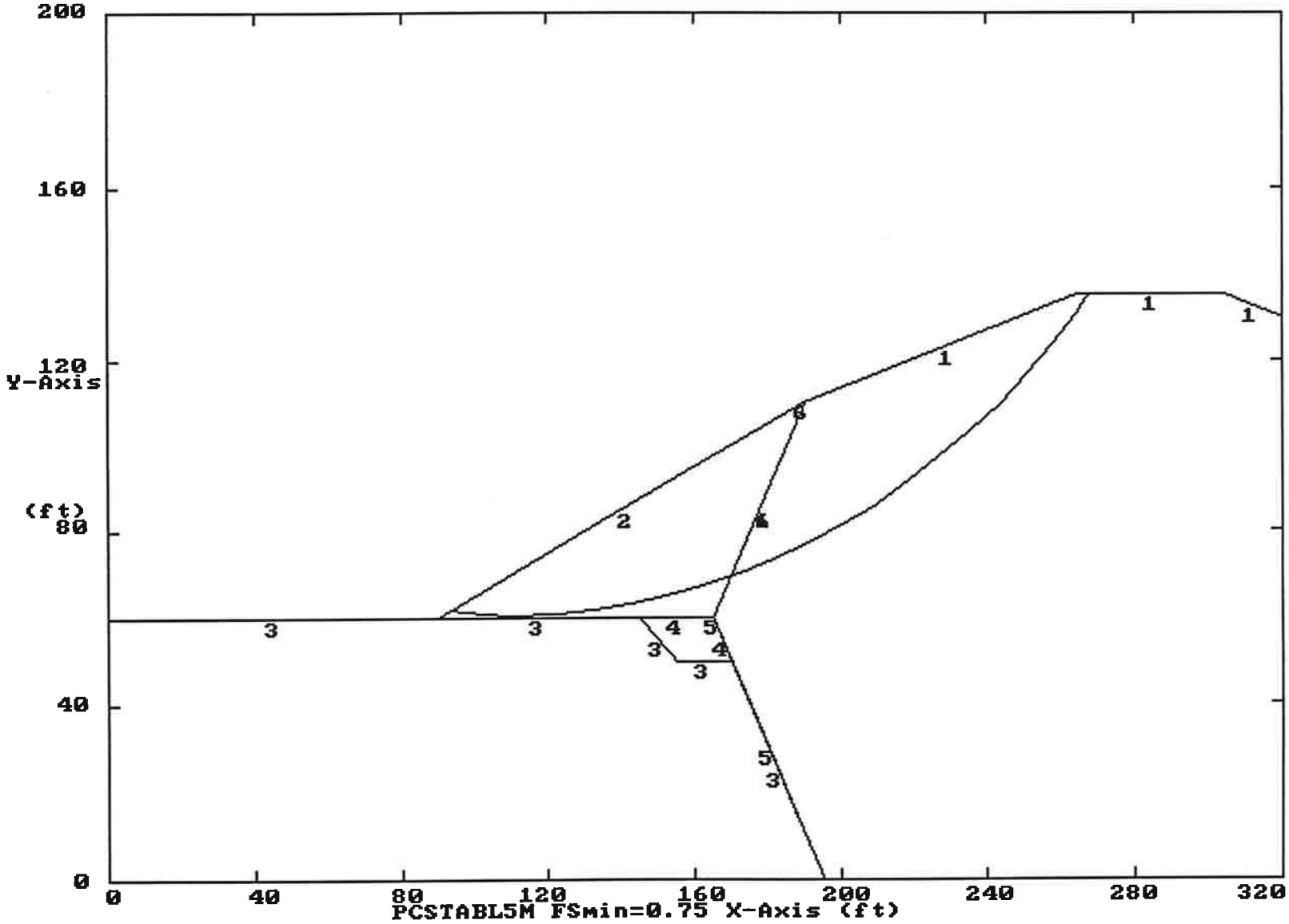
| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 45.79 | 61.93 |
| 2 | 55.79 | 61.85 |
| 3 | 65.78 | 62.22 |
| 4 | 75.75 | 63.05 |
| 5 | 85.67 | 64.31 |
| 6 | 95.52 | 66.02 |
| 7 | 105.29 | 68.17 |
| 8 | 114.95 | 70.76 |
| 9 | 124.48 | 73.78 |
| 10 | 133.87 | 77.23 |
| 11 | 143.09 | 81.09 |
| 12 | 152.13 | 85.36 |
| 13 | 160.97 | 90.03 |
| 14 | 169.60 | 95.10 |
| 15 | 177.98 | 100.55 |
| 16 | 186.11 | 106.37 |
| 17 | 191.35 | 110.48 |

Factor Of Safety For The Preceding Specified Surface = .901

1



FINAL SOUTH SLOPE 3:1 SEISMIC A.E.C. BEATTY, NEVADA
Surface #1-FB32EQ.OUT. C:FB32EQX.PLT By: WSB 03-08-96 10:40am



** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:40am |
| Run By: | WSB |
| Input Data Filename: | C:FB32EQX |
| Output Filename: | C:FB32EQX.OUT |
| Plotted Output Filename: | C:FB32EQX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 3:1 SEISMIC
A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 90.00 | 60.00 | 3 |
| 2 | 90.00 | 60.00 | 190.00 | 110.00 | 2 |
| 3 | 190.00 | 110.00 | 190.20 | 110.10 | 6 |
| 4 | 190.20 | 110.10 | 265.00 | 135.00 | 1 |
| 5 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 6 | 305.00 | 135.00 | 320.00 | 130.00 | 1 |
| 7 | 165.00 | 60.00 | 190.00 | 110.00 | 6 |
| 8 | 165.20 | 60.00 | 190.20 | 110.10 | 1 |
| 9 | 90.00 | 60.00 | 145.00 | 60.00 | 3 |
| 10 | 145.00 | 60.00 | 165.00 | 60.00 | 4 |
| 11 | 165.00 | 60.00 | 165.20 | 60.00 | 5 |
| 12 | 145.00 | 60.00 | 155.00 | 50.00 | 3 |
| 13 | 165.20 | 60.00 | 195.20 | .00 | 5 |
| 14 | 165.00 | 60.00 | 170.00 | 50.00 | 4 |
| 15 | 155.00 | 50.00 | 170.00 | 50.00 | 3 |
| 16 | 170.00 | 50.00 | 195.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .420 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

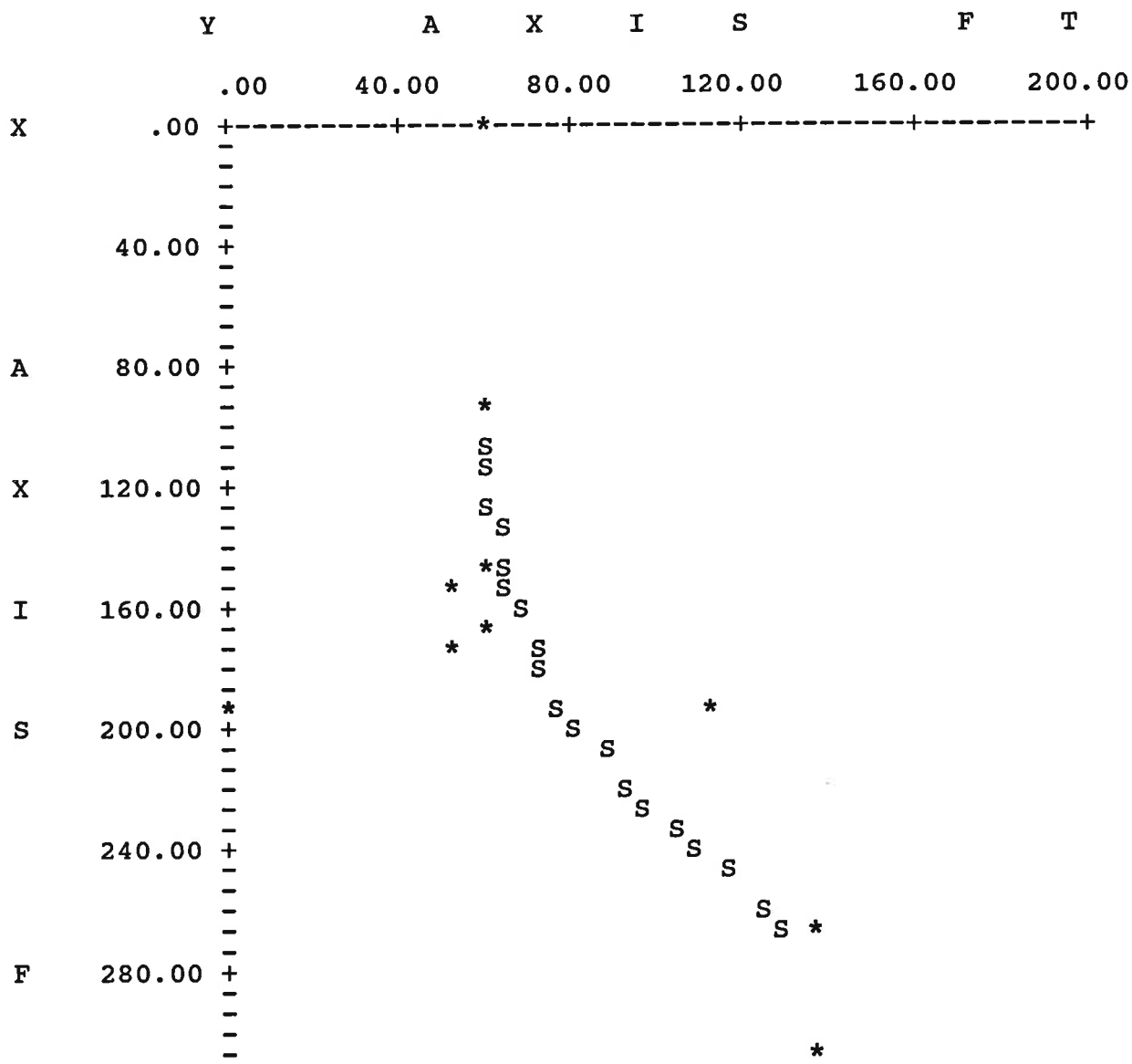
1

Trial Failure Surface Specified By 21 Coordinate Points

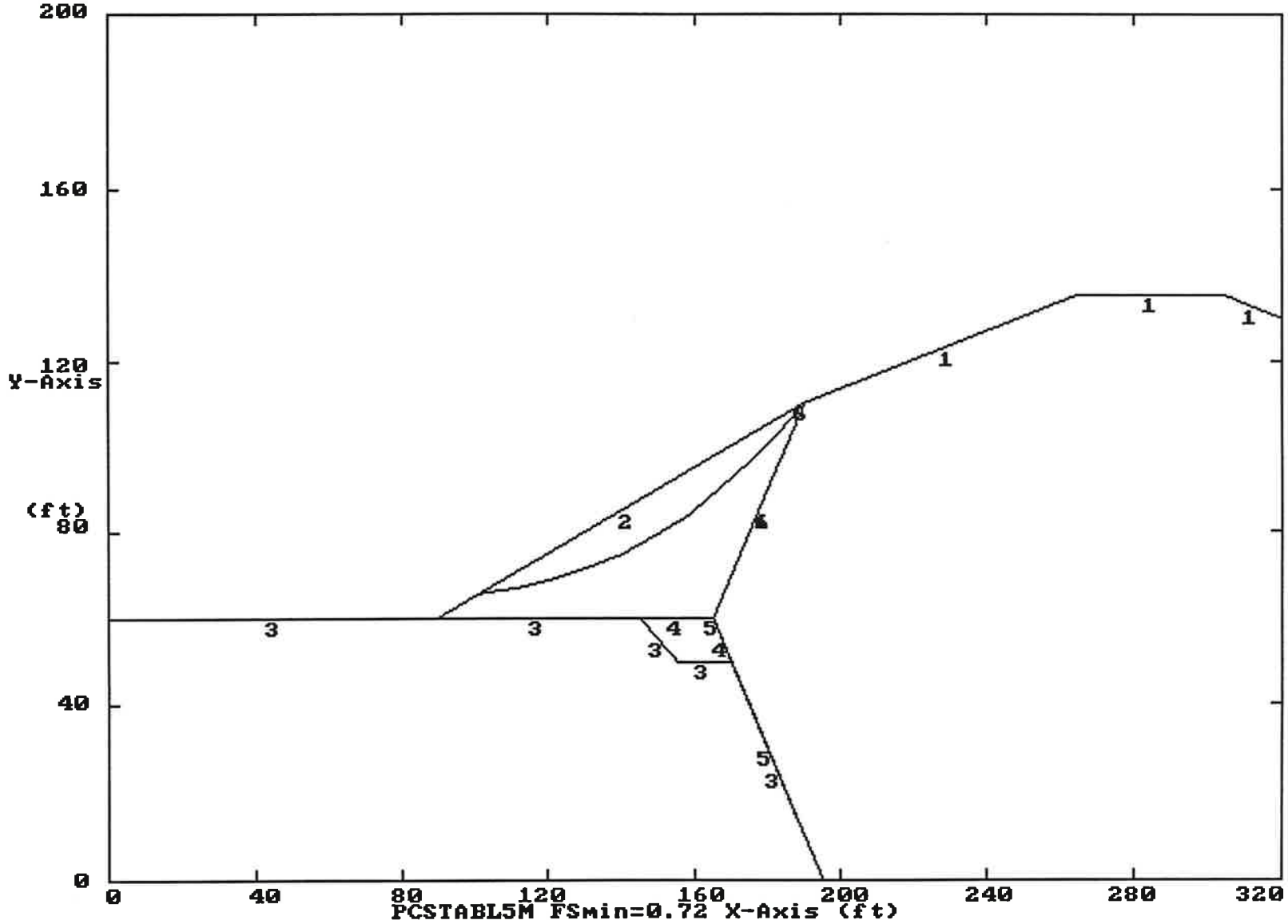
| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 93.68 | 61.84 |
| 2 | 103.66 | 61.16 |
| 3 | 113.66 | 60.98 |
| 4 | 123.65 | 61.30 |
| 5 | 133.62 | 62.12 |
| 6 | 143.54 | 63.43 |
| 7 | 153.37 | 65.23 |
| 8 | 163.10 | 67.52 |
| 9 | 172.71 | 70.30 |
| 10 | 182.17 | 73.55 |
| 11 | 191.45 | 77.27 |
| 12 | 200.54 | 81.45 |
| 13 | 209.40 | 86.07 |
| 14 | 218.03 | 91.13 |
| 15 | 226.39 | 96.61 |
| 16 | 234.47 | 102.51 |
| 17 | 242.24 | 108.80 |
| 18 | 249.69 | 115.47 |
| 19 | 256.80 | 122.50 |
| 20 | 263.55 | 129.88 |
| 21 | 267.79 | 135.00 |

Factor Of Safety For The Preceding Specified Surface = .746

1



FINAL SOUTH SLOPE 3:1, 2:1 BUTT, SEISMIC A.E.C. BEATTY, NEVADA
Surface #1-FB32AEQ.OUT. C:FB32AEQX.PLT By: WSB 03-08-96 10:38am



** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:38am |
| Run By: | WSB |
| Input Data Filename: | C:FB32AEQX |
| Output Filename: | C:FB32AEQX.OUT |
| Plotted Output Filename: | C:FB32AEQX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 3:1, 2:1 BUTT.
SEISMIC A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 90.00 | 60.00 | 3 |
| 2 | 90.00 | 60.00 | 190.00 | 110.00 | 2 |
| 3 | 190.00 | 110.00 | 190.20 | 110.10 | 6 |
| 4 | 190.20 | 110.10 | 265.00 | 135.00 | 1 |
| 5 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 6 | 305.00 | 135.00 | 320.00 | 130.00 | 1 |
| 7 | 165.00 | 60.00 | 190.00 | 110.00 | 6 |
| 8 | 165.20 | 60.00 | 190.20 | 110.10 | 1 |
| 9 | 90.00 | 60.00 | 145.00 | 60.00 | 3 |
| 10 | 145.00 | 60.00 | 165.00 | 60.00 | 4 |
| 11 | 165.00 | 60.00 | 165.20 | 60.00 | 5 |
| 12 | 145.00 | 60.00 | 155.00 | 50.00 | 3 |
| 13 | 165.20 | 60.00 | 195.20 | .00 | 5 |
| 14 | 165.00 | 60.00 | 170.00 | 50.00 | 4 |
| 15 | 155.00 | 50.00 | 170.00 | 50.00 | 3 |
| 16 | 170.00 | 50.00 | 195.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .420 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

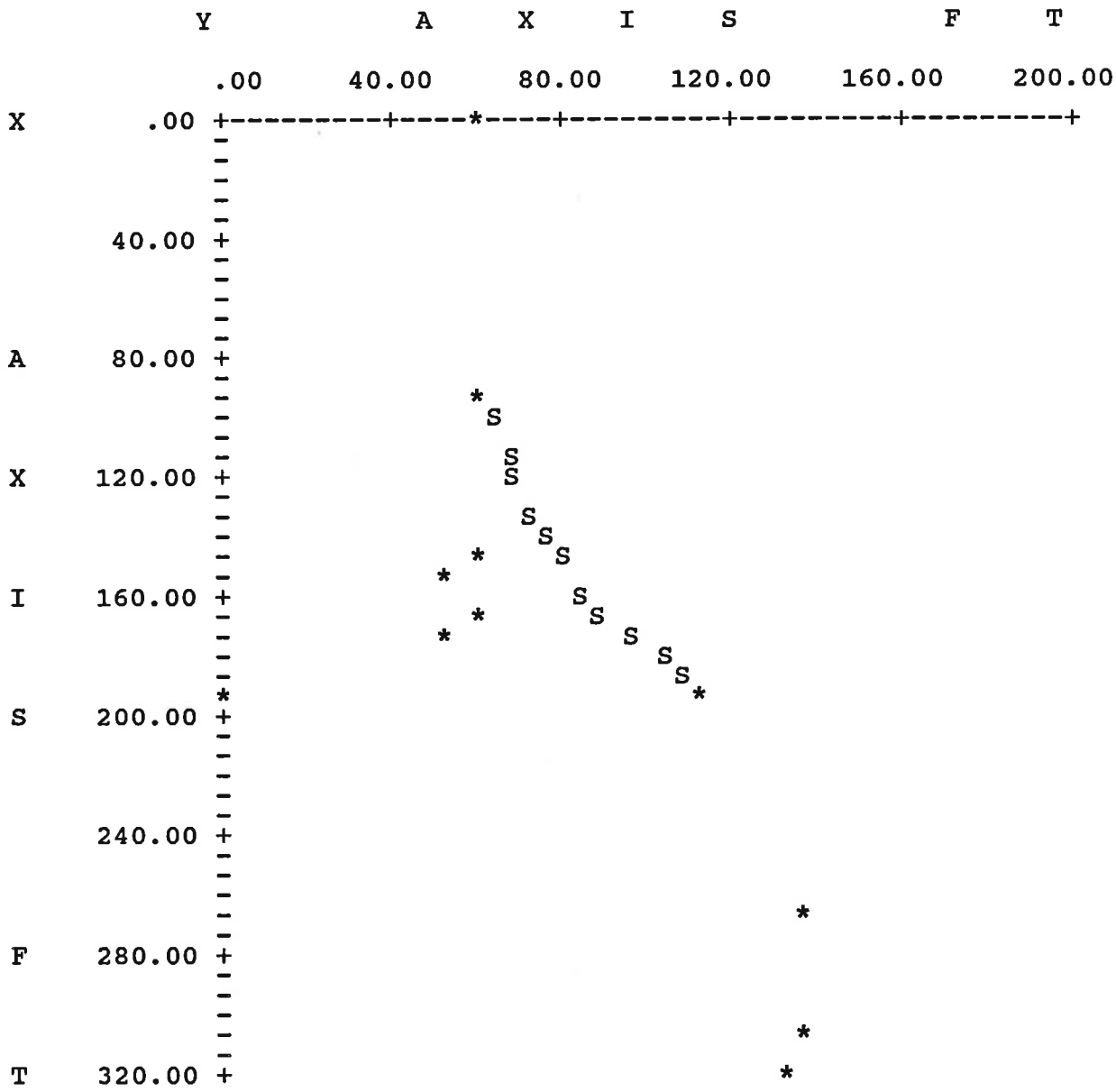
1

Trial Failure Surface Specified By 12 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 101.58 | 65.79 |
| 2 | 111.50 | 67.03 |
| 3 | 121.30 | 69.02 |
| 4 | 130.93 | 71.73 |
| 5 | 140.32 | 75.16 |
| 6 | 149.43 | 79.28 |
| 7 | 158.21 | 84.08 |
| 8 | 166.60 | 89.52 |
| 9 | 174.56 | 95.57 |
| 10 | 182.04 | 102.20 |
| 11 | 189.01 | 109.38 |
| 12 | 189.19 | 109.60 |

Factor Of Safety For The Preceding Specified Surface = .719

1



# ENVIRONMENTAL SOLUTIONS, INC.

By YSB Date 1/10/96 Subject SLOPE STABILITY EVALUATION Sheet No. ____ of ____

Chkd. By ____ Date ____ FINAL CONFIGURATION Proj. No. 95-234

ATTACHMENT D

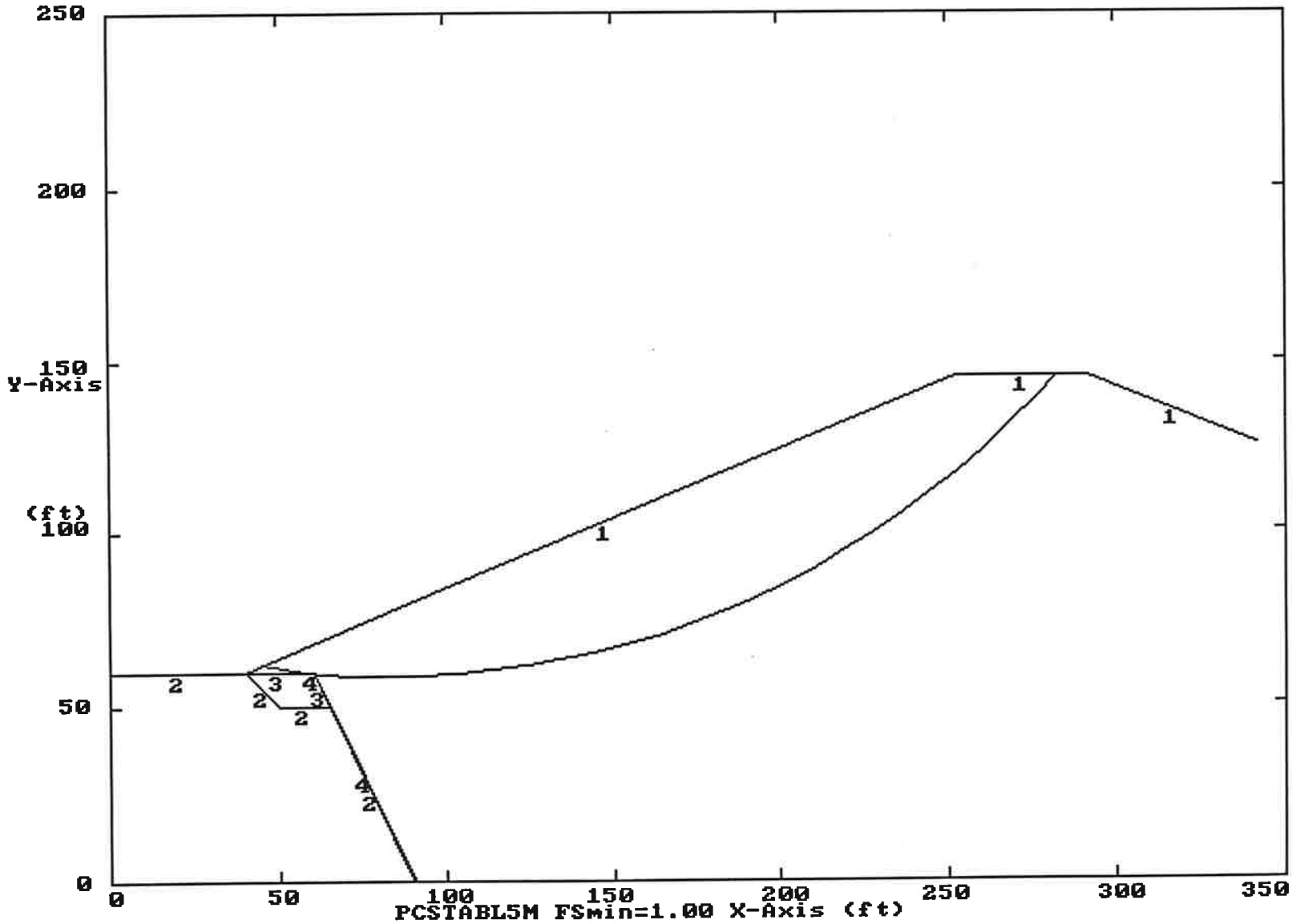
STABLSM COMPUTER OUTPUT

SEISMIC YIELD CASE



Recyclable and made from recycled paper.

FINAL NORTH SLOPE 2.5:1 SEISMIC YIELD A.E.C. BEATTY, NEVADA a=.24g
Surface #1-FA25EQY.OUT. C:FA25EQYX.PLT By: WSB 02-18-96 5:10pm



** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 02-18-96 |
| Time of Run: | 5:10pm |
| Run By: | WSB |
| Input Data Filename: | C:FA25EQYX |
| Output Filename: | C:FA25EQYX.OUT |
| Plotted Output Filename: | C:FA25EQYX.PLT |

PROBLEM DESCRIPTION FINAL NORTH SLOPE 2.5:1 SEISMIC YIELD
A.E.C. BEATTY, NEVADA a=.24g

BOUNDARY COORDINATES

4 Top Boundaries
11 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 2 |
| 2 | 40.00 | 60.00 | 252.50 | 145.00 | 1 |
| 3 | 252.50 | 145.00 | 292.50 | 145.00 | 1 |
| 4 | 292.50 | 145.00 | 342.50 | 125.00 | 1 |
| 5 | 40.00 | 60.00 | 60.00 | 60.00 | 3 |
| 6 | 60.00 | 60.00 | 60.20 | 60.00 | 4 |
| 7 | 40.00 | 60.00 | 50.00 | 50.00 | 2 |
| 8 | 60.20 | 60.00 | 90.20 | .00 | 4 |
| 9 | 60.00 | 60.00 | 65.00 | 50.00 | 3 |
| 10 | 50.00 | 50.00 | 65.00 | 50.00 | 2 |
| 11 | 65.00 | 50.00 | 90.00 | .00 | 2 |

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .240 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

1

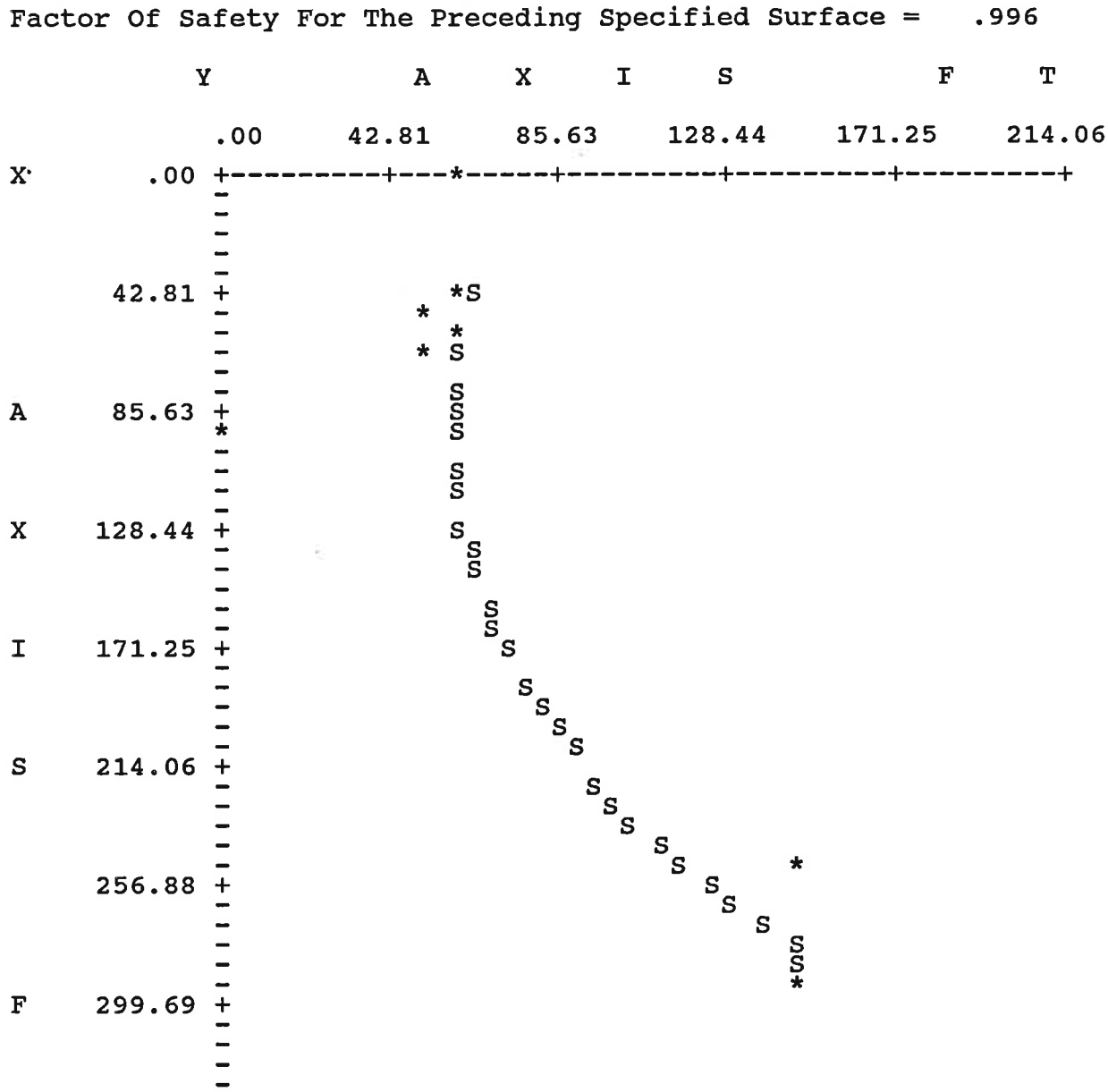
Trial Failure Surface Specified By 28 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 45.26 | 62.10 |
| 2 | 55.17 | 60.72 |

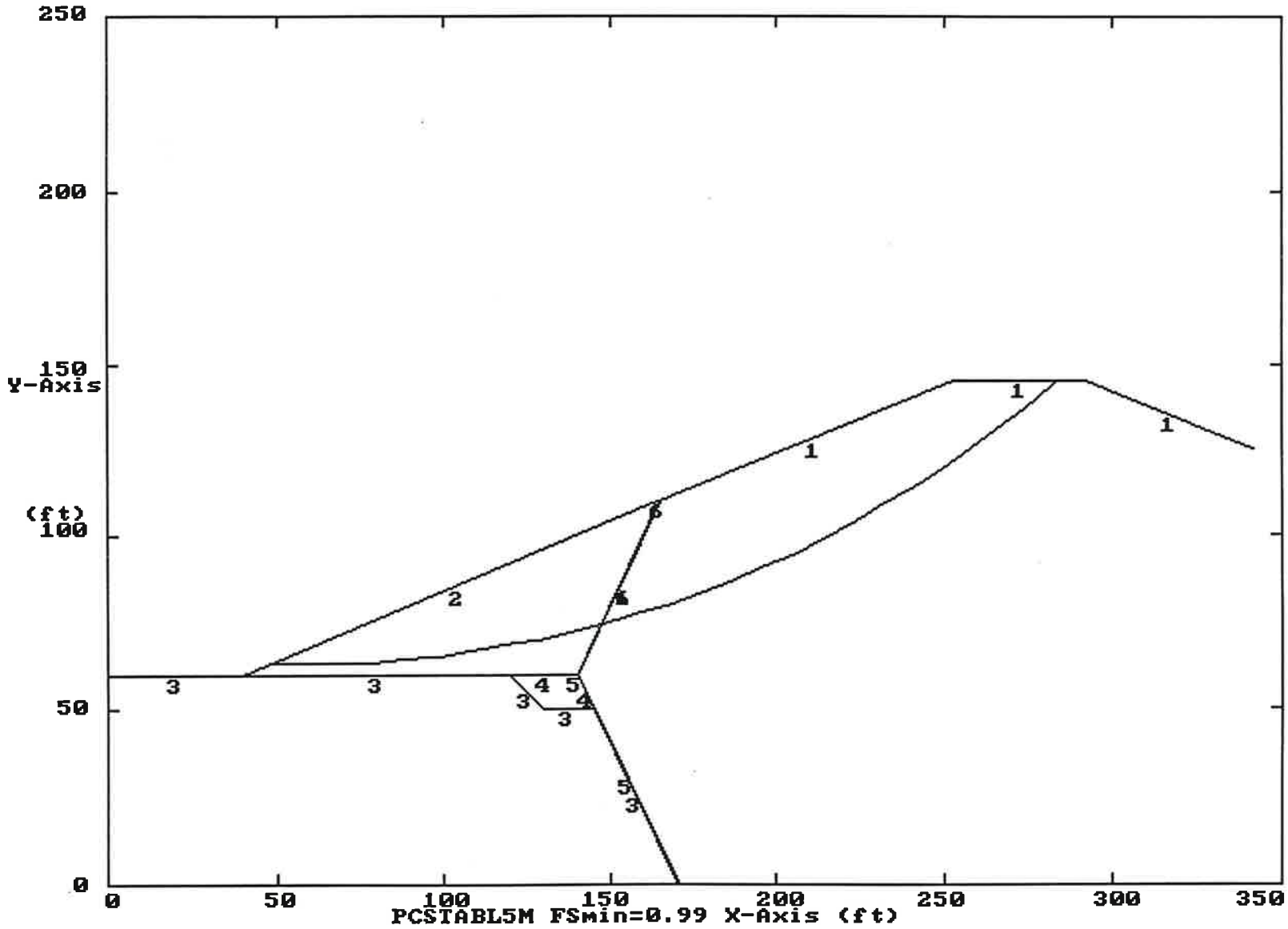
| | | |
|---|---|---|
| 3 | 65.12 | 59.72 |
| 4 | 75.10 | 59.09 |
| 5 | 85.09 | 58.83 |
| 6 | 95.09 | 58.95 |
| 7 | 105.08 | 59.45 |
| 8 | 115.04 | 60.33 |
| 9 | 124.96 | 61.58 |
| 10 | 134.83 | 63.21 |
| 11 | 144.63 | 65.20 |
| 12 | 154.35 | 67.57 |
| 13 | 163.97 | 70.29 |
| 14 | 173.48 | 73.38 |
| 15 | 182.86 | 76.83 |
| 16 | 192.12 | 80.63 |
| 17 | 201.22 | 84.77 |
| 18 | 210.15 | 89.26 |
| 19 | 218.92 | 94.07 |
| 20 | 227.49 | 99.22 |
| 21 | 235.87 | 104.68 |
| 22 | 244.03 | 110.46 |
| 23 | 251.97 | 116.54 |
| 24 | 259.67 | 122.92 |
| 25 | 267.13 | 129.58 |
| 26 | 274.33 | 136.52 |
| 27 | 281.27 | 143.72 |
| 28 | 282.40 | 145.00 |

Factor Of Safety For The Preceding Specified Surface = .996

1



FINAL SOUTH SLOPE 2.5:1 SEISMIC YIELD A.E.C. BEATTY, NEVADA a=.27g
Surface #1-FB25EQY.OUT. C:FB25EQYX.PLT By: WSB 03-08-96 10:31am



** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:31am |
| Run By: | WSB |
| Input Data Filename: | C:FB25EQYX |
| Output Filename: | C:FB25EQYX.OUT |
| Plotted Output Filename: | C:FB25EQYX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 2.5:1 SEISMIC YIELD
A.E.C. BEATTY, NEVADA a=.27g

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 3 |
| 2 | 40.00 | 60.00 | 165.00 | 110.00 | 2 |
| 3 | 165.00 | 110.00 | 165.20 | 110.10 | 6 |
| 4 | 165.20 | 110.10 | 252.50 | 145.00 | 1 |
| 5 | 252.50 | 145.00 | 292.50 | 145.00 | 1 |
| 6 | 292.50 | 145.00 | 342.50 | 125.00 | 1 |
| 7 | 140.00 | 60.00 | 165.00 | 110.00 | 6 |
| 8 | 140.20 | 60.00 | 165.20 | 110.10 | 1 |
| 9 | 40.00 | 60.00 | 120.00 | 60.00 | 3 |
| 10 | 120.00 | 60.00 | 140.00 | 60.00 | 4 |
| 11 | 140.00 | 60.00 | 140.20 | 60.00 | 5 |
| 12 | 120.00 | 60.00 | 130.00 | 50.00 | 3 |
| 13 | 140.20 | 60.00 | 170.20 | .00 | 5 |
| 14 | 140.00 | 60.00 | 145.00 | 50.00 | 4 |
| 15 | 130.00 | 50.00 | 145.00 | 50.00 | 3 |
| 16 | 145.00 | 50.00 | 170.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | .5 | .5 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | .5 | .5 | .0 | 14.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .270 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

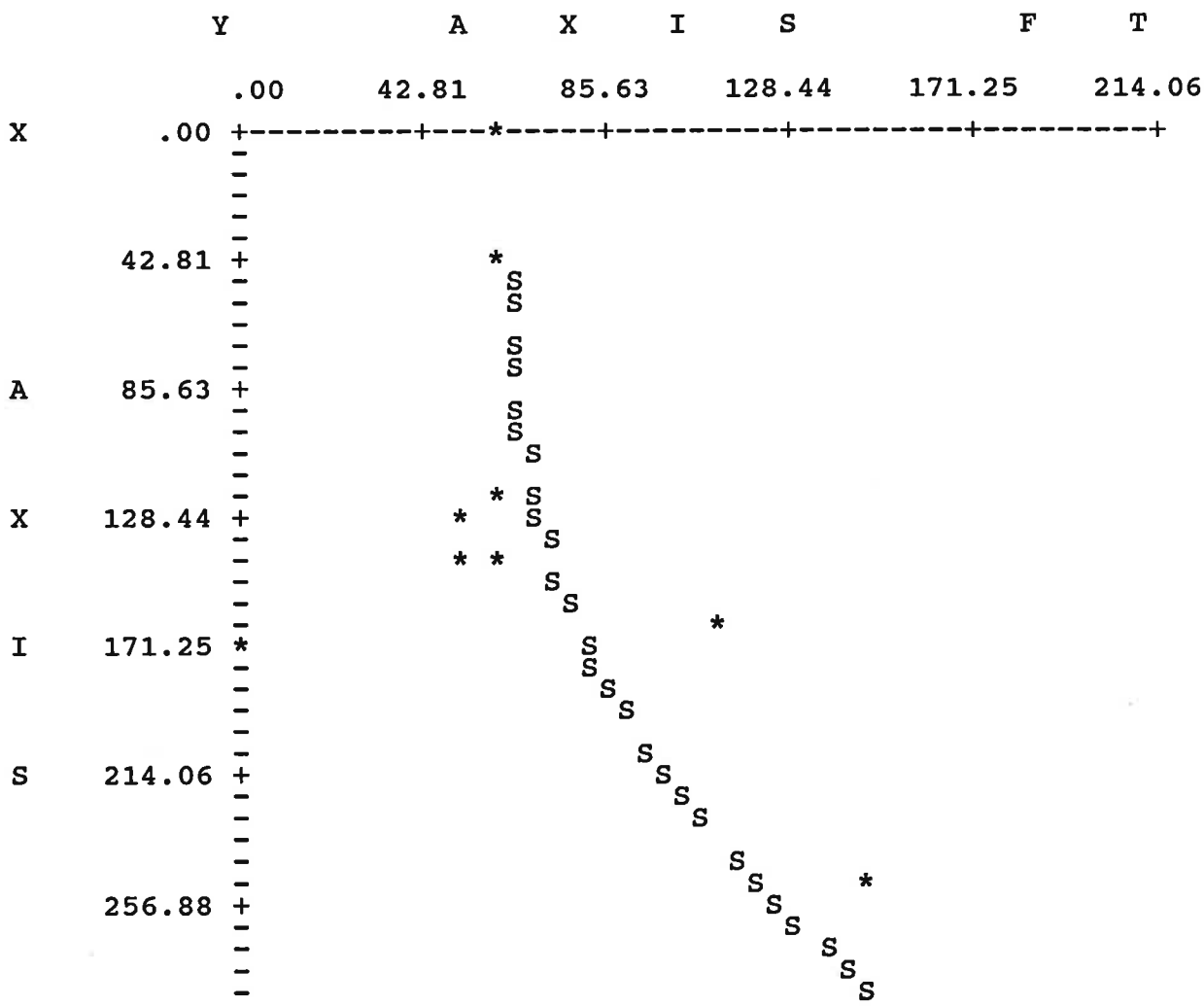
1

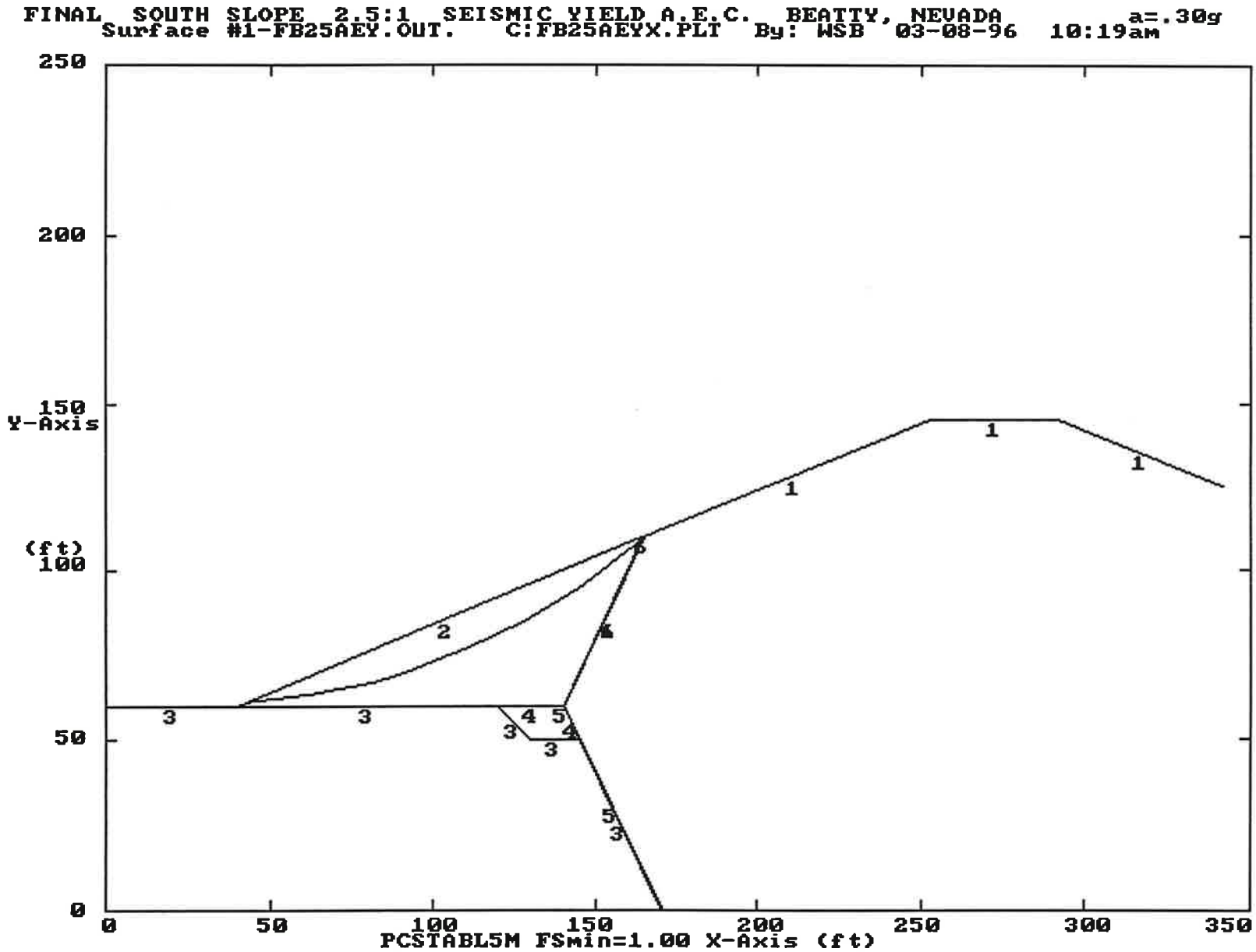
Trial Failure Surface Specified By 27 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 49.47 | 63.79 |
| 2 | 59.47 | 63.56 |
| 3 | 69.47 | 63.62 |
| 4 | 79.47 | 63.97 |
| 5 | 89.44 | 64.62 |
| 6 | 99.40 | 65.56 |
| 7 | 109.32 | 66.79 |
| 8 | 119.21 | 68.31 |
| 9 | 129.04 | 70.12 |
| 10 | 138.82 | 72.23 |
| 11 | 148.53 | 74.61 |
| 12 | 158.17 | 77.28 |
| 13 | 167.72 | 80.24 |
| 14 | 177.18 | 83.47 |
| 15 | 186.55 | 86.98 |
| 16 | 195.80 | 90.76 |
| 17 | 204.95 | 94.82 |
| 18 | 213.96 | 99.14 |
| 19 | 222.85 | 103.72 |
| 20 | 231.60 | 108.56 |
| 21 | 240.21 | 113.66 |
| 22 | 248.66 | 119.00 |
| 23 | 256.94 | 124.60 |
| 24 | 265.07 | 130.43 |
| 25 | 273.01 | 136.50 |
| 26 | 280.78 | 142.80 |
| 27 | 283.33 | 145.00 |

Factor Of Safety For The Preceding Specified Surface = .992

1




FINAL SOUTH SLOPE 2.5:1 SEISMIC YIELD A.E.C. BEATTY, NEVADA a=.30g
Surface #1-FB25AEY.OUT. C:FB25AEYX.PLT By: WSB 03-08-96 10:19am
Y-Axis
(ft)
250
200
150
100
50
0
0
50
100
150
200
250
300
350
PCSTABL5M FSmin=1.00 X-Axis (ft)

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:19am |
| Run By: | WSB |
| Input Data Filename: | C:FB25AEYX |
| Output Filename: | C:FB25AEYX.OUT |
| Plotted Output Filename: | C:FB25AEYX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 2.5:1 SEISMIC YIELD
A.E.C. BEATTY, NEVADA a=.30g

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 3 |
| 2 | 40.00 | 60.00 | 165.00 | 110.00 | 2 |
| 3 | 165.00 | 110.00 | 165.20 | 110.10 | 6 |
| 4 | 165.20 | 110.10 | 252.50 | 145.00 | 1 |
| 5 | 252.50 | 145.00 | 292.50 | 145.00 | 1 |
| 6 | 292.50 | 145.00 | 342.50 | 125.00 | 1 |
| 7 | 140.00 | 60.00 | 165.00 | 110.00 | 6 |
| 8 | 140.20 | 60.00 | 165.20 | 110.10 | 1 |
| 9 | 40.00 | 60.00 | 120.00 | 60.00 | 3 |
| 10 | 120.00 | 60.00 | 140.00 | 60.00 | 4 |
| 11 | 140.00 | 60.00 | 140.20 | 60.00 | 5 |
| 12 | 120.00 | 60.00 | 130.00 | 50.00 | 3 |
| 13 | 140.20 | 60.00 | 170.20 | .00 | 5 |
| 14 | 140.00 | 60.00 | 145.00 | 50.00 | 4 |
| 15 | 130.00 | 50.00 | 145.00 | 50.00 | 3 |
| 16 | 145.00 | 50.00 | 170.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .300 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

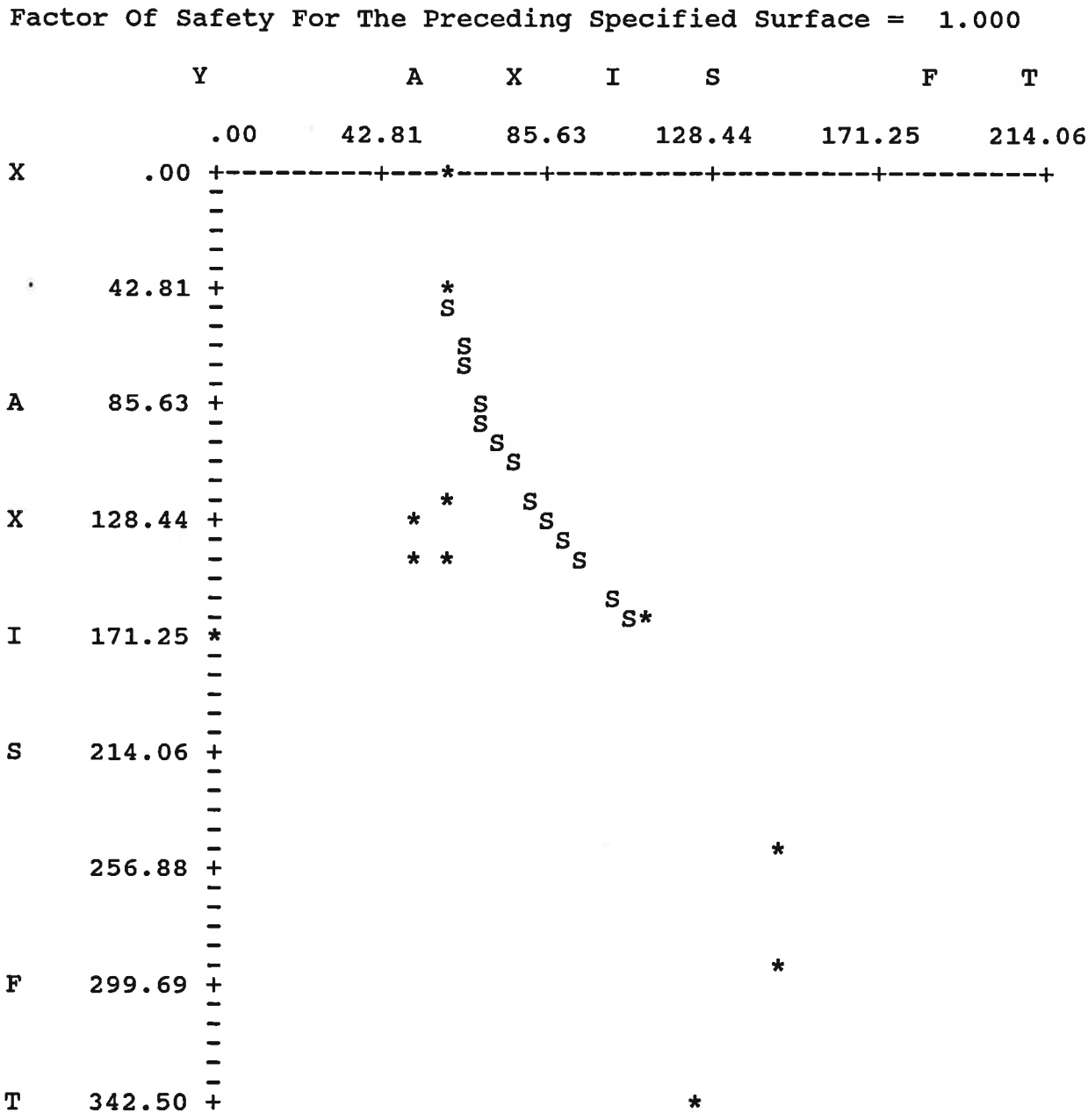
1

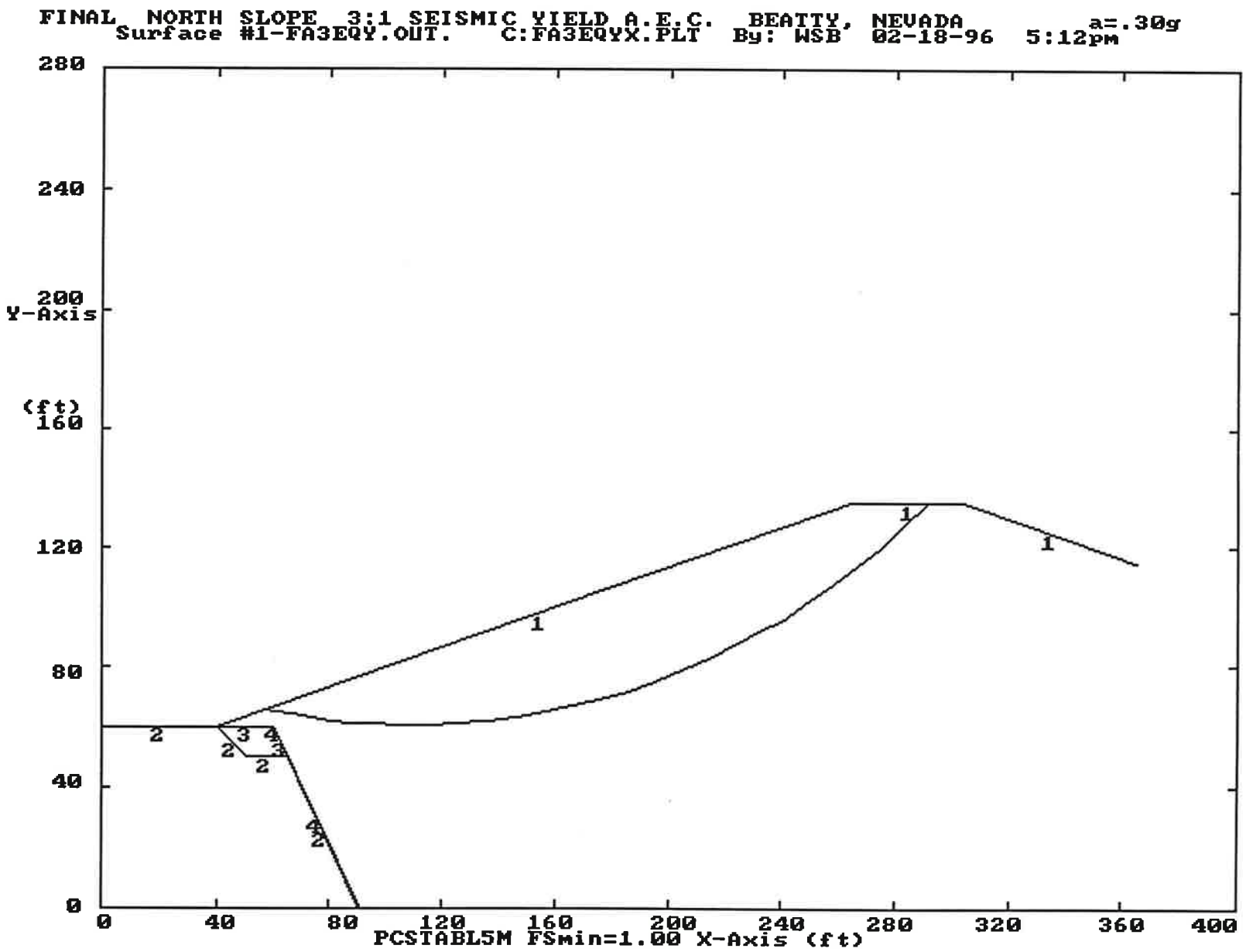
Trial Failure Surface Specified By 15 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 42.63 | 61.05 |
| 2 | 52.59 | 61.93 |
| 3 | 62.50 | 63.28 |
| 4 | 72.34 | 65.09 |
| 5 | 82.08 | 67.36 |
| 6 | 91.70 | 70.09 |
| 7 | 101.18 | 73.26 |
| 8 | 110.50 | 76.88 |
| 9 | 119.64 | 80.93 |
| 10 | 128.58 | 85.41 |
| 11 | 137.30 | 90.31 |
| 12 | 145.79 | 95.61 |
| 13 | 154.01 | 101.30 |
| 14 | 161.95 | 107.37 |
| 15 | 165.20 | 110.10 |

Factor Of Safety For The Preceding Specified Surface = 1.000

1




FINAL NORTH SLOPE 3:1 SEISMIC YIELD A.E.C. BEATTY, NEVADA a=.30g
Surface #1-FA3EQY.OUT. C:FA3EQYX.PLT By: WSB 02-18-96 5:12pm
Y-Axis
(ft)
280
240
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
360
400
PCSTABL5M FSmin=1.00 X-Axis (ft)
1
1
1
2
2
2
3
4
4
2

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 02-18-96 |
| Time of Run: | 5:12pm |
| Run By: | WSB |
| Input Data Filename: | C:FA3EQYX |
| Output Filename: | C:FA3EQYX.OUT |
| Plotted Output Filename: | C:FA3EQYX.PLT |

PROBLEM DESCRIPTION FINAL NORTH SLOPE 3:1 SEISMIC YIELD
A.E.C. BEATTY, NEVADA a=.30g

BOUNDARY COORDINATES

4 Top Boundaries
11 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 2 |
| 2 | 40.00 | 60.00 | 265.00 | 135.00 | 1 |
| 3 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 4 | 305.00 | 135.00 | 365.00 | 115.00 | 1 |
| 5 | 40.00 | 60.00 | 60.00 | 60.00 | 3 |
| 6 | 60.00 | 60.00 | 60.20 | 60.00 | 4 |
| 7 | 40.00 | 60.00 | 50.00 | 50.00 | 2 |
| 8 | 60.20 | 60.00 | 90.20 | .00 | 4 |
| 9 | 60.00 | 60.00 | 65.00 | 50.00 | 3 |
| 10 | 50.00 | 50.00 | 65.00 | 50.00 | 2 |
| 11 | 65.00 | 50.00 | 90.00 | .00 | 2 |

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .300 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

1

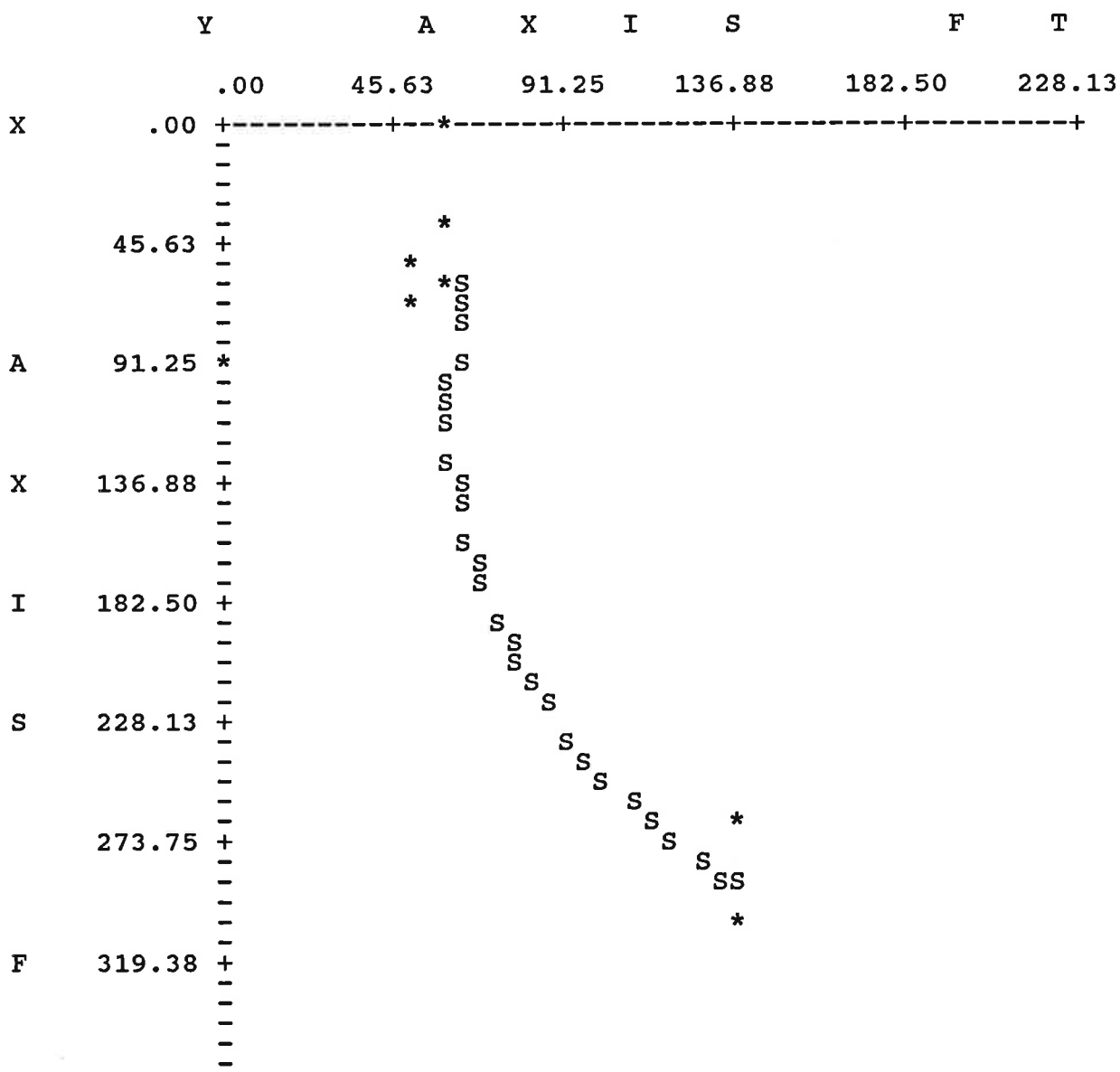
Trial Failure Surface Specified By 27 Coordinate Points

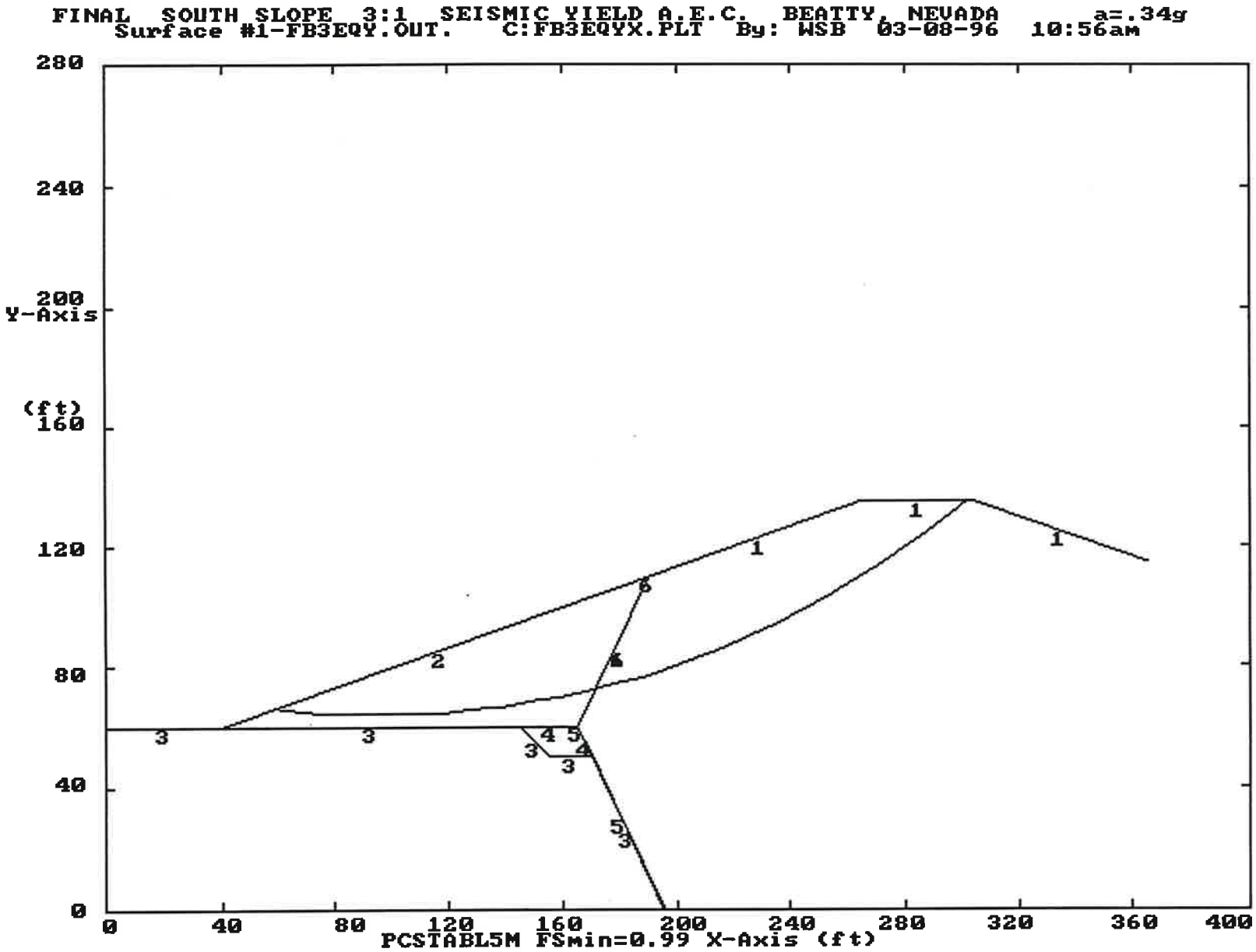
| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 57.90 | 65.97 |
| 2 | 67.73 | 64.14 |

| | | |
|---|---|---|
| 3 | 77.62 | 62.70 |
| 4 | 87.57 | 61.64 |
| 5 | 97.54 | 60.97 |
| 6 | 107.54 | 60.68 |
| 7 | 117.54 | 60.78 |
| 8 | 127.53 | 61.26 |
| 9 | 137.49 | 62.14 |
| 10 | 147.41 | 63.39 |
| 11 | 157.27 | 65.03 |
| 12 | 167.07 | 67.05 |
| 13 | 176.78 | 69.45 |
| 14 | 186.39 | 72.22 |
| 15 | 195.88 | 75.36 |
| 16 | 205.25 | 78.87 |
| 17 | 214.47 | 82.73 |
| 18 | 223.54 | 86.95 |
| 19 | 232.43 | 91.51 |
| 20 | 241.15 | 96.42 |
| 21 | 249.66 | 101.66 |
| 22 | 257.97 | 107.23 |
| 23 | 266.06 | 113.11 |
| 24 | 273.91 | 119.30 |
| 25 | 281.52 | 125.79 |
| 26 | 288.87 | 132.57 |
| 27 | 291.31 | 135.00 |

Factor Of Safety For The Preceding Specified Surface = .999

1




FINAL SOUTH SLOPE 3:1 SEISMIC YIELD A.E.C. BEATTY, NEVADA a=.34g
Surface #1-FB3EQY.OUT. C:FB3EQYX.PLT By: WSB 03-08-96 10:56am
Y-Axis
(ft)
280
240
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
360
400
PCSTABL5M FSmin=0.99 X-Axis (ft)

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:56am |
| Run By: | WSB |
| Input Data Filename: | C:FB3EQYX |
| Output Filename: | C:FB3EQYX.OUT |
| Plotted Output Filename: | C:FB3EQYX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 3:1 SEISMIC YIELD
A.E.C. BEATTY, NEVADA a=.34g

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 3 |
| 2 | 40.00 | 60.00 | 190.00 | 110.00 | 2 |
| 3 | 190.00 | 110.00 | 190.20 | 110.10 | 6 |
| 4 | 190.20 | 110.10 | 265.00 | 135.00 | 1 |
| 5 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 6 | 305.00 | 135.00 | 365.00 | 115.00 | 1 |
| 7 | 165.00 | 60.00 | 190.00 | 110.00 | 6 |
| 8 | 165.20 | 60.00 | 190.20 | 110.10 | 1 |
| 9 | 40.00 | 60.00 | 145.00 | 60.00 | 3 |
| 10 | 145.00 | 60.00 | 165.00 | 60.00 | 4 |
| 11 | 165.00 | 60.00 | 165.20 | 60.00 | 5 |
| 12 | 145.00 | 60.00 | 155.00 | 50.00 | 3 |
| 13 | 165.20 | 60.00 | 195.20 | .00 | 5 |
| 14 | 165.00 | 60.00 | 170.00 | 50.00 | 4 |
| 15 | 155.00 | 50.00 | 170.00 | 50.00 | 3 |
| 16 | 170.00 | 50.00 | 195.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .340 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

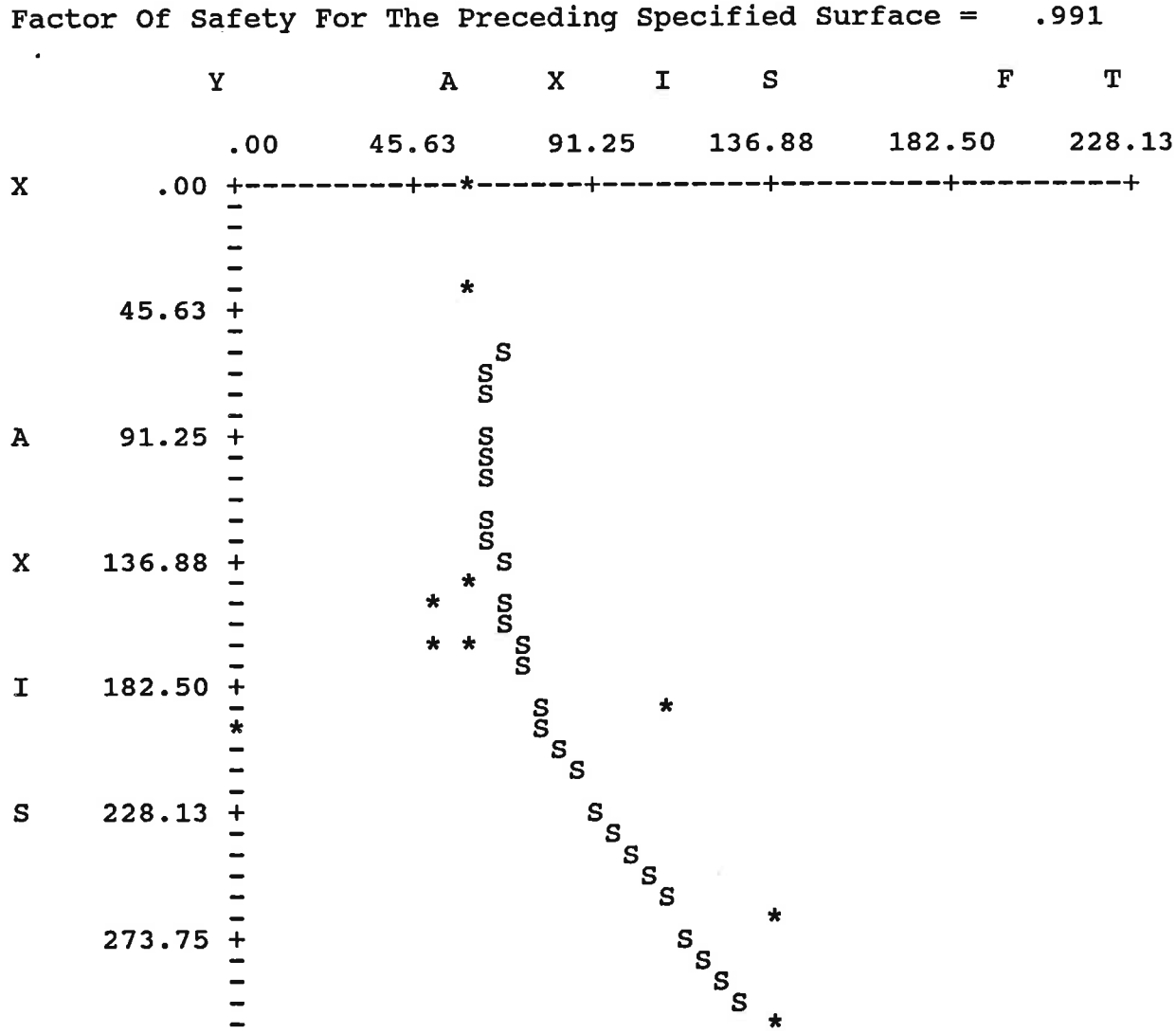
1

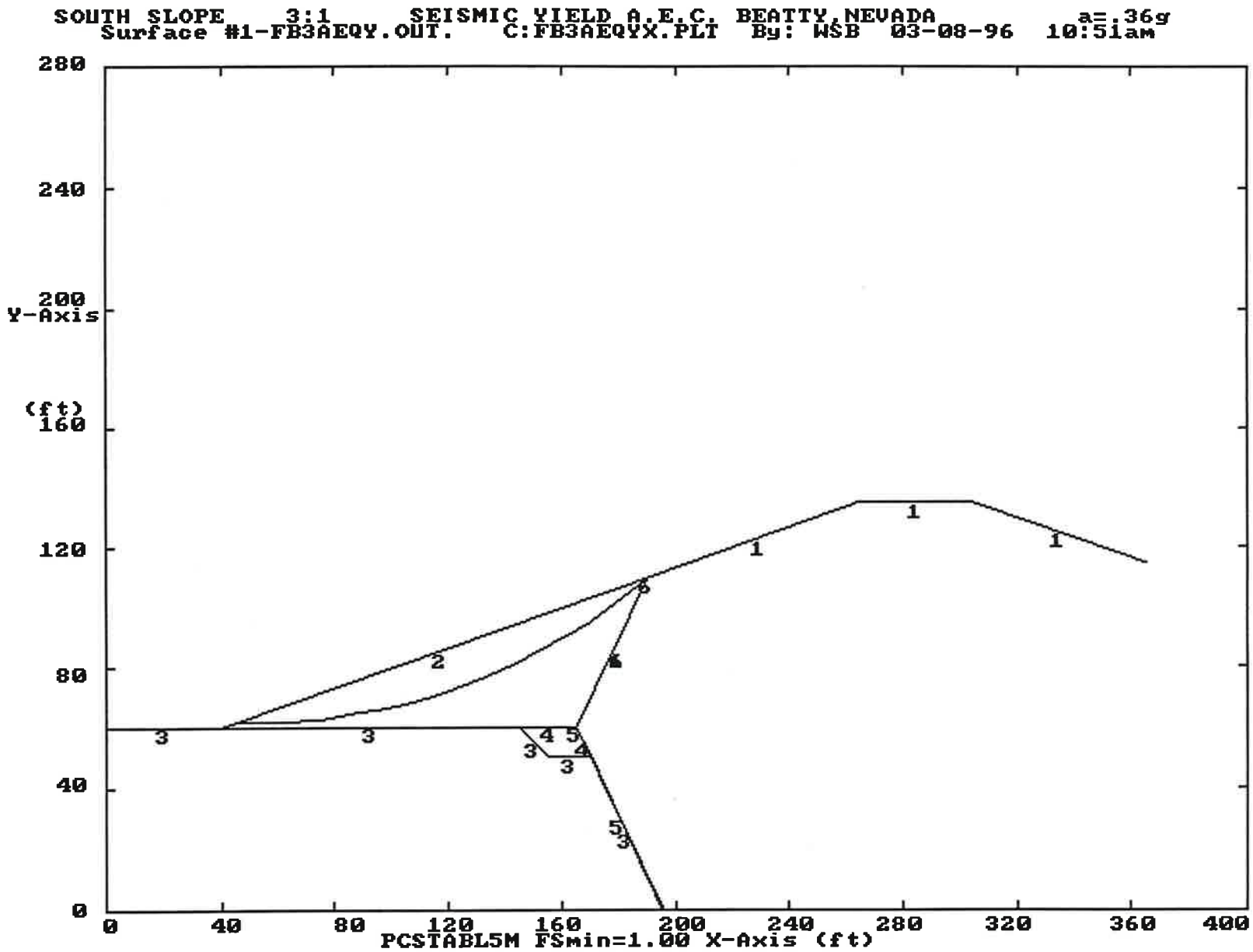
Trial Failure Surface Specified By 27 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 59.47 | 66.49 |
| 2 | 69.42 | 65.46 |
| 3 | 79.40 | 64.74 |
| 4 | 89.39 | 64.32 |
| 5 | 99.39 | 64.21 |
| 6 | 109.38 | 64.40 |
| 7 | 119.37 | 64.90 |
| 8 | 129.34 | 65.70 |
| 9 | 139.28 | 66.80 |
| 10 | 149.18 | 68.21 |
| 11 | 159.03 | 69.92 |
| 12 | 168.83 | 71.93 |
| 13 | 178.56 | 74.23 |
| 14 | 188.21 | 76.83 |
| 15 | 197.79 | 79.73 |
| 16 | 207.26 | 82.92 |
| 17 | 216.64 | 86.39 |
| 18 | 225.91 | 90.15 |
| 19 | 235.06 | 94.19 |
| 20 | 244.08 | 98.50 |
| 21 | 252.96 | 103.09 |
| 22 | 261.70 | 107.95 |
| 23 | 270.29 | 113.07 |
| 24 | 278.72 | 118.45 |
| 25 | 286.98 | 124.09 |
| 26 | 295.06 | 129.98 |
| 27 | 301.54 | 135.00 |

Factor Of Safety For The Preceding Specified Surface = .991

1




SOUTH SLOPE 3:1 SEISMIC YIELD A.E.C. BEATTY,NEVADA a=.36g
Surface #1-FB3AEQY.OUT. C:FB3AEQYX.PLT By: WSB 03-08-96 10:51am
Y-Axis
(ft)
280
240
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
360
400
PCSTABL5M FSmin=1.00 X-Axis (ft)

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:51am |
| Run By: | WSB |
| Input Data Filename: | C:FB3AEQYX |
| Output Filename: | C:FB3AEQYX.OUT |
| Plotted Output Filename: | C:FB3AEQYX.PLT |

PROBLEM DESCRIPTION SOUTH SLOPE 3:1 SEISMIC YIELD
A.E.C. BEATTY,NEVADA a=.36g

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 40.00 | 60.00 | 3 |
| 2 | 40.00 | 60.00 | 190.00 | 110.00 | 2 |
| 3 | 190.00 | 110.00 | 190.20 | 110.10 | 6 |
| 4 | 190.20 | 110.10 | 265.00 | 135.00 | 1 |
| 5 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 6 | 305.00 | 135.00 | 365.00 | 115.00 | 1 |
| 7 | 165.00 | 60.00 | 190.00 | 110.00 | 6 |
| 8 | 165.20 | 60.00 | 190.20 | 110.10 | 1 |
| 9 | 40.00 | 60.00 | 145.00 | 60.00 | 3 |
| 10 | 145.00 | 60.00 | 165.00 | 60.00 | 4 |
| 11 | 165.00 | 60.00 | 165.20 | 60.00 | 5 |
| 12 | 145.00 | 60.00 | 155.00 | 50.00 | 3 |
| 13 | 165.20 | 60.00 | 195.20 | .00 | 5 |
| 14 | 165.00 | 60.00 | 170.00 | 50.00 | 4 |
| 15 | 155.00 | 50.00 | 170.00 | 50.00 | 3 |
| 16 | 170.00 | 50.00 | 195.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .360 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

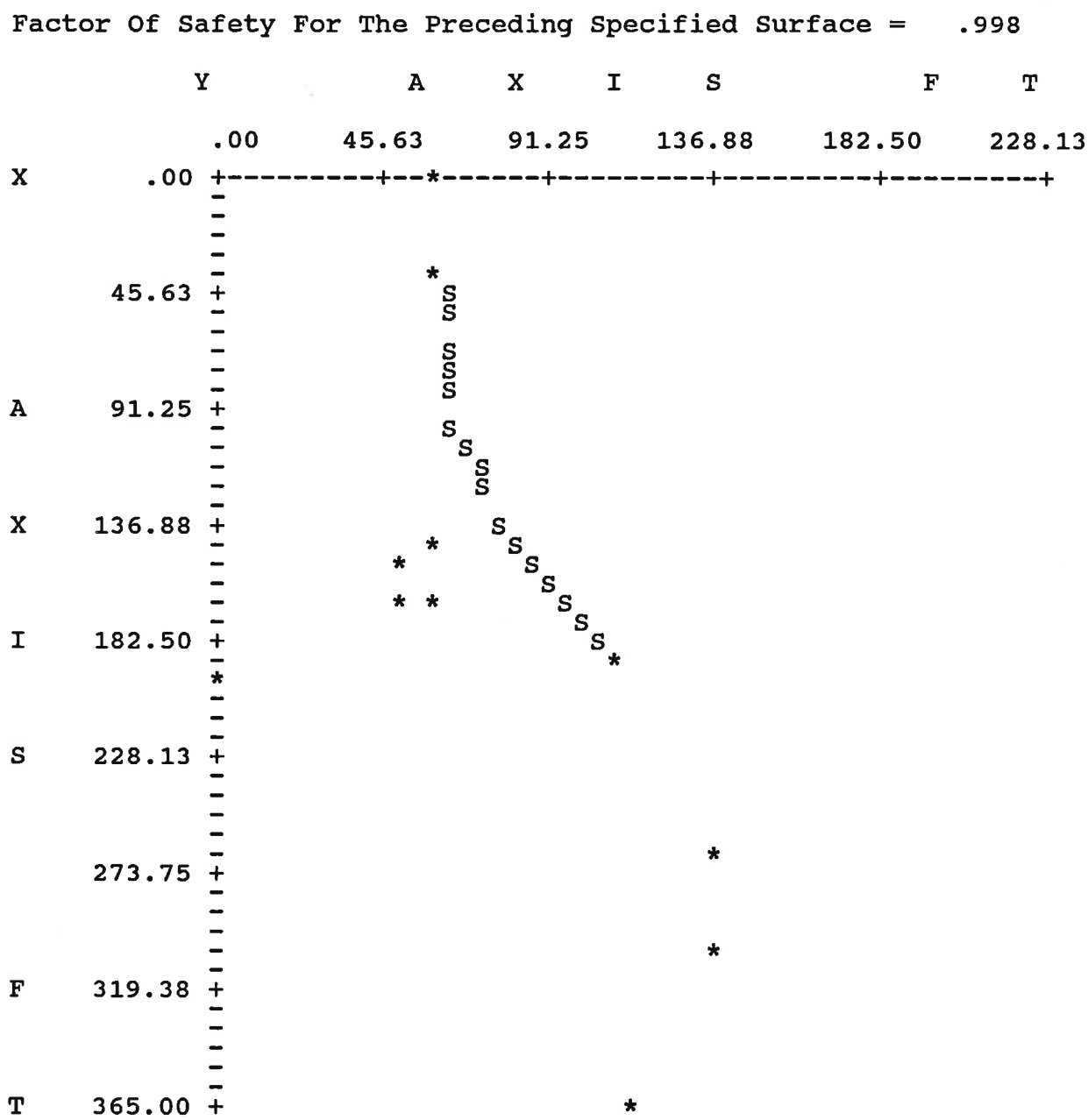
Cavitation Pressure = .0 psf

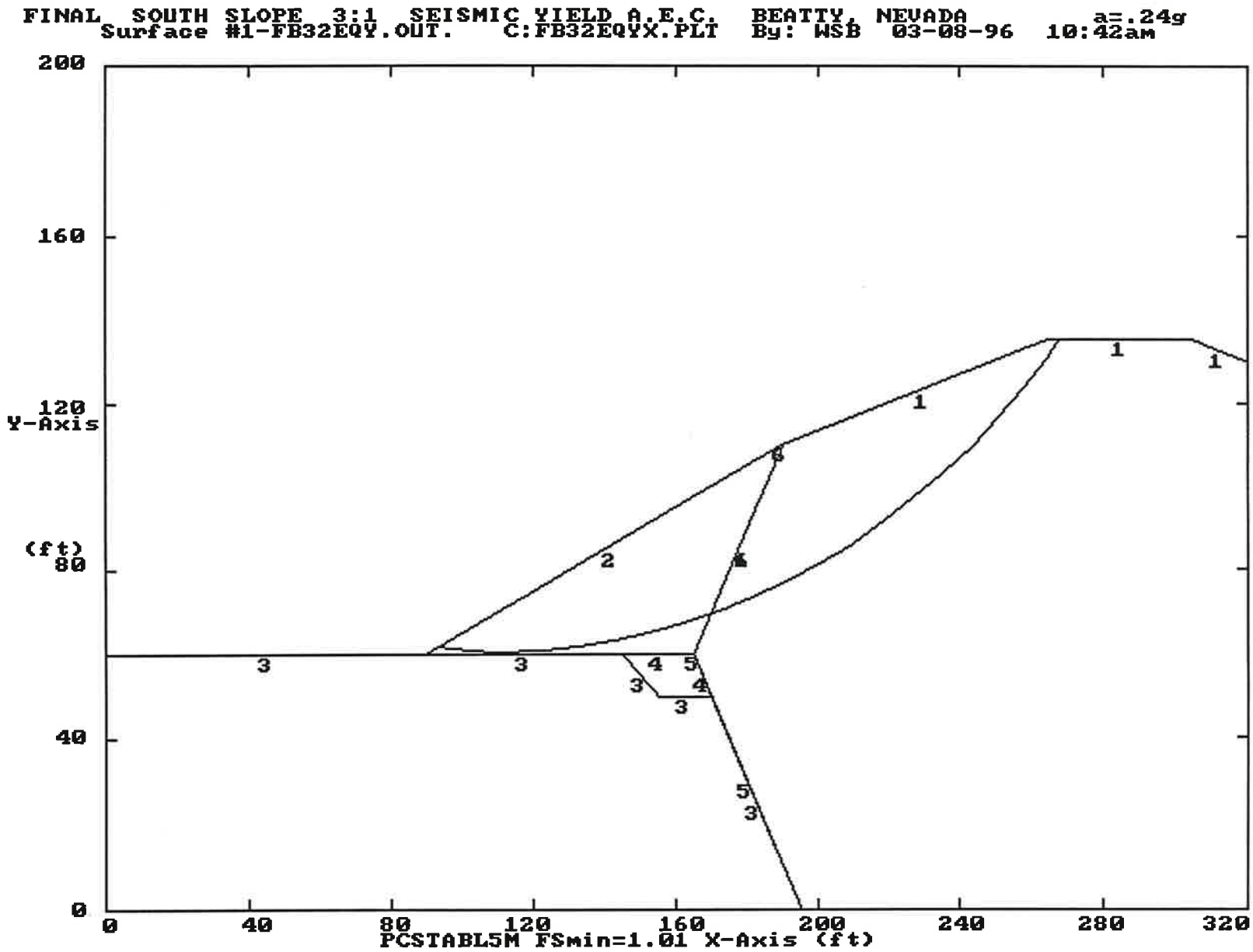
1

Trial Failure Surface Specified By 17 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 45.79 | 61.93 |
| 2 | 55.79 | 61.85 |
| 3 | 65.78 | 62.22 |
| 4 | 75.75 | 63.05 |
| 5 | 85.67 | 64.31 |
| 6 | 95.52 | 66.02 |
| 7 | 105.29 | 68.17 |
| 8 | 114.95 | 70.76 |
| 9 | 124.48 | 73.78 |
| 10 | 133.87 | 77.23 |
| 11 | 143.09 | 81.09 |
| 12 | 152.13 | 85.36 |
| 13 | 160.97 | 90.03 |
| 14 | 169.60 | 95.10 |
| 15 | 177.98 | 100.55 |
| 16 | 186.11 | 106.37 |
| 17 | 191.35 | 110.48 |

1 Factor Of Safety For The Preceding Specified Surface = .998




FINAL SOUTH SLOPE 3:1 SEISMIC YIELD A.E.C. BEATTY, NEVADA a=.24g
Surface #1-FB32EQY.OUT. C:FB32EQYX.PLT By: WSB 03-08-96 10:42am
Y-Axis (ft)
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
PCSTABL5M FSmin=1.01 X-Axis (ft)

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 10:42am |
| Run By: | WSB |
| Input Data Filename: | C:FB32EQYX |
| Output Filename: | C:FB32EQYX.OUT |
| Plotted Output Filename: | C:FB32EQYX.PLT |

PROBLEM DESCRIPTION FINAL SOUTH SLOPE 3:1 SEISMIC YIELD
A.E.C. BEATTY, NEVADA a=.24g

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 90.00 | 60.00 | 3 |
| 2 | 90.00 | 60.00 | 190.00 | 110.00 | 2 |
| 3 | 190.00 | 110.00 | 190.20 | 110.10 | 6 |
| 4 | 190.20 | 110.10 | 265.00 | 135.00 | 1 |
| 5 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 6 | 305.00 | 135.00 | 320.00 | 130.00 | 1 |
| 7 | 165.00 | 60.00 | 190.00 | 110.00 | 6 |
| 8 | 165.20 | 60.00 | 190.20 | 110.10 | 1 |
| 9 | 90.00 | 60.00 | 145.00 | 60.00 | 3 |
| 10 | 145.00 | 60.00 | 165.00 | 60.00 | 4 |
| 11 | 165.00 | 60.00 | 165.20 | 60.00 | 5 |
| 12 | 145.00 | 60.00 | 155.00 | 50.00 | 3 |
| 13 | 165.20 | 60.00 | 195.20 | .00 | 5 |
| 14 | 165.00 | 60.00 | 170.00 | 50.00 | 4 |
| 15 | 155.00 | 50.00 | 170.00 | 50.00 | 3 |
| 16 | 170.00 | 50.00 | 195.00 | .00 | 3 |

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .240 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

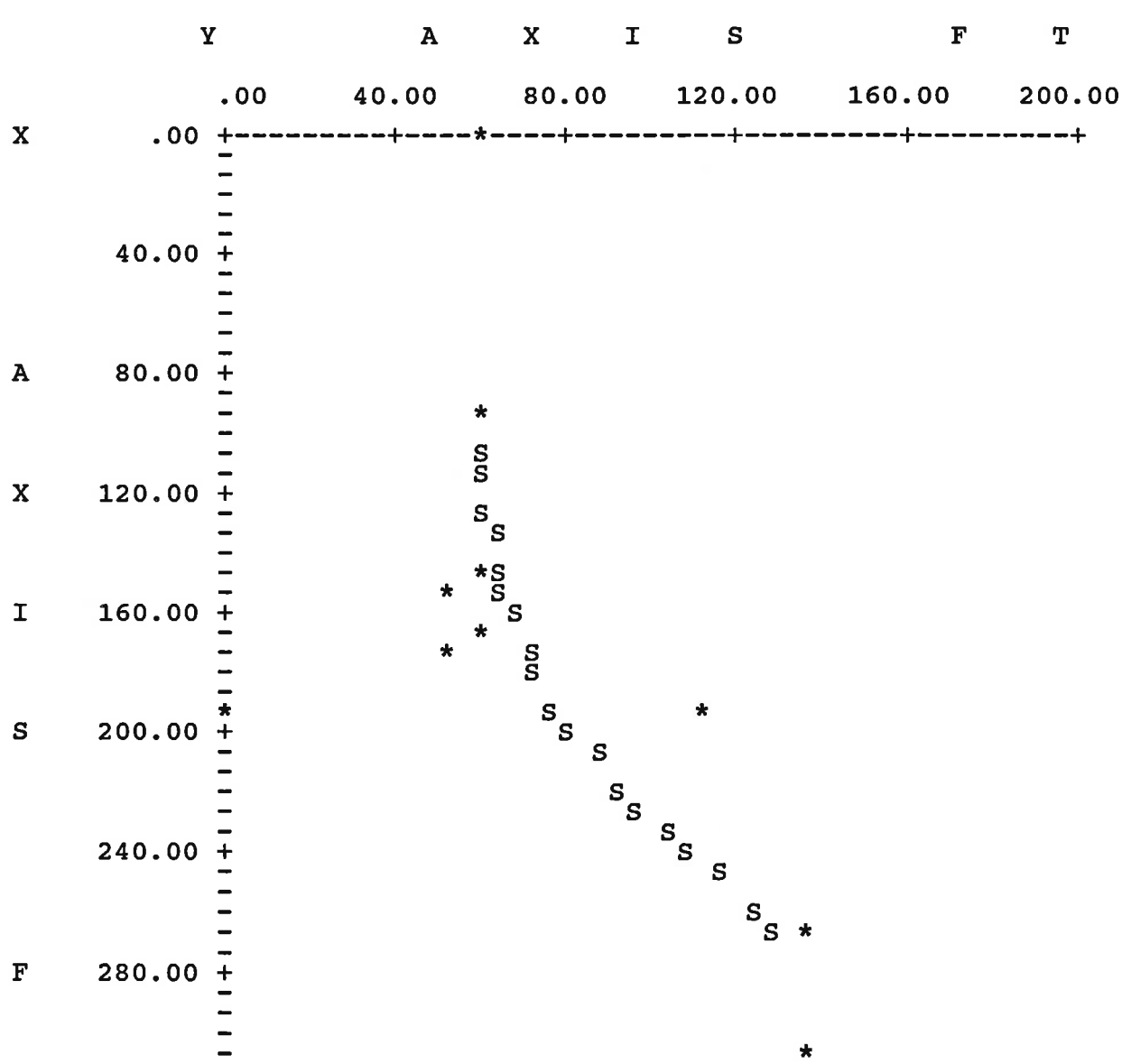
Cavitation Pressure = .0 psf

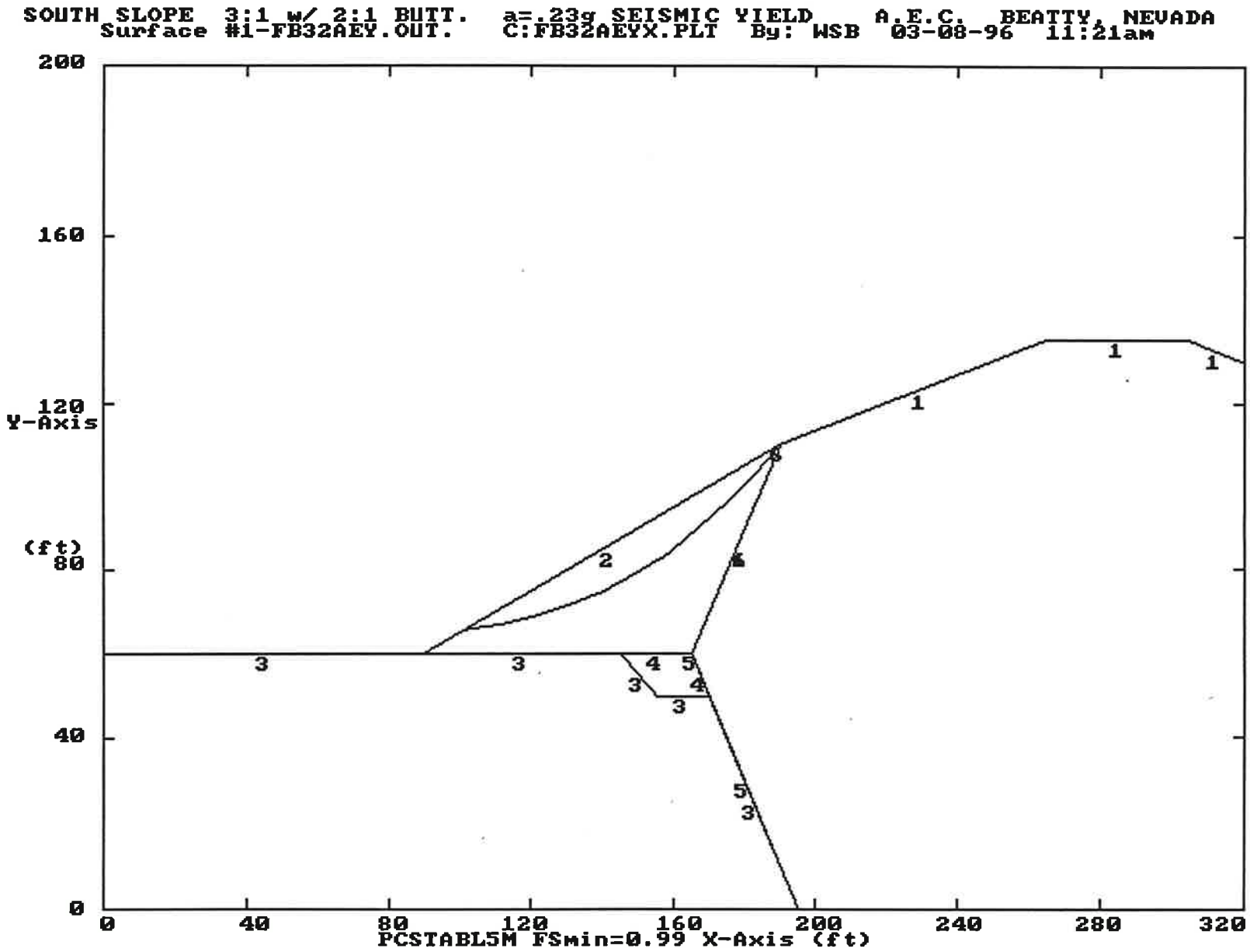
Trial Failure Surface Specified By 21 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 93.68 | 61.84 |
| 2 | 103.66 | 61.16 |
| 3 | 113.66 | 60.98 |
| 4 | 123.65 | 61.30 |
| 5 | 133.62 | 62.12 |
| 6 | 143.54 | 63.43 |
| 7 | 153.37 | 65.23 |
| 8 | 163.10 | 67.52 |
| 9 | 172.71 | 70.30 |
| 10 | 182.17 | 73.55 |
| 11 | 191.45 | 77.27 |
| 12 | 200.54 | 81.45 |
| 13 | 209.40 | 86.07 |
| 14 | 218.03 | 91.13 |
| 15 | 226.39 | 96.61 |
| 16 | 234.47 | 102.51 |
| 17 | 242.24 | 108.80 |
| 18 | 249.69 | 115.47 |
| 19 | 256.80 | 122.50 |
| 20 | 263.55 | 129.88 |
| 21 | 267.79 | 135.00 |

Factor Of Safety For The Preceding Specified Surface = 1.008

1




SOUTH SLOPE 3:1 w/ 2:1 BUTT. a=.23g SEISMIC YIELD A.E.C. BEATTY, NEVADA
Surface #1-FB32AEY.OUT. C:FB32AEYX.PLT By: WSB 03-08-96 11:21am
Y-Axis (ft)
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
PCSTABL5M FSmin=0.99 X-Axis (ft)

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

| | |
|---|---|
| Run Date: | 03-08-96 |
| Time of Run: | 11:21am |
| Run By: | WSB |
| Input Data Filename: | C:FB32AEYX |
| Output Filename: | C:FB32AEYX.OUT |
| Plotted Output Filename: | C:FB32AEYX.PLT |

PROBLEM DESCRIPTION SOUTH SLOPE 3:1 w/ 2:1 BUTT. a=.23g
SEISMIC YIELD A.E.C. BEATTY, NEVADA

BOUNDARY COORDINATES

6 Top Boundaries
16 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 60.00 | 90.00 | 60.00 | 3 |
| 2 | 90.00 | 60.00 | 190.00 | 110.00 | 2 |
| 3 | 190.00 | 110.00 | 190.20 | 110.10 | 6 |
| 4 | 190.20 | 110.10 | 265.00 | 135.00 | 1 |
| 5 | 265.00 | 135.00 | 305.00 | 135.00 | 1 |
| 6 | 305.00 | 135.00 | 320.00 | 130.00 | 1 |
| 7 | 165.00 | 60.00 | 190.00 | 110.00 | 6 |
| 8 | 165.20 | 60.00 | 190.20 | 110.10 | 1 |
| 9 | 90.00 | 60.00 | 145.00 | 60.00 | 3 |
| 10 | 145.00 | 60.00 | 165.00 | 60.00 | 4 |
| 11 | 165.00 | 60.00 | 165.20 | 60.00 | 5 |
| 12 | 145.00 | 60.00 | 155.00 | 50.00 | 3 |
| 13 | 165.20 | 60.00 | 195.20 | .00 | 5 |
| 14 | 165.00 | 60.00 | 170.00 | 50.00 | 4 |
| 15 | 155.00 | 50.00 | 170.00 | 50.00 | 3 |
| 16 | 170.00 | 50.00 | 195.00 | .00 | 3 |

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 120.0 | 120.0 | 50.0 | 35.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 4 | 120.0 | 120.0 | 1000.0 | 35.0 | .00 | .0 | 0 |
| 5 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |
| 6 | 65.0 | 65.0 | .0 | 14.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .230 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

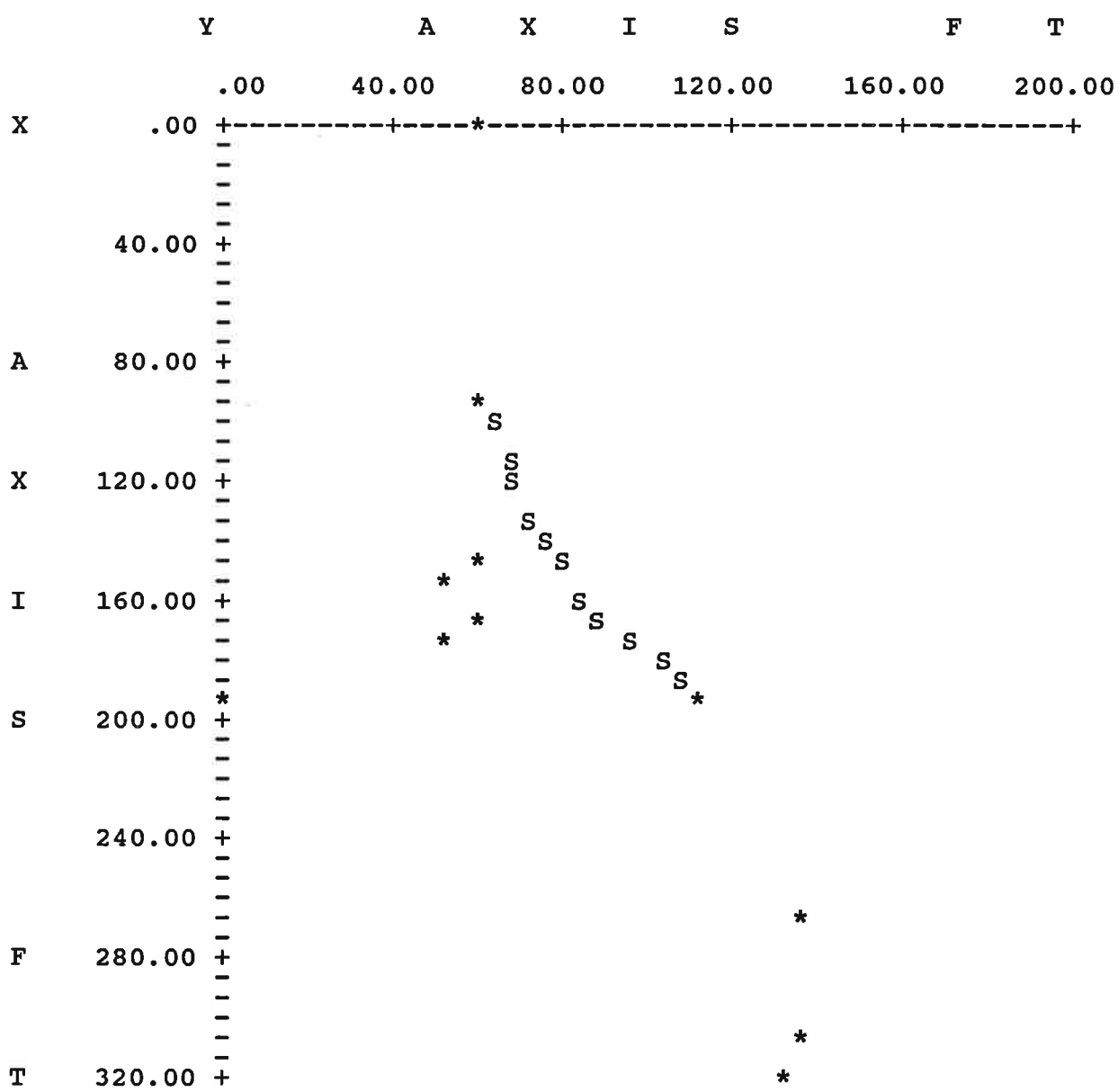
1

Trial Failure Surface Specified By 12 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 101.58 | 65.79 |
| 2 | 111.50 | 67.03 |
| 3 | 121.30 | 69.02 |
| 4 | 130.93 | 71.73 |
| 5 | 140.32 | 75.16 |
| 6 | 149.43 | 79.28 |
| 7 | 158.21 | 84.08 |
| 8 | 166.60 | 89.52 |
| 9 | 174.56 | 95.57 |
| 10 | 182.04 | 102.20 |
| 11 | 189.01 | 109.38 |
| 12 | 189.19 | 109.60 |

Factor Of Safety For The Preceding Specified Surface = .994

1



# ENVIRONMENTAL SOLUTIONS, INC.

By _____ Date _____ Subject _____ Sheet No. ____ of ____

Chkd. By ____ Date _____ _____ Proj. No. _____

BEATTY

SURFICIAL STABILITY



Recyclable and made from recycled paper.

# ENVIRONMENTAL SOLUTIONS, INC.

By WB Date 2/13/96 Subject Surficial Slope Stability Sheet No. 1 of 13
Chkd. By JM Date 2/18/96 Proj. No. 95-284

SURFICIAL STABILITY

TABLE OF CONTENTS


| | |
|---|---|
| OBJECTIVE | 2 |
| METHOD | 2 |
| REFERENCES | 4 |
| ASSUMPTIONS | 4 |
| STABILITY EVALUATION | 5 |
| CONCLUSIONS | 9 |

ATTACHMENT A - SURFICIAL STABILITY SPREADSHEET CALCULATIONS

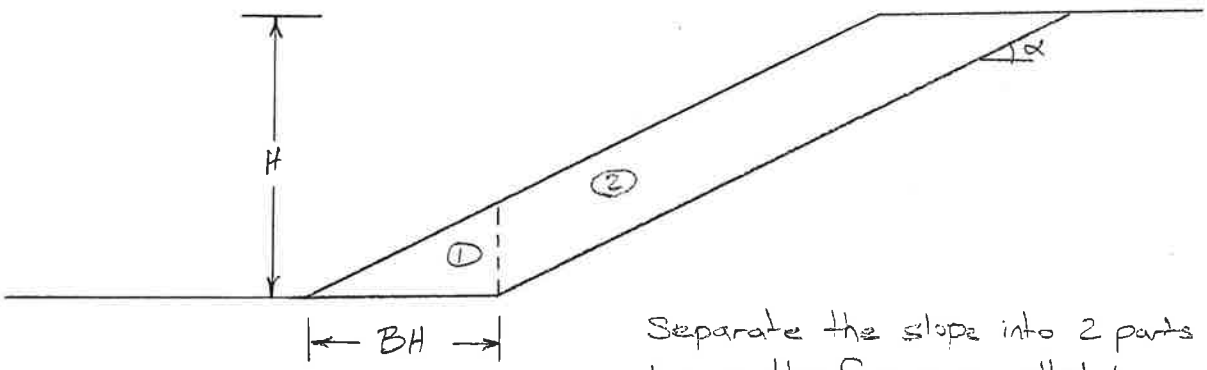
# ENVIRONMENTAL SOLUTIONS, INC.

By NB Date 2/13/96 Subject Surficial Slope Stability Sheet No. 2 of 13

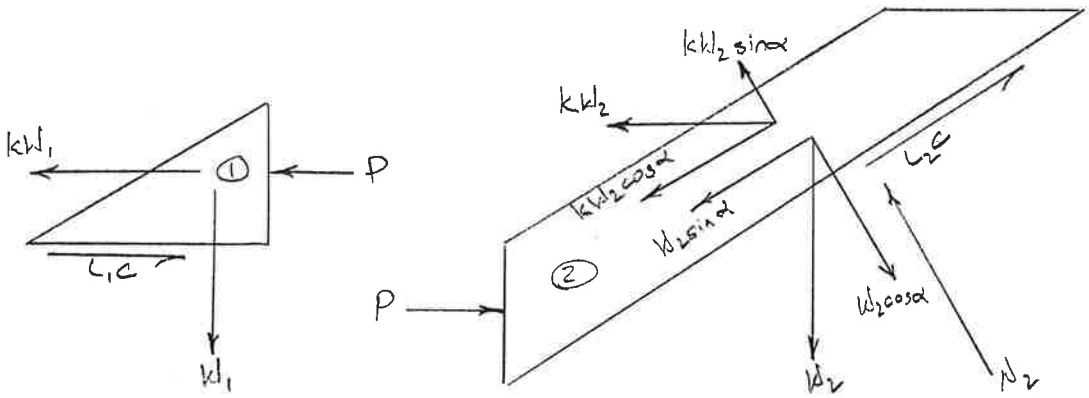
Chkd. By JM Date 2/18/96 Proj. No. 95-234

OBJECTIVE: Evaluate the surficial stability of the cover components, which include surficial soils, liner interfaces and select waste fill.

METHOD: Use the wedge method as described by Huang (Reference 1). The wedge method model that was used is shown below.




Separate the slope into 2 parts & sum the forces parallel & normal to the slope.



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# ENVIRONMENTAL SOLUTIONS, INC.

By WB Date 7/13/96 Subject Surficial Slope Stability Sheet No. 3 of 13

Chkd. By JM Date 7/18/96 Proj. No. 95-284

By summing the forces & substituting for P & $N_2$ the resulting equation can be obtained:

$$a_1 F^2 + a_2 F + a_3 = 0 \qquad \text{(Reference 1)}$$

Where

$$a_1 = a_4 \sin\alpha + k(a_4 + a_5)\cos\alpha$$

$$a_2 = -\left[\frac{c}{\gamma H}\left(B\cos\alpha + \frac{1}{\sin\alpha}\right) + (\cos\alpha - k\sin\alpha)(a_4 + a_5)\tan\phi\right]$$

$$a_3 = -B\sin\alpha\tan\phi\left[\frac{c}{\gamma H} + \left(\frac{a_5}{B}\tan\phi\right)\right]$$

$$a_4 = \frac{1}{2}\left[\frac{1}{\tan\alpha} - (1 - B\tan\alpha)^2/\tan\alpha\right]$$

$$a_5 = \frac{1}{2}B^2\tan\alpha$$



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## REFERENCES

1. Huang, Y. H., "Stability Analysis of Earth Slopes" Van Nostrand Reinhold Company, New York, 1983.

2. Franklin & Chang, "Permanent Displacements of Earth Embankments by Newmark Sliding Block Analysis", Earthquake Resistance of Earth & Rockfill Dams, Report 5, U.S. Army Engineer Waterways Experiment Station, November 1977.

3. Donovan, N., "A Practioners View of Site Effects on Strong Ground Motion", Proceeding of Conference XXII on Site Specific Effects of Soil & Rock on Ground Motions & the Implications for Earthquake Resistant Design", July 25-27, 1983, USGS, Santa Fe, New Mexico, July 1983.

## ASSUMPTIONS

1. Hydrostatic forces are not allowed to build up in either the top soil or existing cover soil.

2. The proposed cover strength should be tested & compared with the values used in this analysis.

3. Interface strengths also need to be confirmed prior to construction.



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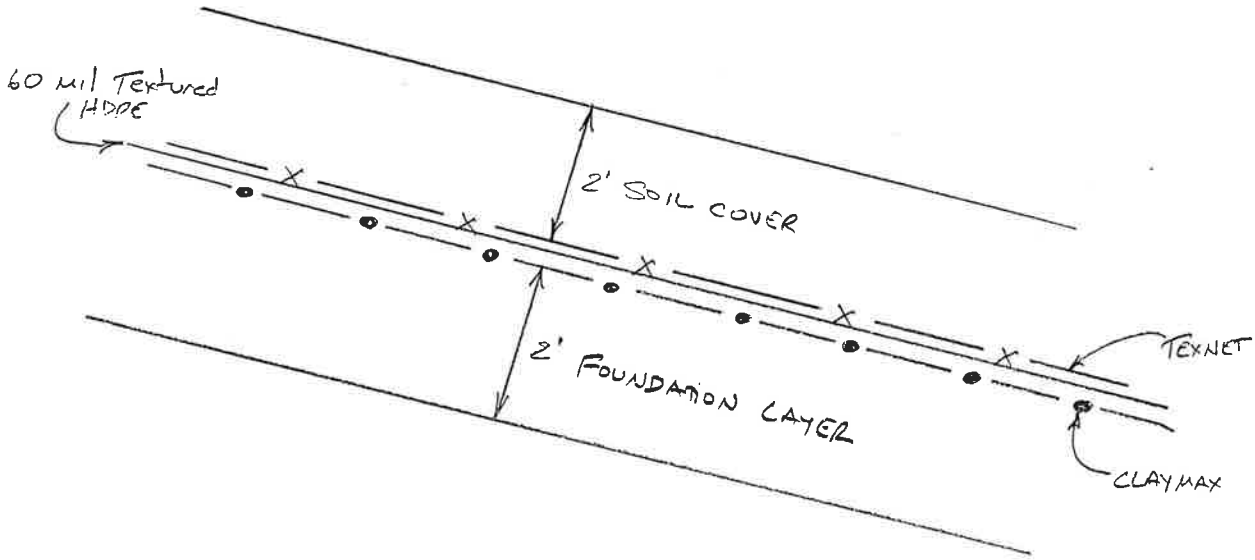
# ENVIRONMENTAL SOLUTIONS, INC.

By WB Date 2/13/96 Subject Surficial Slope Stability Sheet No. 5 of 13
Chkd. By JA Date 2/18/96 Proj. No. 95-254

STABILITY EVALUATION

The selected liner configuration evaluated is as shown below:




The following materials & interfaces were evaluated:

1.) Soil cover

2.) Soil on Texnet

3.) Texnet on 60 mil Textured HDPE

4.) 60 mil Textured HDPE on GCL

5.) GCL (internal)

6.) GCL on FOUNDATION LAYER



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7. FOUNDATION LAYER

Table 1 presents a summary of the material properties in the static & pseudostatic analyses. These materials & interfaces will be evaluated on a 2.5:1 slope & a 3:1 slope. Since one alternative is to have the buttress at a 2:1 slope, the soil cover will be also evaluated for a 2:1 slope.

For the 2.5:1 & 3:1 slopes, the slope length used in the calculations was 250 feet long.

Table 2 presents the summary of the static factors of safety for each of the 7 cases for both a 2.5:1 & 3:1 slope and the 2:1 slope buttress case. Table 2 also presents the strength values used in the calculations. The strength values were evaluated in a previous calculation in this Appendix.

The data & calculation spreadsheet are presented in Attachment A.

Table 3 presents the summary of seismic factors of safety using a maximum horizontal acceleration of 0.42g. All strength values used & all three slope steepnesses are also presented on Table 3. The data & calculation spreadsheet are presented in Attachment A.



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TABLE 1. SUMMARY OF MATERIAL PROPERTIES

| | | STATIC | | PSEUDOSTATIC | |
|---|---|---|---|---|---|
| NUMBER | MATERIAL OR INTERFACE | COHESION (psf) | PHI (deg) | COHESION (psf) | PHI (deg) |
| 1 | SOIL COVER LAYER | 0 | 35 | 0 | 35 |
| 2 | SOIL COVER ON TEXNET | 25 | 21 | 144 | 28 |
| 3 | TEXNET ON 60 MIL TEXTURED HDPE | 280 | 7 | 400 | 13 |
| 4 | 60 MIL HDPE ON GCL | 53 | 17 | 53 | 20 |
| 5 | GCL (INTERNAL) | 500 | 0 | 500 | 0 |
| 6 | GCL ON SELECT Foundation Layer | 50 | 25 | 50 | 25 |
| 7 | Foundation Layer | 300 | 27 | 300 | 27 |

TABLE 2. SUMMARY STATIC FACTORS OF SAFETY

| | COHESION (psf) | PHI (deg) | SLOPE STEEPNESS 2.5:1 | SLOPE STEEPNESS 3:1 (1) |
|---|---|---|---|---|
| SOIL COVER LAYER | 0 | 35 | 1.77 | 2.13 |
| SOIL COVER ON TEXNET | 25 | 21 | 1.26 | 1.51 |
| TEXNET ON 60 MIL TEXTURED HDPE | 280 | 7 | 3.55 | 4.21 |
| 60 MIL HDPE ON GCL | 53 | 17 | 1.39 | 1.66 |
| GCL (INTERNAL) | 500 | 0 | 5.79 | 6.85 |
| GCL ON Foundation Layer | 50 | 25 | 1.76 | 2.10 |
| Foundation Layer | 300 | 27 | 3.09 | 3.70 |

| | COHESION (psf) | PHI (deg) | SLOPE STEEPNESS 2:1 BUTTRESS |
|---|---|---|---|
| SOIL COVER LAYER | 0 | 35 | 1.43 |

(1) Selected Configuration.

TABLE 3. SUMMARY SEISMIC FACTOR OF SAFETY[1] (a = .42g)

| | COHESION (psf) | PHI (deg) | SLOPE STEEPNESS 2.5:1 | SLOPE STEEPNESS 3:1[2] |
|---|---|---|---|---|
| SOIL COVER LAYER | 0 | 35 | 0.71 | 0.80 |
| SOIL COVER ON TEXNET | 144 | 28 | 1.35 | 1.47 |
| TEXNET ON 60 MIL TEXTURED HDPE | 400 | 13 | 2.48 | 2.67 |
| 60 MIL HDPE ON GCL | 53 | 20 | 0.67 | 0.73 |
| GCL (INTERNAL) | 500 | 0 | 2.81 | 3.01 |
| GCL ON Foundation Layer | 50 | 25 | 0.75 | 0.83 |
| Foundation Layer | 300 | 27 | 1.38 | 1.51 |

| | COHESION (psf) | PHI (deg) | SLOPE STEEPNESS 2:1 BUTTRESS |
|---|---|---|---|
| SOIL COVER LAYER | 0 | 35 | 0.29 |

(1) Seismic factor of safety less than one indicates displacement analysis is necessary to determine magnitude of deformation, (see Sheets 10, 11 and 12).

(2) Selected Configuration.

Because some of the pseudo static factors of safety are less than 1.0 a permanent displacement analysis should be performed. This analysis is similar to the analysis described in the Deep Seated Slope Stability Section.

Table 4 presents all the yield accelerations calculated for all conditions evaluated. Table 4 also presents the material or interface strength used. The spreadsheet for the yield acceleration calculation are presented in Attachment A.

Table 5 presents the summary of permanent displacements in inches due to a 0.42g ground acceleration for all slope steepnesses & materials evaluated. The spreadsheets for the permanent displacements are presented in Attachment A.

## CONCLUSIONS

The static surficial Factors of Safety for the 3:1 slopes are all above the 1.5 acceptable Factor of Safety. The static surficial Factors of Safety for the 2.5:1 slopes have 2 interfaces that are below the 1.5 acceptable Factor of Safety. The 2:1 buttress slope has a static surficial Factor of safety of 1.43, which is just below the generally accepted 1.5 value. However, the buttress soil strength was assumed to have no cohesion. When the cover soil material is tested there is a potential that the material will have a small cohesion value. Any cohesion should be sufficient to




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TABLE 4. SUMMARY SEISMIC YIELD VALUES

| | COHESION (psf) | PHI (deg) | SLOPE STEEPNESS 2.5:1 | SLOPE STEEPNESS 3:1(1) |
|---|---|---|---|---|
| SOIL COVER LAYER | 0 | 35 | 0.24g | 0.30g |
| SOIL COVER ON TEXNET | 144 | 28 | >0.42g | >0.42g |
| TEXNET ON 60 MIL TEXTURED HDPE | 400 | 13 | >0.42g | >0.42g |
| 60 MIL HDPE ON GCL | 53 | 20 | 0.19g | 0.25g |
| GCL (INTERNAL) | 500 | 0 | >0.42g | >0.42g |
| GCL ON Foundation Layer | 50 | 25 | 0.25g | 0.31g |
| Foundation Layer | 300 | 27 | >0.42g | >0.42g |

| | COHESION (psf) | PHI (deg) | SLOPE STEEPNESS 2:1 BUTTRESS(1) |
|---|---|---|---|
| SOIL COVER LAYER | 0 | 35 | 0.15g |

(1) Selected Configuration.

TABLE 5. SUMMARY SEISMIC PERMANENT DEFORMATIONS (inches)

| | COHESION (psf) | PHI (deg) | SLOPE STEEPNESS 2.5:1 | SLOPE STEEPNESS 3:1 (1) |
|---|---|---|---|---|
| SOIL COVER LAYER | 0 | 35 | 2.2 | 0.7 |
| SOIL COVER ON TEXNET | 144 | 28 | --- | --- |
| TEXNET ON 60 MIL TEXTURED HDPE | 400 | 13 | --- | --- |
| 60 MIL HDPE ON GCL | 53 | 20 | 4.5 | 2.0 |
| GCL (INTERNAL) | 500 | 0 | --- | --- |
| GCL ON Foundation Layer | 50 | 25 | 2.0 | 0.6 |
| Foundation Layer | 300 | 27 | --- | --- |

| | COHESION (psf) | PHI (deg) | SLOPE STEEPNESS 2:1 BUTTRESS (1) |
|---|---|---|---|
| SOIL COVER LAYER | 0 | 35 | 10.5 |

-- = Seismic factor of safety greater than one.

(1) Selected Configuration

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increase the Factor of Safety to an acceptable value.

The seismic permanent deformations are estimated to be within the tolerable limit of 12 inches for all slopes & interfaces evaluated. The largest deformation estimated is 10.5 inches for the 2:1 buttress slope surficial soils. Again, the strength of the buttress soils assume no cohesion. Any cohesion, even though small, will reduce the deformations estimated for the 2:1 buttress slope.



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ATTACHMENT A

SURFICIAL STABILITY SPREADSHEET CALCULATIONS



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SURFICIAL STABILITY EVALUATION

| NUMBER | MATERIAL OR INTERFACE | STATIC COHESION (psf) | STATIC PHI (deg) | PSEUDOSTATIC COHESION (psf) | PSEUDOSTATIC PHI (deg) |
|---|---|---|---|---|---|
| 1 | SOIL COVER LAYER | 0 | 35 | 0 | 35 |
| 2 | SOIL COVER ON TEXNET | 25 | 21 | 144 | 28 |
| 3 | TEXNET ON 60 MIL TEXTURED HDPE | 280 | 7 | 400 | 13 |
| 4 | 60 MIL HDPE ON GCL | 53 | 17 | 53 | 20 |
| 5 | GCL (INTERNAL) | 500 | 0 | 500 | 0 |
| 6 | GCL ON Foundation Layer | 50 | 25 | 50 | 25 |
| 7 | Foundation Layer | 300 | 27 | 300 | 27 |

## SURFICIAL STABILITY (Huang Equation) STATIC CASE 2.5:1 SLOPE

| STATIC CASE | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|---|---|
| with wedge | | Soil | Soil/ Texnet | Texnet/ Text. HDPE | Text. HDPE/ GCL | GCL | GCL/ Foundation | Foundation |
| COVER SOIL | | | | | | | | |
| unit weight (pcf) | | 120 | 120 | 120 | 120 | 120 | 120 | 120 |
| cohesion (psf) | | 0 | 25 | 280 | 53 | 500 | 50 | 300 |
| phi (deg.) | | 35 | 21 | 7 | 17 | 0 | 25 | 27 |
| | | | | | | | | |
| THICKNESS | | | | | | | | |
| thickness (ft) | | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 4.0 |
| depth (ft) | | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 | 4.3 |
| | | | | | | | | |
| | | | | | | | | |
| SLOPE STEEPNESS | | 2.5:1 | 2.5:1 | 2.5:1 | 2.5:1 | 2.5:1 | 2.5:1 | 2.5:1 |
| degrees | | 21.8 | 21.8 | 21.8 | 21.8 | 21.8 | 21.8 | 21.8 |
| | | | | | | | | |
| | | | | | | | | |
| ACCELERATION | | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| | | | | | | | | |
| | | | | | | | | |
| | SLOPE | | | | | | | |
| | LENGTH | FACTOR OF SAFETY | | | | | | |
| | | | | | | | | |
| Huang Equation | 250 | 1.77 | 1.26 | 3.55 | 1.39 | 5.79 | 1.76 | 3.09 |
| | | | | | | | | |

## SURFICIAL STABILITY (Huang Equation) STATIC CASE 3:1 SLOPE

| STATIC CASE | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|---|---|
| with wedge | | | | | | | | |
| COVER SOIL | | | | | | | | |
| unit weight (pcf) | | 120 | 120 | 120 | 120 | 120 | 120 | 120 |
| cohesion (psf) | | 0 | 25 | 280 | 53 | 500 | 50 | 300 |
| phi (deg.) | | 35 | 21 | 7 | 17 | 0 | 25 | 27 |
| | | | | | | | | |
| THICKNESS | | | | | | | | |
| thickness (ft) | | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 4.0 |
| depth (ft) | | 2.1 | 2.1 | 2.1 | 2.1 | 2.1 | 2.1 | 4.2 |
| | | | | | | | | |
| | | | | | | | | |
| SLOPE STEEPNESS | | 3:1 | 3:1 | 3:1 | 3:1 | 3:1 | 3:1 | 3:1 |
| degrees | | 18.4 | 18.4 | 18.4 | 18.4 | 18.4 | 18.4 | 18.4 |
| | | | | | | | | |
| | | | | | | | | |
| ACCELERATION | | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| | | | | | | | | |
| | | | | | | | | |
| | **SLOPE** | | | | | | | |
| | **LENGTH** | **FACTOR OF SAFETY** | | | | | | |
| | | | | | | | | |
| Huang Equation | **250** | **2.13** | **1.51** | **4.21** | **1.66** | **6.85** | **2.10** | **3.70** |
| | | | | | | | | |

## SURFICIAL STABILITY (Huang Equation) SEISMIC CASE 2.5:1 SLOPE

| SEISMIC CASE | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|---|---|
| with wedge | | | | | | | | |
| COVER SOIL | | | | | | | | |
| unit weight (pcf) | | 120 | 120 | 120 | 120 | 120 | 120 | 120 |
| cohesion (psf) | | 0 | 144 | 400 | 53 | 500 | 50 | 300 |
| phi (deg.) | | 35 | 28 | 13 | 20 | 0 | 25 | 27 |
| | | | | | | | | |
| THICKNESS | | | | | | | | |
| thickness (ft) | | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 4.0 |
| depth (ft) | | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 | 4.3 |
| | | | | | | | | |
| | | | | | | | | |
| SLOPE STEEPNESS | | 2.5:1 | 2.5:1 | 2.5:1 | 2.5:1 | 2.5:1 | 2.5:1 | 2.5:1 |
| degrees | | 21.8 | 21.8 | 21.8 | 21.8 | 21.8 | 21.8 | 21.8 |
| | | | | | | | | |
| | | | | | | | | |
| ACCELERATION | | 0.42 | 0.42 | 0.42 | 0.42 | 0.42 | 0.42 | 0.42 |
| | | | | | | | | |
| | | | | | | | | |
| | SLOPE | | | | | | | |
| | LENGTH | FACTOR OF SAFETY | | | | | | |
| | | | | | | | | |
| Huang Equation | 250 | 0.71 | 1.35 | 2.48 | 0.67 | 2.81 | 0.75 | 1.38 |
| | | | | | | | | |

## SURFICIAL STABILITY (Huang Equation) SEISMIC CASE 3:1 SLOPE

| SEISMIC CASE | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|---|---|
| with wedge | | | | | | | | |
| COVER SOIL | | | | | | | | |
| unit weight (pcf) | | 120 | 120 | 120 | 120 | 120 | 120 | 120 |
| cohesion (psf) | | 0 | 144 | 400 | 53 | 500 | 50 | 300 |
| phi (deg.) | | 35 | 28 | 13 | 20 | 0 | 25 | 27 |
| | | | | | | | | |
| THICKNESS | | | | | | | | |
| thickness (ft) | | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 4.0 |
| depth (ft) | | 2.1 | 2.1 | 2.1 | 2.1 | 2.1 | 2.1 | 4.2 |
| | | | | | | | | |
| | | | | | | | | |
| SLOPE STEEPNESS | | 3:1 | 3:1 | 3:1 | 3:1 | 3:1 | 3:1 | 3:1 |
| degrees | | 18.4 | 18.4 | 18.4 | 18.4 | 18.4 | 18.4 | 18.4 |
| | | | | | | | | |
| | | | | | | | | |
| ACCELERATION | | 0.42 | 0.42 | 0.42 | 0.42 | 0.42 | 0.42 | 0.42 |
| | | | | | | | | |
| | | | | | | | | |
| | SLOPE | | | | | | | |
| | LENGTH | FACTOR OF SAFETY | | | | | | |
| | | | | | | | | |
| Huang Equation | 250 | 0.80 | 1.47 | 2.67 | 0.73 | 3.01 | 0.83 | 1.51 |
| | | | | | | | | |

## SURFICIAL STABILITY (Huang Equation) SEISMIC YIELD 2.5:1 SLOPE

| SEISMIC CASE | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|---|---|
| with wedge | | | | | | | | |
| COVER SOIL | | | | | | | | |
| unit weight (pcf) | | 120 | 120 | 120 | 120 | 120 | 120 | 120 |
| cohesion (psf) | | 0 | 144 | 400 | 53 | 500 | 50 | 300 |
| phi (deg.) | | 35 | 28 | 13 | 20 | 0 | 25 | 27 |
| | | | | | | | | |
| THICKNESS | | | | | | | | |
| thickness (ft) | | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 |
| depth (ft) | | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 |
| | | | | | | | | |
| | | | | | | | | |
| SLOPE STEEPNESS | | 2.5:1 | 2.5:1 | 2.5:1 | 2.5:1 | 2.5:1 | 2.5:1 | 2.5:1 |
| degrees | | 21.8 | 21.8 | 21.8 | 21.8 | 21.8 | 21.8 | 21.8 |
| | | | | | | | | |
| | | | | | | | | |
| | SLOPE | | | | | | | |
| | LENGTH | YIELD ACCELERATION | | | | | | |
| | | | | | | | | |
| | 250 | 0.24 | 0.42 | 0.42 | 0.19 | 0.42 | 0.25 | 0.42 |
| | | | | | | | | |
| | SLOPE | | | | | | | |
| | LENGTH | FACTOR OF SAFETY | | | | | | |
| | | | | | | | | |
| Huang Equation | 250 | 1.00 | >1.00 | >1.00 | 1.00 | >1.00 | 1.00 | >1.00 |
| | | | | | | | | |
| | | | | | | | | |

## SURFICIAL STABILITY (Huang Equation) SEISMIC YIELD 3:1 SLOPE

| SEISMIC CASE | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|---|---|
| with wedge | | | | | | | | |
| COVER SOIL | | | | | | | | |
| unit weight (pcf) | | 120 | 120 | 120 | 120 | 120 | 120 | 120 |
| cohesion (psf) | | 0 | 144 | 400 | 53 | 500 | 50 | 300 |
| phi (deg.) | | 35 | 28 | 13 | 20 | 0 | 25 | 27 |
| | | | | | | | | |
| THICKNESS | | | | | | | | |
| thickness (ft) | | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 |
| depth (ft) | | 2.1 | 2.1 | 2.1 | 2.1 | 2.1 | 2.1 | 2.1 |
| | | | | | | | | |
| | | | | | | | | |
| SLOPE STEEPNESS | | 3:1 | 3:1 | 3:1 | 3:1 | 3:1 | 3:1 | 3:1 |
| degrees | | 18.4 | 18.4 | 18.4 | 18.4 | 18.4 | 18.4 | 18.4 |
| | | | | | | | | |
| | | | | | | | | |
| | SLOPE | | | | | | | |
| | LENGTH | YIELD ACCELERATION | | | | | | |
| | | | | | | | | |
| | 250 | 0.30 | 0.42 | 0.42 | 0.25 | 0.42 | 0.31 | 0.42 |
| | | | | | | | | |
| | SLOPE | | | | | | | |
| | LENGTH | FACTOR OF SAFETY | | | | | | |
| | | | | | | | | |
| Huang Equation | 250 | 1.00 | >1.00 | >1.00 | 1.00 | >1.00 | 1.00 | >1.00 |
| | | | | | | | | |
| | | | | | | | | |

## SURFICIAL STABILITY (Huang Equation) 2:1 SLOPE

| | | STATIC | | SEISMIC | | SEISMIC | | |
|---|---|---|---|---|---|---|---|---|
| | | Soil | | Soil | | YIELD | | |
| COVER SOIL | | | | | | | | |
| unit weight (pcf) | | 120 | | 120 | | 120 | | |
| cohesion (psf) | | 0 | | 0 | | 0 | | |
| phi (deg.) | | 35 | | 35 | | 35 | | |
| | | | | | | | | |
| THICKNESS | | | | | | | | |
| thickness (ft) | | 2.0 | | 2.0 | | 2.0 | | |
| depth (ft) | | 2.2 | | 2.2 | | 2.2 | | |
| | | | | | | | | |
| | | | | | | | | |
| SLOPE STEEPNESS | | 2:1 | | 2:1 | | 2:1 | | |
| degrees | | 26.6 | | 26.6 | | 26.6 | | |
| | | | | | | | | |
| | | | | | | | | |
| ACCELERATION | | 0 | | 0.83 | | 0.15 | | |
| | | | | | | | | |
| | | | | | | | | |
| | SLOPE | | | | | | | |
| | LENGTH | FACTOR OF SAFETY | | | | | | |
| | | | | | | | | |
| Huang Equation | 100 | 1.43 | | 0.29 | | 1.00 | | |
| | | | | | | | | |

# C.2 SLOPE STABILITY INTERIM CONFIGURATION

# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 3/5/96 Subject Slope Stability During Waste Fill Placement Sheet No. 0 of 9

Chkd. By WB Date 3/5/96 Proj. No. 95-284

SLOPE STABILITY DURING WASTE

FILL PLACEMENT

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By JH Date 3/5/96 Subject Slope Stability During Waste Fill Placement Sheet No. 1 of 9
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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 3/4/96 Subject Slope Stability During Waste Fill Placement Sheet No. 2 of 9

Chkd. By WB Date 3/5/96 Proj. No. 95-284

1.0 Purpose – to evaluate waste fill stability during placement.

2.0 Assumptions

(a) Failure would occur along (or through) weakest interface in liner system. It is assumed that the face of the fill slope is perpendicular to A–A'.

(b) Design parameters are the same as those used in other calculations for the landfill (see References 1, 2, and 3), except the interface parameters (see 6.0)

3.0 Design profile. Cross-section A–A' is selected for analysis (see Figure 1 for location of A–A', and Figure 2 for design geometry).

4.0 Approach. The design profile is analyzed using computer program PCSTABL5 (Ref 2). Material A is waste fill, material B is the weakest interface in the liner system, and material C is native soils.

Block analysis was performed to search the most critical failure surface and the associated factor of safety under both static and pseudo-static conditions.



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5.0

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By JH Date 3/4/96 Subject Slope Stability During Waste Fill Placement Sheet No. 4 of 9

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6.0 Interface Strength Parameters.

| Interface | Residual Strength Parameters | | Range of Confining Pressure (psi) | Ref. |
|---|---|---|---|---|
| | Cohesion (psf) | $\phi$ (degree) | | |
| TEXNET 3002/ Textured HDPE | 805 | 9 | 20-80 | 1 |
| | 500 | 11 | 20-80 | 1 |
| Geonet/Smooth HDPE * | 0 | 8 | 23-69 | 7 |
| INTERNAL SHEAR ** STRENGTH OF CLAYMAX SHEARPRO | 550 | 5 | 0.35-142.4 | 3 |

The weakest interface is the internal face of GCL. The internal shear strength parameters are used for the slope stability calculations. The range of confining pressures under which the shear strength parameters were reported covered the expected confining pressures in the field (0-50 psi).

Notes:

* Most critical interface on side slope

** Most critical interface in bottom liner system.



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By WB Date 3/5/96 Subject Slope Stability During Waste Fill Placement Sheet No. 5 of 9

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7.0 Deformation Analysis

Estimate permanent deformation based on 50 yr seismic event (90% chance of not being exceeded = 475 year return period)

$a = 0.20g$

$k_y$ = seismic yield for slope = 0.1g

$k_y/a = \frac{.1}{.2} = 0.5$ $V/A = 40$ (Ref. 6) $V = (40)(.2) = 8$ in/sec

From Figure 2 $k_y/a = 0.5$ $U_s = 9''$

$$U_m = \frac{U_s V^2}{1800\, a} = \frac{(9)(8)^2}{1800\,(.2)} = 1.6''$$



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Ref (5) 6/9

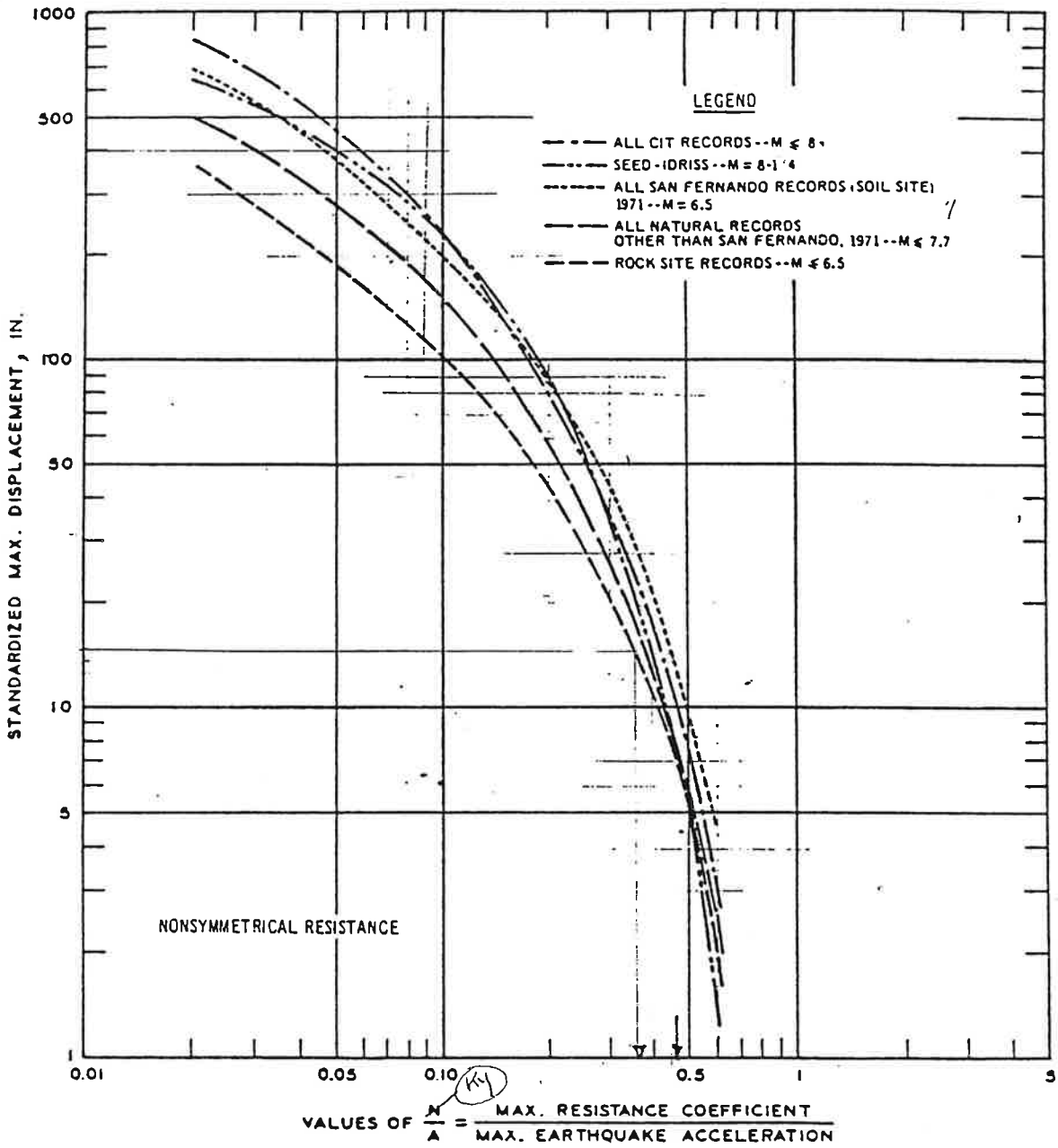


Figure 16. Upper bound envelope curves of permanent displacements for all natural and synthetic records analyzed

# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 3/4/96 Subject Slope Stability During Waste Fill Placement Sheet No. 7 of 9

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8.0 Results

1. Static factor of safety ≒ 1.45. Failure surface is assumed to go through the internal face of GCL. The static factor of safety is considered sufficient.

2. Pseudo-static factor of safety ≒ 0.75, with an design acceleration of 0.2 g.

3. Yield Acceleration ≒ 0.1 g.

4. A seismic deformation analysis is performed to evaluate deformation. It is recognized that the design acceleration corresponds to a 475-year return period which maybe conservative for temporary slope stability analysis.

A seismic deformation of 1.6" is calculated, assuming the waste fill is moving as a rigid body under seismic loads. In reality, as the seismic wave is propagating through the waste fill, it is unlikely that the entire waste fill would slide as a rigid body. The actual deformation response would be depth (or overburden) dependent (Ref. 4), and are likely to be less at the base than at the top of the waste fill.

We expect that the actual base deformation would be less than 1.6", and distress in the liner system is not expected under the design earthquake load.



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N9000'00"E
1,219.40'
2780 2770 2760 2750 2740 2730 2720
1% SLOPE (MIN.)
EL. 2717.1
EL. 2717.1
2% SLOPE
EL.
EL.
N0000'00"E
234.69'
2720 2730 2740 2750 2760 2770 2780 2785
1% SLOPE (MIN.)
1% SLOPE (MIN. SEE NOTE 4)
2716
2715
2714
A
006
A
2712
RIDGE LINE
2713
2714
2715
2716
2717
2% SLOPE
2712
BOUNDARY SUMP 12B
EL. 2711.50
SEE DETAIL 1/010
FOR LOS DRAINAGE LATER DEVELOPMENT
1% SLOPE (MIN.)
UFFER
BOUNDARY SUMP 12A
EL. 2713.00
(SEE DETAIL 1/010 FOR LOS DRAINAGE LATER DEVELOPMENT AT SUMP)
1% SLOPE (MIN.)
EL. 2717.1
EL. 2717.2
EL. 2717.2
EL. 2717.4
(0.5:1 TYP. SEE NOTE 3)
N9,745.17
E10,020.00
2720 2730 2740 2750 2760 2770 2780
581.70'
N8938'13"E
0.5%
(MIN., SEE NOTE 6)
5'(TYP.)
TOP OF
EL. 2782
HORIZONTAL CONTROL LINE (SEE NOTE 2)
N 9610.6
E 10817.4
SUMP @ EXCAVATION
SEE DETAIL
1
008
LOCAL HIGH POINT OF ANCHOR TRENCH EL.2786.0 (GRADE LOCALLY AS NECESSARY, SEE NOTE 6)
150' BUFFER (MIN)
ANCHOR TRENCH (LIMIT OF LINER)
EDGE OF LOW LEVEL RADIOACTIVE WASTE (LLRW) TRENCHES
N 9512.76
E 10735.86
Figure 1
Approximate Scale 1" = 50'
Reference: Excavation Plan and Limits of Grading Cell 12, ESI, D-95284-BL003, march, 96,
N 9508.08
E 9994.84
2785
2785

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By JH Date 3/4/96 Subject Slope stability during Sheet No. 9 of 9

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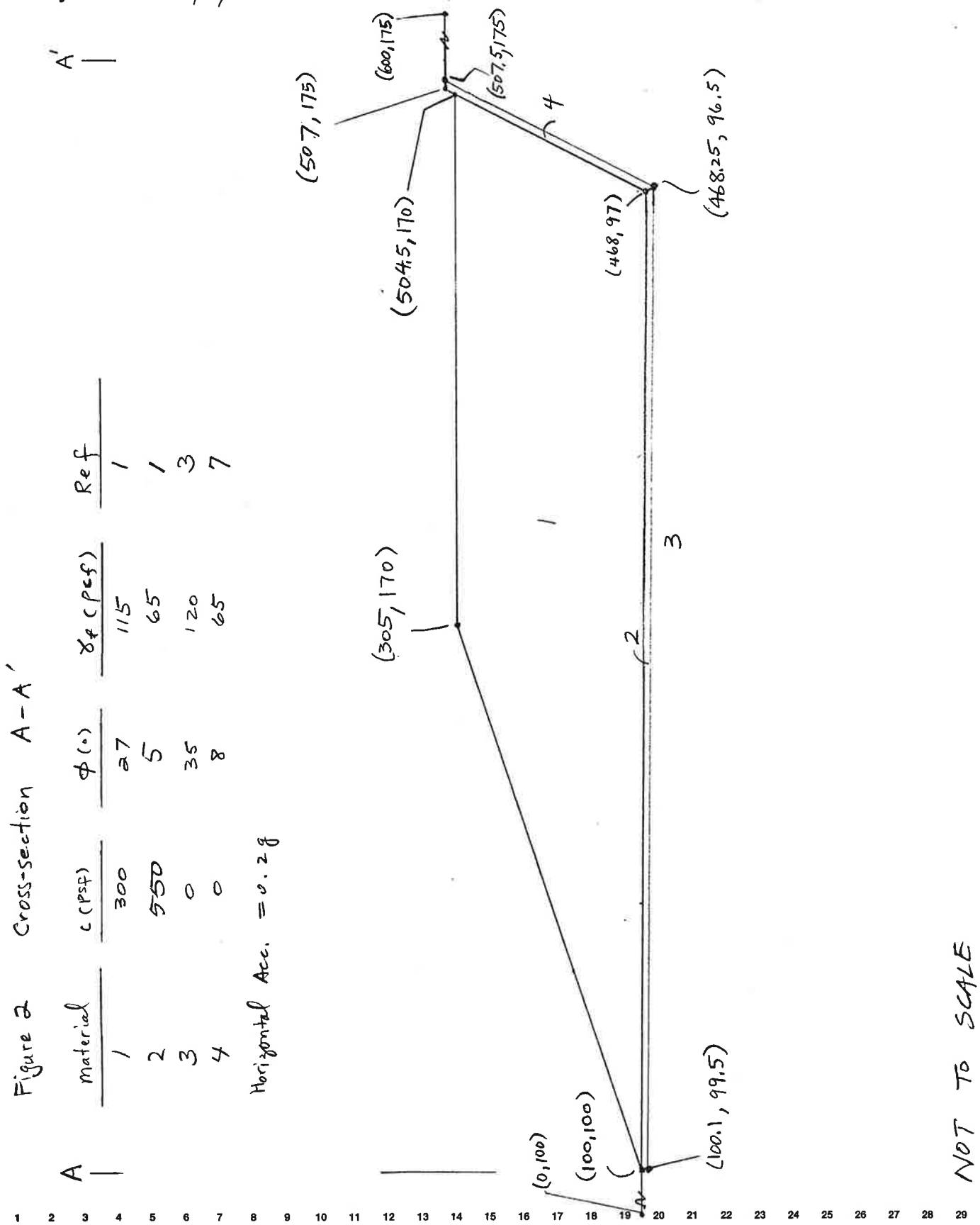
Figure 2 Cross-section A-A'

| Material | c (PSF) | φ (°) | γ (pcf) | Ref |
|---|---|---|---|---|
| 1 | 300 | 27 | 115 | 1 |
| 2 | 550 | 5 | 65 | 1 |
| 3 | 0 | 35 | 120 | 3 |
| 4 | 0 | 8 | 65 | 7 |

Horizontal Acc. = 0.2g



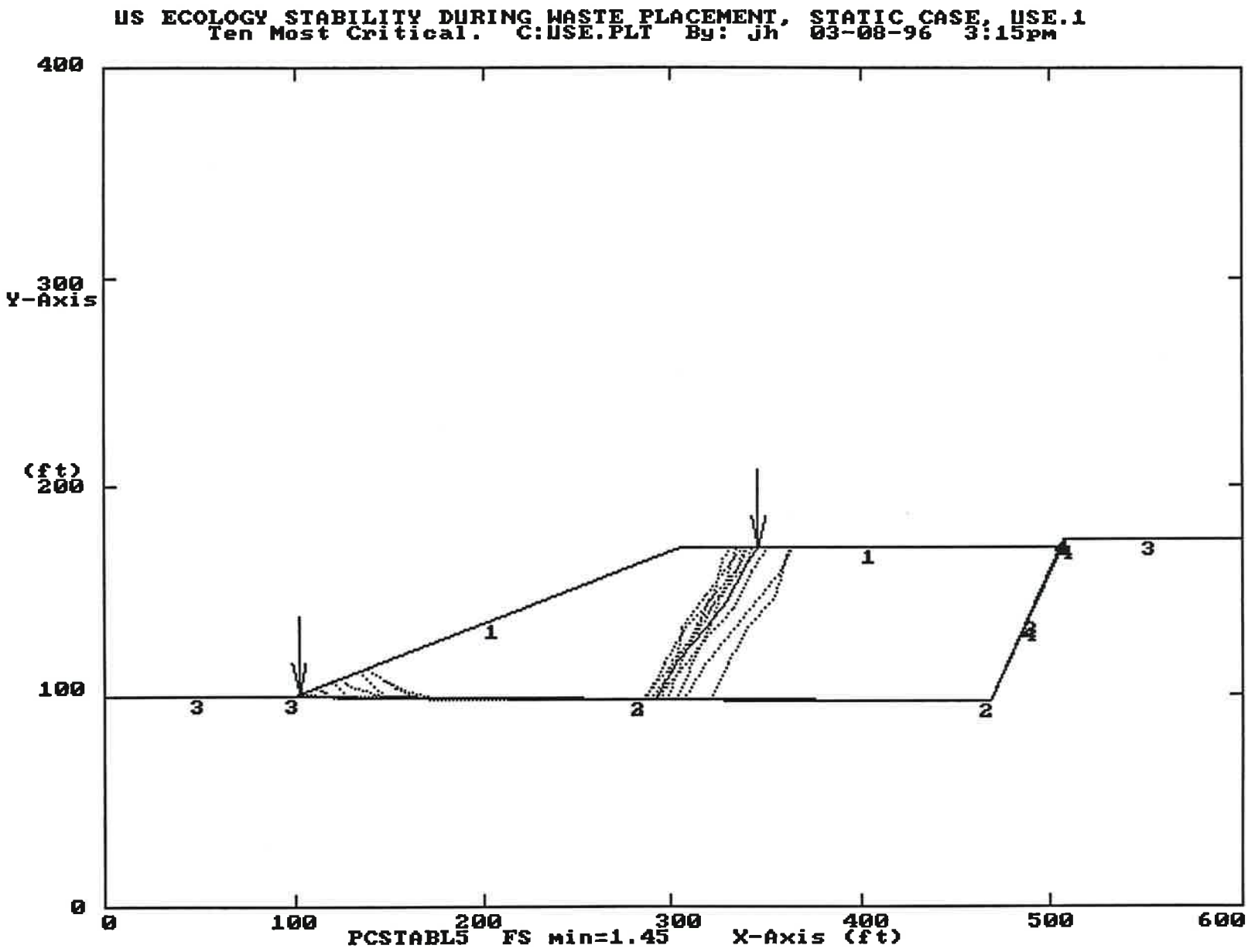



Recyclable and made from recycled paper.

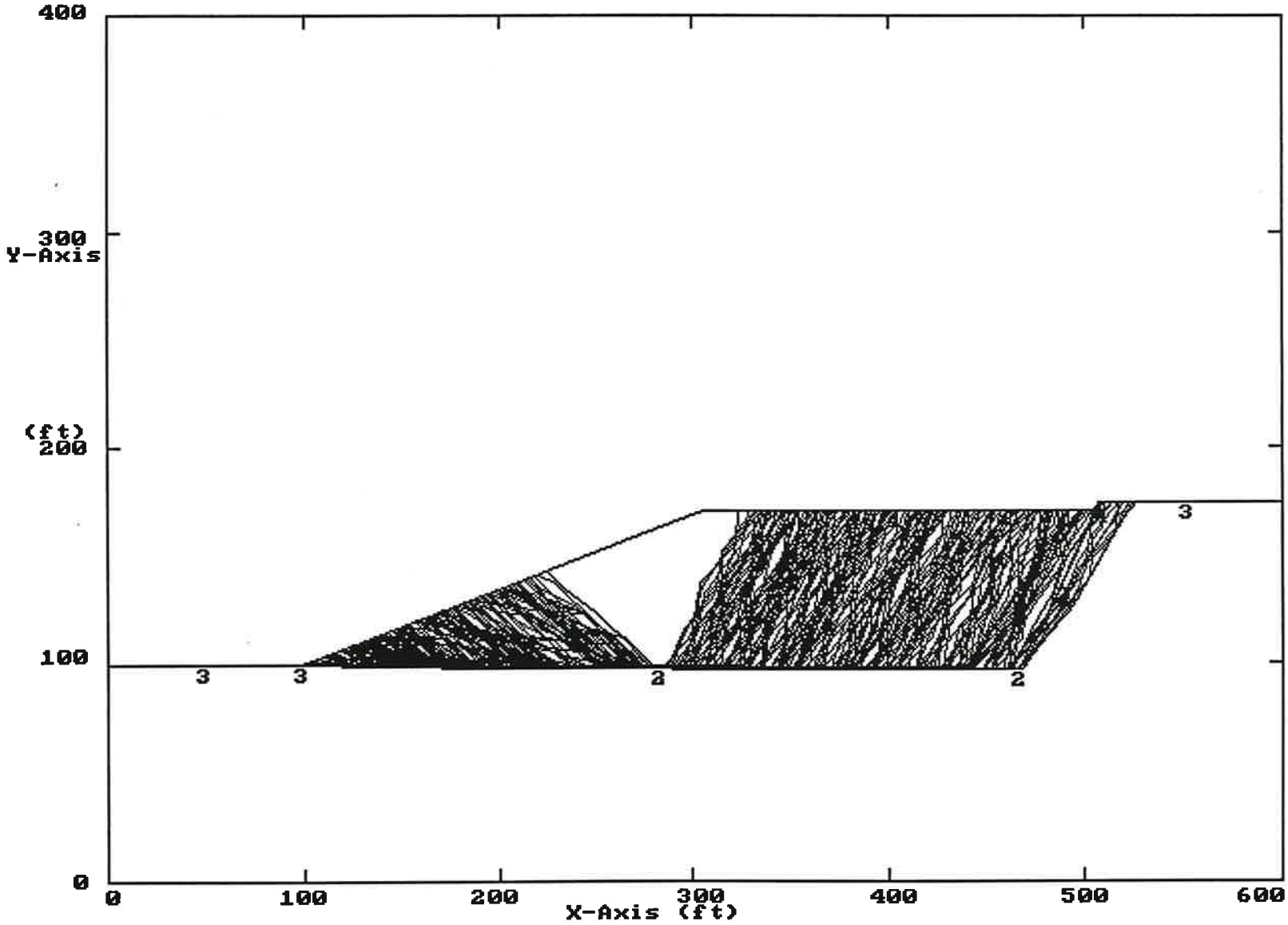
STATIC STABILITY

FILE NAME = USE.1

F.S. = 1.45


US ECOLOGY STABILITY DURING WASTE PLACEMENT, STATIC CASE, USE.1
Ten Most Critical. C:USE.PLT By: jh 03-08-96 3:15pm
400
300
Y-Axis
(ft)
200
100
0
0
100
200
300
400
500
600
PCSTABL5 FS min=1.45
X-Axis (ft)
1
1
1
2
2
3
3
3

US ECOLOGY STABILITY DURING WASTE PLACEMENT, STATIC CASE, USE.1
All surfaces evaluated. C:USE.PLT By: jh 03-08-96 3:15pm



```
PROFIL        C:USE.1  PCSTABL Version 5
US ECOLOGY STABILITY DURING WASTE       PLACEMENT, STATIC CASE, USE.1
12 6
0. 100. 100. 100. 3
100. 100. 305. 170. 1
305. 170. 504.5 170. 1
504.5 170. 507. 175. 4
507. 175. 507.5 175. 4
507.5 175. 600. 175. 3
100. 100. 468. 97. 2
468. 97. 504.5 170. 4
100. 100. 100.1 99.5 3
468. 97. 468.25 96.5 2
100.1 99.5 468.3 96.5 3
468.3 96.5 507.5 175. 3
SOIL
4
115. 115. 300. 27. 0. 0. 0
65. 65. 550. 5. 0. 0. 0
120. 120. 0. 35. 0. 0. 0
65. 65. 0. 8. 0. 0. 0
BLOCK -Sliding block, search.
200 2 20.
101. 99.75 280. 98.3 0.5
281. 98.3 468. 96.75 0.5
```

** PCSTABL5 **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date: 03-08-96
Time of Run: 3:15pm
Run By: jh
Input Data Filename: C:USE.1
Output Filename: C:USE.OUT
Plotted Output Filename: C:USE.PLT

PROBLEM DESCRIPTION US ECOLOGY STABILITY DURING WASTE PLACEMENT, STATIC CASE, USE.1

BOUNDARY COORDINATES

6 Top Boundaries
12 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 100.00 | 100.00 | 100.00 | 3 |
| 2 | 100.00 | 100.00 | 305.00 | 170.00 | 1 |
| 3 | 305.00 | 170.00 | 504.50 | 170.00 | 1 |
| 4 | 504.50 | 170.00 | 507.00 | 175.00 | 4 |
| 5 | 507.00 | 175.00 | 507.50 | 175.00 | 4 |
| 6 | 507.50 | 175.00 | 600.00 | 175.00 | 3 |
| 7 | 100.00 | 100.00 | 468.00 | 97.00 | 2 |
| 8 | 468.00 | 97.00 | 504.50 | 170.00 | 4 |
| 9 | 100.00 | 100.00 | 100.10 | 99.50 | 3 |
| 10 | 468.00 | 97.00 | 468.25 | 96.50 | 2 |
| 11 | 100.10 | 99.50 | 468.30 | 96.50 | 3 |
| 12 | 468.30 | 96.50 | 507.50 | 175.00 | 3 |

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Total Saturated Cohesion Friction Pore Pressure Piez.

| Type No. | Unit Wt. (pcf) | Unit Wt. (pcf) | Intercept (psf) | Angle (deg) | Pressure Param. | Constant (psf) | Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 65.0 | 65.0 | 550.0 | 5.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | .0 | 35.0 | .00 | .0 | 0 |
| 4 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

200 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 20.0

| Box No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Height (ft) |
|---|---|---|---|---|---|
| 1 | 101.00 | 99.75 | 280.00 | 98.30 | .50 |
| 2 | 281.00 | 98.30 | 468.00 | 96.75 | .50 |

Factor Of Safety Calculation Has Gone Through Ten Iterations

The Trial Failure Surface In Question Is Defined By The Following 11 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 214.37 | 139.05 |
| 2 | 221.20 | 133.04 |
| 3 | 238.11 | 122.36 |
| 4 | 253.78 | 109.94 |
| 5 | 270.20 | 98.51 |
| 6 | 323.63 | 98.13 |
| 7 | 323.94 | 118.13 |
| 8 | 337.10 | 133.19 |
| 9 | 350.81 | 147.74 |
| 10 | 354.51 | 167.40 |
| 11 | 356.89 | 170.00 |

Factor Of Safety For The Preceding Specified Surface = 11.536

Factor Of Safety Calculation Has Gone Through Ten Iterations

The Trial Failure Surface In Question Is Defined
By The Following 11 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 133.71 | 111.51 |
| 2 | 146.82 | 105.79 |
| 3 | 166.61 | 102.87 |
| 4 | 186.56 | 101.41 |
| 5 | 206.40 | 98.89 |
| 6 | 454.20 | 96.63 |
| 7 | 454.90 | 116.62 |
| 8 | 466.38 | 133.00 |
| 9 | 466.79 | 153.00 |
| 10 | 480.14 | 167.89 |
| 11 | 481.50 | 170.00 |

Factor Of Safety For The Preceding Specified Surface = 5.447

Factor Of Safety Calculation Has Gone Through Ten Iterations

The Trial Failure Surface In Question Is Defined
By The Following 9 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 147.74 | 116.30 |
| 2 | 150.67 | 113.38 |
| 3 | 164.92 | 99.35 |
| 4 | 398.82 | 97.52 |
| 5 | 399.06 | 117.52 |
| 6 | 400.49 | 137.47 |
| 7 | 413.80 | 152.40 |
| 8 | 427.00 | 167.42 |
| 9 | 428.25 | 170.00 |

Factor Of Safety For The Preceding Specified Surface = 5.468

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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 Janbu Method * *

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 101.65 | 100.56 |
| 2 | 112.00 | 99.79 |
| 3 | 291.31 | 98.38 |
| 4 | 301.95 | 115.32 |
| 5 | 315.99 | 129.56 |
| 6 | 329.08 | 144.68 |
| 7 | 339.11 | 161.98 |
| 8 | 345.60 | 170.00 |

*** 1.450 ***

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 102.59 | 100.89 |
| 2 | 103.96 | 99.59 |
| 3 | 297.52 | 98.33 |
| 4 | 305.92 | 116.48 |
| 5 | 314.54 | 134.52 |
| 6 | 328.22 | 149.12 |
| 7 | 337.97 | 166.58 |
| 8 | 339.61 | 170.00 |

*** 1.502 ***

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Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 120.15 | 106.88 |
| 2 | 128.87 | 99.33 |
| 3 | 306.74 | 98.18 |
| 4 | 317.83 | 114.82 |
| 5 | 331.28 | 129.62 |
| 6 | 344.35 | 144.76 |
| 7 | 357.38 | 159.93 |
| 8 | 363.42 | 170.00 |

*** 1.517 ***

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 104.52 | 101.54 |
| 2 | 113.84 | 99.81 |
| 3 | 293.95 | 97.99 |
| 4 | 300.61 | 116.85 |
| 5 | 314.68 | 131.07 |
| 6 | 325.64 | 147.80 |
| 7 | 339.77 | 161.96 |
| 8 | 341.63 | 170.00 |

*** 1.524 ***

---

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 134.46 | 111.77 |
| 2 | 148.23 | 99.45 |
| 3 | 285.69 | 98.41 |
| 4 | 296.08 | 115.50 |
| 5 | 307.71 | 131.77 |
| 6 | 321.85 | 145.92 |
| 7 | 327.02 | 165.24 |
| 8 | 331.77 | 170.00 |

*** 1.539 ***

Failure Surface Specified By 9 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 138.47 | 113.14 |
| 2 | 147.51 | 107.17 |
| 3 | 165.80 | 99.08 |
| 4 | 284.32 | 98.32 |
| 5 | 298.39 | 112.54 |
| 6 | 311.56 | 127.59 |
| 7 | 319.75 | 145.83 |
| 8 | 333.88 | 159.99 |
| 9 | 338.17 | 170.00 |

*** 1.545 ***

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Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 111.52 | 103.93 |
| 2 | 118.75 | 99.83 |
| 3 | 138.75 | 99.56 |
| 4 | 303.18 | 98.34 |
| 5 | 311.52 | 116.52 |
| 6 | 320.94 | 134.16 |
| 7 | 327.55 | 153.04 |
| 8 | 333.72 | 170.00 |

*** 1.554 ***

Failure Surface Specified By 9 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 124.13 | 108.24 |
| 2 | 128.02 | 105.41 |
| 3 | 147.14 | 99.54 |
| 4 | 288.57 | 98.20 |
| 5 | 301.33 | 113.60 |
| 6 | 305.73 | 133.11 |
| 7 | 318.82 | 148.23 |
| 8 | 331.49 | 163.70 |
| 9 | 337.80 | 170.00 |

*** 1.576 ***

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Failure Surface Specified By 9 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 139.41 | 113.46 |
| 2 | 154.64 | 103.13 |
| 3 | 174.23 | 99.10 |
| 4 | 292.04 | 97.97 |
| 5 | 305.89 | 112.39 |
| 6 | 319.06 | 127.44 |
| 7 | 333.18 | 141.61 |
| 8 | 342.21 | 159.45 |
| 9 | 350.71 | 170.00 |

*** 1.589 ***

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 112.85 | 104.39 |
| 2 | 120.48 | 99.35 |
| 3 | 321.35 | 97.81 |
| 4 | 330.25 | 115.72 |
| 5 | 340.30 | 133.01 |
| 6 | 354.39 | 147.21 |
| 7 | 360.83 | 166.14 |
| 8 | 361.13 | 170.00 |

*** 1.627 ***

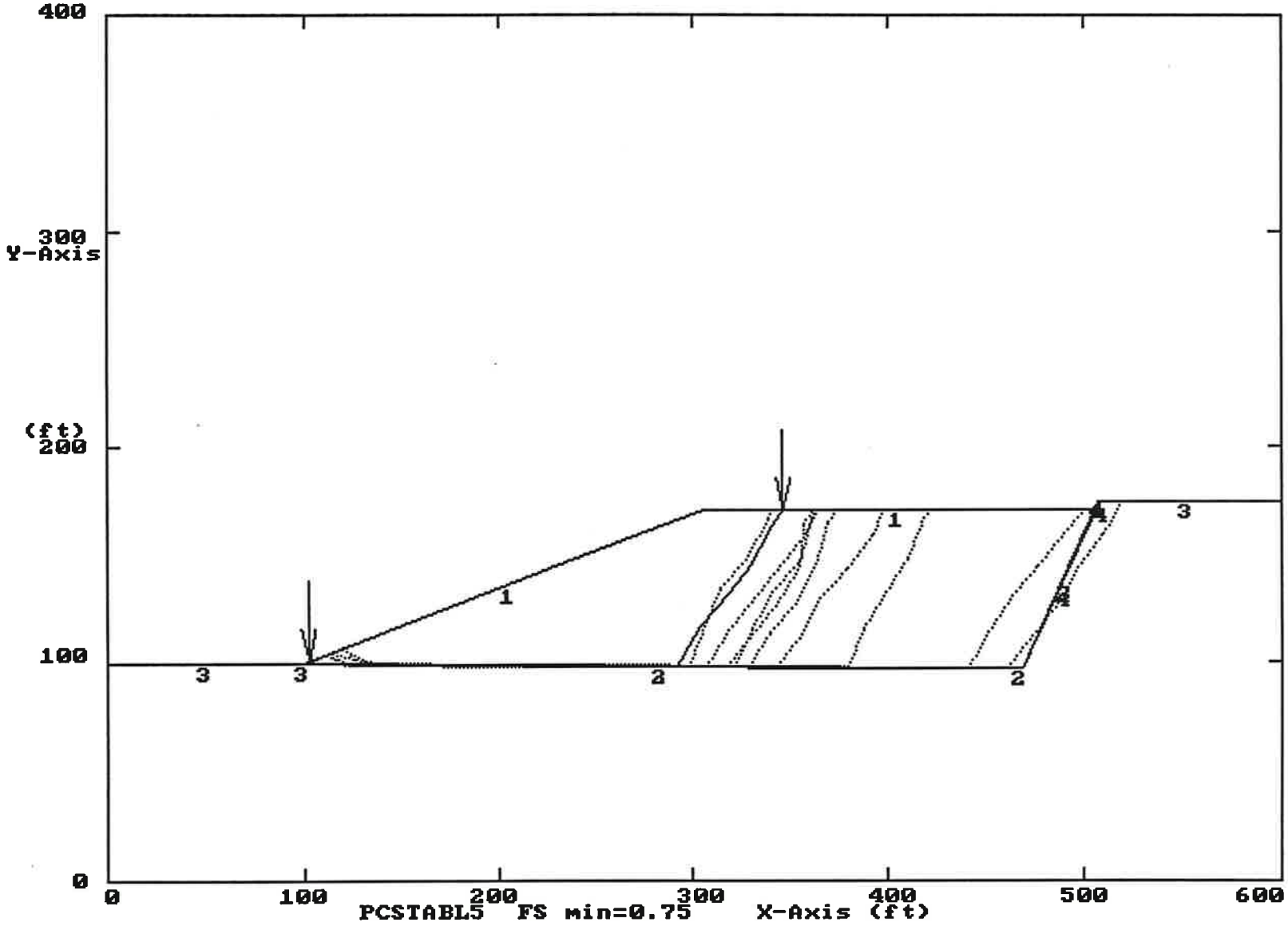
PSEUDO-STATIC STABILITY

FILE NAME = USE.2

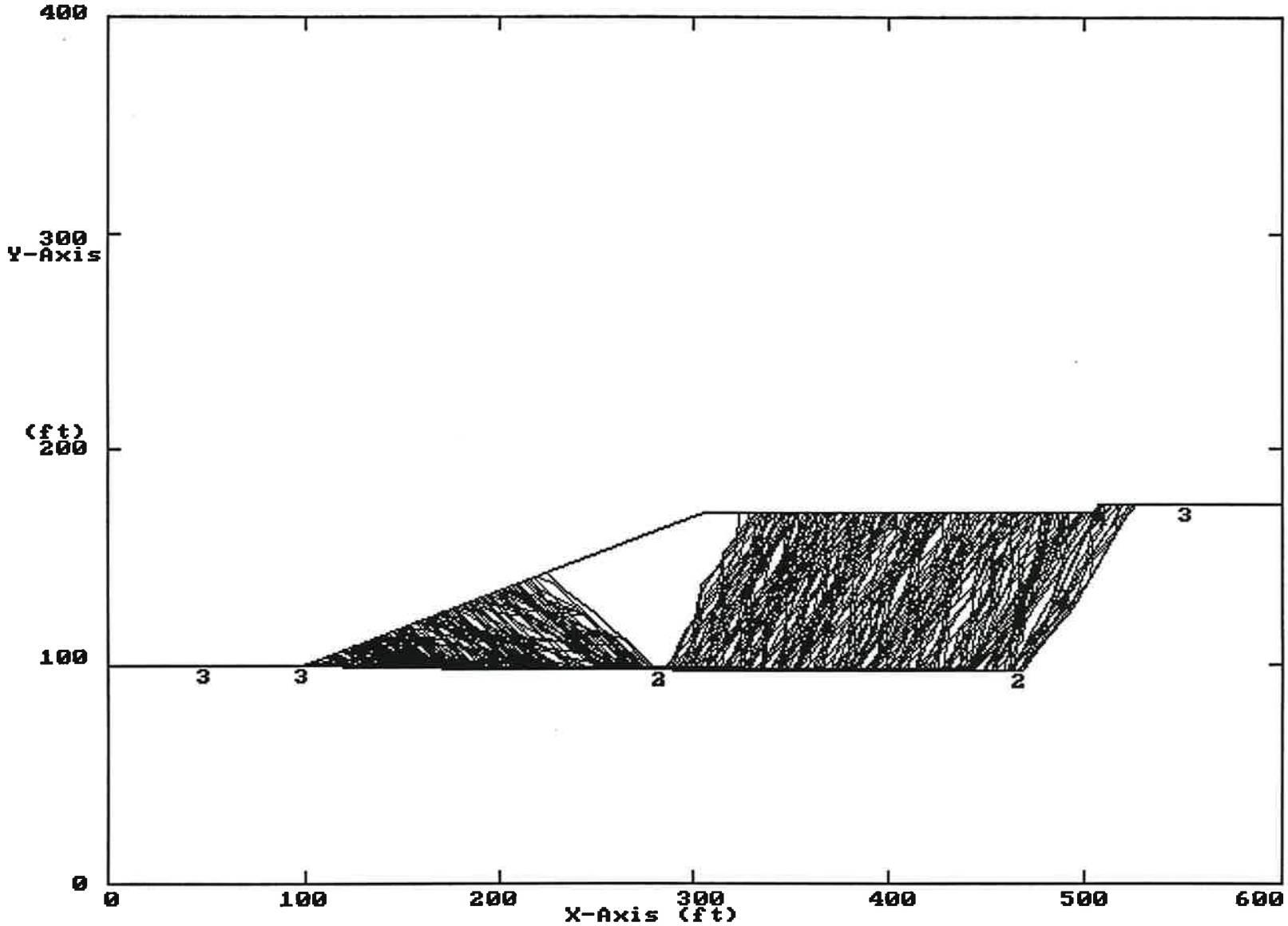
F.S. = 0.75

$a = 0.2\ g$

US ECOLOGY STABILITY DURING WASTE PLACEMENT, PSEUDO-STATIC, USE.2
Ten Most Critical. C:USE.PLT By: jh 03-08-96 3:23pm



US ECOLOGY STABILITY DURING WASTE PLACEMENT, PSEUDO-STATIC, USE.2
All surfaces evaluated. C:USE.PLT By: jh 03-08-96 3:23pm



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US ECOLOGY STABILITY DURING WASTE       PLACEMENT, PSEUDO-STATIC, USE.2
12 6
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100. 100. 305. 170. 1
305. 170. 504.5 170. 1
504.5 170. 507. 175. 4
507. 175. 507.5 175. 4
507.5 175. 600. 175. 3
100. 100. 468. 97. 2
468. 97. 504.5 170. 4
100. 100. 100.1 99.5 3
468. 97. 468.25 96.5 2
100.1 99.5 468.3 96.5 3
468.3 96.5 507.5 175. 3
SOIL
4
115. 115. 300. 27. 0. 0. 0
65. 65. 550. 5. 0. 0. 0
120. 120. 0. 35. 0. 0. 0
65. 65. 0. 8. 0. 0. 0
EQUAKE
0.2 0. 0.
BLOCK -Sliding block, search.
200 2 20.
101. 99.75 280. 98.3 0.5
281. 98.3 468. 96.75 0.5
```

** PCSTABL5 **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date: 03-08-96
Time of Run: 3:23pm
Run By: jh
Input Data Filename: C:USE.2
Output Filename: C:USE.OUT
Plotted Output Filename: C:USE.PLT

PROBLEM DESCRIPTION US ECOLOGY STABILITY DURING WASTE PLACEMENT, PSEUDO-STATIC, USE.2

BOUNDARY COORDINATES

6 Top Boundaries
12 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 100.00 | 100.00 | 100.00 | 3 |
| 2 | 100.00 | 100.00 | 305.00 | 170.00 | 1 |
| 3 | 305.00 | 170.00 | 504.50 | 170.00 | 1 |
| 4 | 504.50 | 170.00 | 507.00 | 175.00 | 4 |
| 5 | 507.00 | 175.00 | 507.50 | 175.00 | 4 |
| 6 | 507.50 | 175.00 | 600.00 | 175.00 | 3 |
| 7 | 100.00 | 100.00 | 468.00 | 97.00 | 2 |
| 8 | 468.00 | 97.00 | 504.50 | 170.00 | 4 |
| 9 | 100.00 | 100.00 | 100.10 | 99.50 | 3 |
| 10 | 468.00 | 97.00 | 468.25 | 96.50 | 2 |
| 11 | 100.10 | 99.50 | 468.30 | 96.50 | 3 |
| 12 | 468.30 | 96.50 | 507.50 | 175.00 | 3 |

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Total Saturated Cohesion Friction Pore Pressure Piez.

| Type No. | Unit Wt. (pcf) | Unit Wt. (pcf) | Intercept (psf) | Angle (deg) | Pressure Param. | Constant (psf) | Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 65.0 | 65.0 | 550.0 | 5.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | .0 | 35.0 | .00 | .0 | 0 |
| 4 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .200 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

---

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

200 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 20.0

| Box No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Height (ft) |
|---|---|---|---|---|---|
| 1 | 101.00 | 99.75 | 280.00 | 98.30 | .50 |
| 2 | 281.00 | 98.30 | 468.00 | 96.75 | .50 |

---

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 Janbu Method * *

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 101.65 | 100.56 |
| 2 | 112.00 | 99.79 |

| | | |
|---|---|---|
| 3 | 291.31 | 98.38 |
| 4 | 301.95 | 115.32 |
| 5 | 315.99 | 129.56 |
| 6 | 329.08 | 144.68 |
| 7 | 339.11 | 161.98 |
| 8 | 345.60 | 170.00 |

*** .747 ***

Failure Surface Specified By 9 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 112.70 | 104.34 |
| 2 | 113.73 | 104.04 |
| 3 | 133.20 | 99.43 |
| 4 | 459.91 | 96.68 |
| 5 | 473.05 | 111.76 |
| 6 | 486.62 | 126.44 |
| 7 | 497.11 | 143.47 |
| 8 | 510.02 | 158.75 |
| 9 | 519.81 | 175.00 |

*** .750 ***

---

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 104.59 | 101.57 |
| 2 | 106.98 | 99.60 |
| 3 | 341.94 | 97.71 |
| 4 | 355.41 | 112.50 |
| 5 | 365.50 | 129.76 |
| 6 | 379.50 | 144.05 |
| 7 | 393.13 | 158.68 |
| 8 | 396.46 | 170.00 |

*** .756 ***

Failure Surface Specified By 8 Coordinate Points

| Point | X-Surf | Y-Surf |
|---|---|---|

| No. | (ft) | (ft) |
|---|---|---|
| 1 | 106.50 | 102.22 |
| 2 | 109.51 | 99.71 |
| 3 | 327.78 | 97.94 |
| 4 | 339.50 | 114.15 |
| 5 | 353.62 | 128.31 |
| 6 | 363.37 | 145.77 |
| 7 | 368.87 | 165.00 |
| 8 | 373.82 | 170.00 |

*** .760 ***

---

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 112.85 | 104.39 |
| 2 | 120.48 | 99.35 |
| 3 | 321.35 | 97.81 |
| 4 | 330.25 | 115.72 |
| 5 | 340.30 | 133.01 |
| 6 | 354.39 | 147.21 |
| 7 | 360.83 | 166.14 |
| 8 | 361.13 | 170.00 |

*** .760 ***

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 102.59 | 100.89 |
| 2 | 103.96 | 99.59 |
| 3 | 297.52 | 98.33 |
| 4 | 305.92 | 116.48 |
| 5 | 314.54 | 134.52 |
| 6 | 328.22 | 149.12 |
| 7 | 337.97 | 166.58 |
| 8 | 339.61 | 170.00 |

*** .761 ***

---

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 110.93 | 103.73 |
| 2 | 123.28 | 99.67 |
| 3 | 441.14 | 96.73 |
| 4 | 450.85 | 114.22 |
| 5 | 462.41 | 130.54 |
| 6 | 476.30 | 144.93 |
| 7 | 489.55 | 159.91 |
| 8 | 499.27 | 170.00 |

*** .762 ***

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 104.96 | 101.69 |
| 2 | 108.02 | 99.47 |
| 3 | 316.74 | 98.14 |
| 4 | 330.30 | 112.84 |
| 5 | 343.54 | 127.82 |
| 6 | 354.38 | 144.63 |
| 7 | 357.23 | 164.43 |
| 8 | 360.96 | 170.00 |

*** .763 ***

---

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 119.18 | 106.55 |
| 2 | 135.60 | 99.44 |
| 3 | 378.78 | 97.56 |
| 4 | 387.75 | 115.44 |
| 5 | 398.23 | 132.47 |
| 6 | 410.49 | 148.27 |
| 7 | 418.77 | 166.48 |
| 8 | 420.52 | 170.00 |

*** .766 ***

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 120.15 | 106.88 |
| 2 | 128.87 | 99.33 |
| 3 | 306.74 | 98.18 |
| 4 | 317.83 | 114.82 |
| 5 | 331.28 | 129.62 |
| 6 | 344.35 | 144.76 |
| 7 | 357.38 | 159.93 |
| 8 | 363.42 | 170.00 |

*** .767 ***

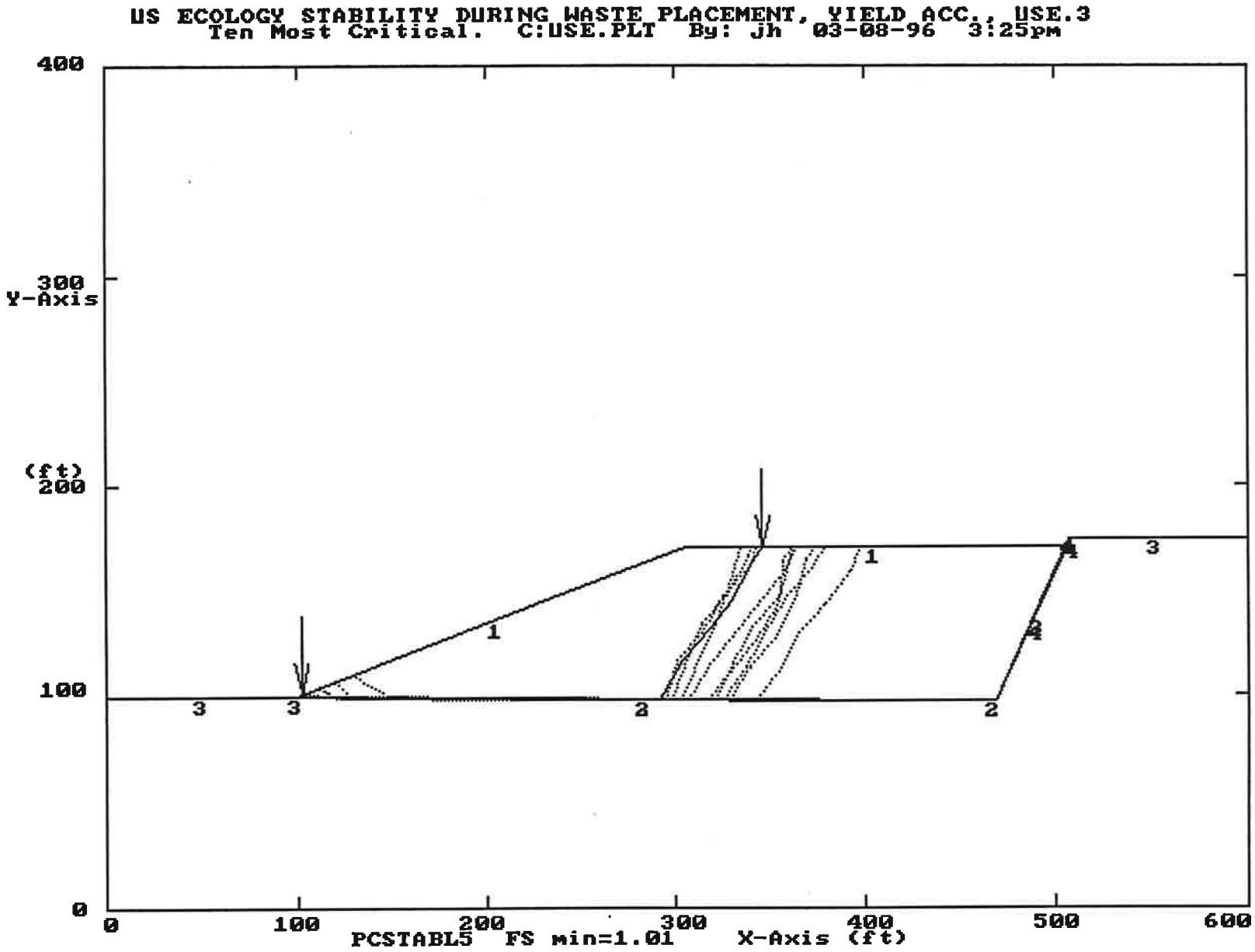
YIELD ACCELERATION

DETERMINATION

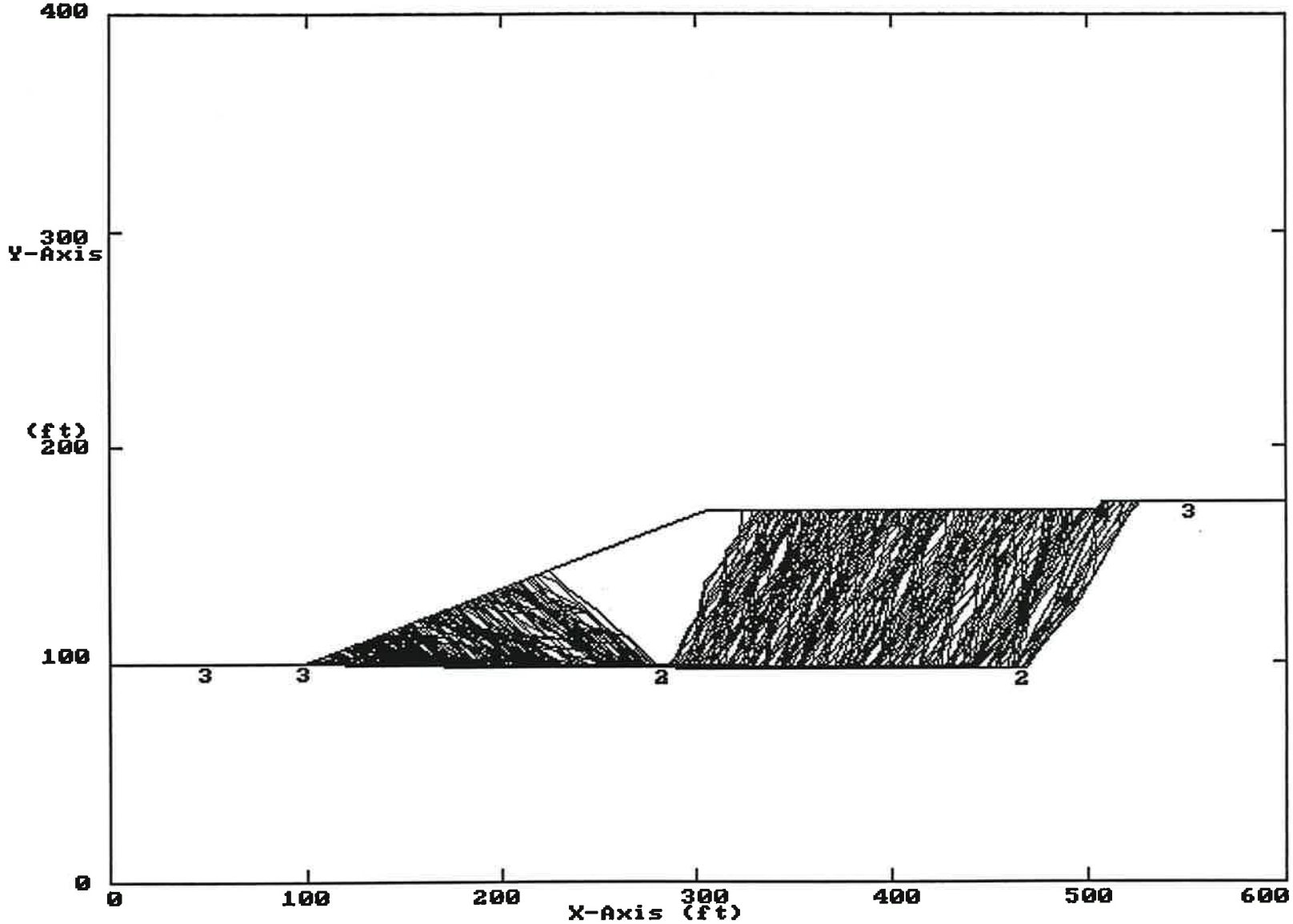
FILE NAME = USE.3

F.S. = 1.01

$a = 0.1\ g$


US ECOLOGY STABILITY DURING WASTE PLACEMENT, YIELD ACC., USE.3
Ten Most Critical. C:USE.PLT By: jh 03-08-96 3:25pm
400
300
Y-Axis
(ft)
200
100
0
0
100
200
300
400
500
600
PCSTABL5 FS min=1.01 X-Axis (ft)
1
2
3
4

US ECOLOGY STABILITY DURING WASTE PLACEMENT, YIELD ACC., USE.3
All surfaces evaluated. C:USE.PLT By: jh 03-08-96 3:25pm



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PROFIL        C:USE.3  PCSTABL Version 5
US ECOLOGY STABILITY DURING WASTE        PLACEMENT, YIELD ACC., USE.3
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100. 100. 305. 170. 1
305. 170. 504.5 170. 1
504.5 170. 507. 175. 4
507. 175. 507.5 175. 4
507.5 175. 600. 175. 3
100. 100. 468. 97. 2
468. 97. 504.5 170. 4
100. 100. 100.1 99.5 3
468. 97. 468.25 96.5 2
100.1 99.5 468.3 96.5 3
468.3 96.5 507.5 175. 3
SOIL
4
115. 115. 300. 27. 0. 0. 0
65. 65. 550. 5. 0. 0. 0
120. 120. 0. 35. 0. 0. 0
65. 65. 0. 8. 0. 0. 0
EQUAKE
0.1 0. 0.
BLOCK -Sliding block, search.
200 2 20.
101. 99.75 280. 98.3 0.5
281. 98.3 468. 96.75 0.5
```

** PCSTABL5 **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date: 03-08-96
Time of Run: 3:25pm
Run By: jh
Input Data Filename: C:USE.3
Output Filename: C:USE.OUT
Plotted Output Filename: C:USE.PLT

PROBLEM DESCRIPTION US ECOLOGY STABILITY DURING WASTE PLACEMENT, YIELD ACC., USE.3

BOUNDARY COORDINATES

6 Top Boundaries
12 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 100.00 | 100.00 | 100.00 | 3 |
| 2 | 100.00 | 100.00 | 305.00 | 170.00 | 1 |
| 3 | 305.00 | 170.00 | 504.50 | 170.00 | 1 |
| 4 | 504.50 | 170.00 | 507.00 | 175.00 | 4 |
| 5 | 507.00 | 175.00 | 507.50 | 175.00 | 4 |
| 6 | 507.50 | 175.00 | 600.00 | 175.00 | 3 |
| 7 | 100.00 | 100.00 | 468.00 | 97.00 | 2 |
| 8 | 468.00 | 97.00 | 504.50 | 170.00 | 4 |
| 9 | 100.00 | 100.00 | 100.10 | 99.50 | 3 |
| 10 | 468.00 | 97.00 | 468.25 | 96.50 | 2 |
| 11 | 100.10 | 99.50 | 468.30 | 96.50 | 3 |
| 12 | 468.30 | 96.50 | 507.50 | 175.00 | 3 |

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Total Saturated Cohesion Friction Pore Pressure Piez.

| Type No. | Unit Wt. (pcf) | Unit Wt. (pcf) | Intercept (psf) | Angle (deg) | Pressure Param. | Constant (psf) | Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 115.0 | 115.0 | 300.0 | 27.0 | .00 | .0 | 0 |
| 2 | 65.0 | 65.0 | 550.0 | 5.0 | .00 | .0 | 0 |
| 3 | 120.0 | 120.0 | .0 | 35.0 | .00 | .0 | 0 |
| 4 | 65.0 | 65.0 | .0 | 8.0 | .00 | .0 | 0 |

A Horizontal Earthquake Loading Coefficient
Of .100 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

---

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

200 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 20.0

| Box No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Height (ft) |
|---|---|---|---|---|---|
| 1 | 101.00 | 99.75 | 280.00 | 98.30 | .50 |
| 2 | 281.00 | 98.30 | 468.00 | 96.75 | .50 |

Factor Of Safety Calculation Has Gone Through Ten Iterations

The Trial Failure Surface In Question Is Defined
By The Following 11 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 214.37 | 139.05 |
| 2 | 221.20 | 133.04 |
| 3 | 238.11 | 122.36 |

| | | |
|---|---|---|
| 4 | 253.78 | 109.94 |
| 5 | 270.20 | 98.51 |
| 6 | 323.63 | 98.13 |
| 7 | 323.94 | 118.13 |
| 8 | 337.10 | 133.19 |
| 9 | 350.81 | 147.74 |
| 10 | 354.51 | 167.40 |
| 11 | 356.89 | 170.00 |

Factor Of Safety For The Preceding Specified Surface = 4.871

---

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 Janbu Method * *

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 101.65 | 100.56 |
| 2 | 112.00 | 99.79 |
| 3 | 291.31 | 98.38 |
| 4 | 301.95 | 115.32 |
| 5 | 315.99 | 129.56 |
| 6 | 329.08 | 144.68 |
| 7 | 339.11 | 161.98 |
| 8 | 345.60 | 170.00 |

*** 1.005 ***

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 120.15 | 106.88 |
| 2 | 128.87 | 99.33 |
| 3 | 306.74 | 98.18 |
| 4 | 317.83 | 114.82 |
| 5 | 331.28 | 129.62 |
| 6 | 344.35 | 144.76 |
| 7 | 357.38 | 159.93 |
| 8 | 363.42 | 170.00 |

*** 1.029 ***

---

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 102.59 | 100.89 |
| 2 | 103.96 | 99.59 |
| 3 | 297.52 | 98.33 |
| 4 | 305.92 | 116.48 |
| 5 | 314.54 | 134.52 |
| 6 | 328.22 | 149.12 |
| 7 | 337.97 | 166.58 |
| 8 | 339.61 | 170.00 |

*** 1.031 ***

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 104.52 | 101.54 |
| 2 | 113.84 | 99.81 |
| 3 | 293.95 | 97.99 |
| 4 | 300.61 | 116.85 |
| 5 | 314.68 | 131.07 |
| 6 | 325.64 | 147.80 |
| 7 | 339.77 | 161.96 |
| 8 | 341.63 | 170.00 |

*** 1.045 ***

---

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 111.52 | 103.93 |
| 2 | 118.75 | 99.83 |
| 3 | 138.75 | 99.56 |
| 4 | 303.18 | 98.34 |
| 5 | 311.52 | 116.52 |

| | | |
|---|---|---|
| 6 | 320.94 | 134.16 |
| 7 | 327.55 | 153.04 |
| 8 | 333.72 | 170.00 |

*** 1.049 ***

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 112.85 | 104.39 |
| 2 | 120.48 | 99.35 |
| 3 | 321.35 | 97.81 |
| 4 | 330.25 | 115.72 |
| 5 | 340.30 | 133.01 |
| 6 | 354.39 | 147.21 |
| 7 | 360.83 | 166.14 |
| 8 | 361.13 | 170.00 |

*** 1.058 ***

---

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 104.96 | 101.69 |
| 2 | 108.02 | 99.47 |
| 3 | 316.74 | 98.14 |
| 4 | 330.30 | 112.84 |
| 5 | 343.54 | 127.82 |
| 6 | 354.38 | 144.63 |
| 7 | 357.23 | 164.43 |
| 8 | 360.96 | 170.00 |

*** 1.062 ***

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 106.50 | 102.22 |
| 2 | 109.51 | 99.71 |

| | | |
|---|---|---|
| 3 | 327.78 | 97.94 |
| 4 | 339.50 | 114.15 |
| 5 | 353.62 | 128.31 |
| 6 | 363.37 | 145.77 |
| 7 | 368.87 | 165.00 |
| 8 | 373.82 | 170.00 |

*** 1.064 ***

---

Failure Surface Specified By 8 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 104.59 | 101.57 |
| 2 | 106.98 | 99.60 |
| 3 | 341.94 | 97.71 |
| 4 | 355.41 | 112.50 |
| 5 | 365.50 | 129.76 |
| 6 | 379.50 | 144.05 |
| 7 | 393.13 | 158.68 |
| 8 | 396.46 | 170.00 |

*** 1.071 ***

Failure Surface Specified By 9 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 129.01 | 109.91 |
| 2 | 130.23 | 109.37 |
| 3 | 147.57 | 99.41 |
| 4 | 324.82 | 98.08 |
| 5 | 337.18 | 113.80 |
| 6 | 348.08 | 130.57 |
| 7 | 359.08 | 147.27 |
| 8 | 372.62 | 161.99 |
| 9 | 377.71 | 170.00 |

*** 1.073 ***

# C.3 SURFICIAL SOIL IMPROVEMENTS

# ENVIRONMENTAL SOLUTIONS, INC.

By WB Date 1/9/96 Subject _____ Sheet No. 1 of 5

Chkd. By MJL Date 1/12/96 _____ Proj. No. 95-284

## SURFICIAL SOIL IMPROVEMENT

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# ENVIRONMENTAL SOLUTIONS, INC.

By UB Date 1/9/96 Subject SURFICIAL SOIL IMPROVEMENT Sheet No. 2 of 5

Chkd. By MLL Date 1/12/96 Proj. No. 95-284

## PURPOSE

EVALUATE COHESION NEEDED FOR SURFICIAL SOIL FOR STABILITY.

## REFERENCES

1. PECK, HANSON, THORNBURN, "FOUNDATION ENGINEERING", 1974, JOHN WILEY & SONS, INC.

2. GRANT ENVIRONMENTAL, JULY 14, 1994, "GEOTECHNICAL INVESTIGATION FOR CELL 12 AT THE US ECOLOGY HAZARDOUS WASTE MANAGEMENT FACILITY, BEATTY, NEVADA."

## ASSUMPTIONS

1. EVALUATE COHESION NECESSARY FOR VERTICAL CUT

2. MAXIMUM DEPTH OF "0" COHESIVE MATERIALS IS 20 FT.

3. USE A FACTOR OF SAFETY OF 2

4. USE VISUAL & LABORATORY CLASSIFICATION BY GRANT ENVIRONMENTAL



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# ENVIRONMENTAL SOLUTIONS, INC.

By WB Date 1/9/96 Subject SURFICIAL SOIL IMPROVEMENT Sheet No. 3 of 5

Chkd. By MLL Date 1/12/96 Proj. No. 95-284

CALCULATION

USE STABILITY CHARTS DEVELOPED BY TERZAGHI, 1943 IN REFERENCE 1. THE CHART IS PRESENTED ON SHEET 4.

$$N_s = \frac{\gamma H_c (FS)}{c}$$

$N_s$ = STABILITY NUMBER
$H_c$ = CRITICAL HEIGHT
$c$ = COHESION
$\gamma$ = UNIT WEIGHT = 120 pcf
FS = FACTOR OF SAFETY = 2

FOR VERTICAL SLOPES ($\beta = 90°$) $N_s = 3.85$

$$3.85 = \frac{(120\text{pcf})\, H_c\, (2)}{c}$$

$H_c = 20$ FT $c = 1246$ psf

$H_c = 10$ FT $c = 623$ psf

FOR $\beta = 63°$ $N_s = 5.1$

$$5.1 = \frac{(120\text{pcf})\, H_c\, (2)}{c}$$

$H_c = 20$ FT $c = 940$ psf

$H_c = 10$ FT $c = 470$ psf

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$$t\hat{l}r = Wl_w \qquad 18.10$$

where $\hat{l}$ is the length of the arc. Hence, the shear stress required for equilibrium is

$$t = \frac{Wl_w}{r\hat{l}} \qquad 18.11$$

The factor of safety against shear failure along the arbitrarily selected circular arc is the ratio of the available strength $c$ to the required strength $t$. Thus

$$F = \frac{c}{t} = \frac{cr\hat{l}}{Wl_w} \qquad 18.12$$

The failure may not, of course, take place along the arbitrarily selected arc; it will occur along the arc for which the factor of safety is a minimum. Therefore, various locations for the center $O$ and various radii must be selected until the minimum factor of safety of the slope is found.

Both theory and trial calculations have been utilized to reduce the number of trials necessary to locate the critical center and radius. It has been established, for example, that if the slope angle $\beta$ is 53° or steeper, the critical circle passes through the toe of the slope. For flatter slopes the critical circle is generally tangent to the firm base and its center generally lies on a vertical line through the midpoint of the slope. However, if the firm stratum is at a very shallow depth below the toe of the slope, failure may take place along a circle, known as a slope circle, that intersects the slope above the toe.

Charts have been prepared (Taylor, 1937) for the solution of the problems illustrated in Fig. 18.12. A convenient form of these charts is shown in Fig. 18.13 (Terzaghi, 1943). For any slope angle and any depth $n_dH$ to the firm base, a dimensionless stability number

$$N_s = \frac{\gamma H_c}{c} \qquad 18.13$$

may be determined, and the value $H_c$ computed from eq. 18.13. The quantity $H_c$ is

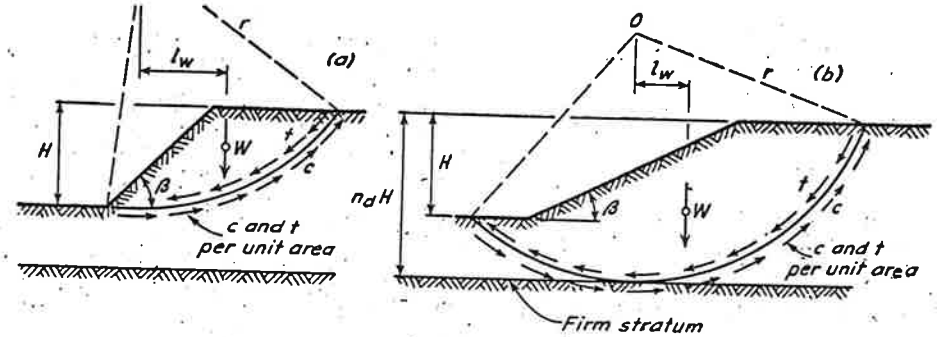


FIGURE 18.12. Position of failure surfaces. (a) Toe circle. (b) Base circle.

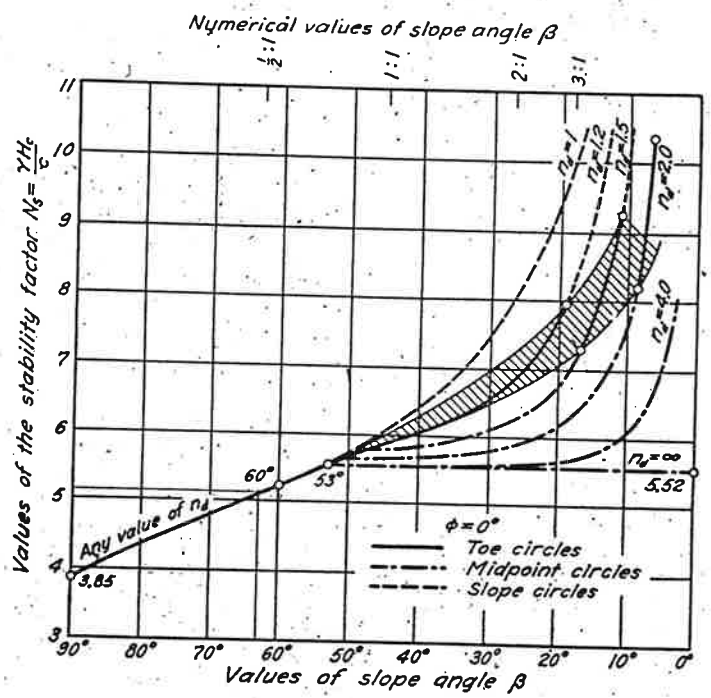


FIGURE 18.13. Relation for frictionless material between slope angle $\beta$ and stability factor $N_s$, for different values of depth factor $n_d$ (after Taylor, 1937).

the *critical height* of the slope; that is, the vertical height of the slope for a factor of safety of unity. The factor of safety of the proposed slope may be determined approximately from

$$F = \frac{H_c}{H} \qquad 18.14$$

where $H$ is the height of the slope.

If the deposit consists of strata of soft and medium clays, similar analyses may be developed. These are discussed in Chap. 21.

*Slopes in Stiff Clays.* The maximum height to which slopes can be excavated in stiff, satu-

# ENVIRONMENTAL SOLUTIONS, INC.

By WB Date 1/9/96 Subject SURFICIAL SOIL IMPROVEMENT Sheet No. 5 of 5

Chkd. By MH Date 1/12/96 Proj. No. 95-284

CONCLUSION

AMOUNT OF COHESION NEEDED TO STABILIZE SURFICIAL SOIL LAYER IS ABOUT 1000 psf.

IT IS UNLIKELY THAT AN UNCEMENTED DRY SILTY SAND WOULD HAVE A COHESION OF 1000psf. THEREFORE, SOME FORM OF SOIL IMPROVEMENT IS NEEDED TO INCREASE THE COHESION TO THE CALCULATED VALUE. TWO VIABLE METHODS INCLUDE

1.) MIX CEMENT WITH THE SOIL (ABOUT 3-6%)(1) TO CREATE A SOIL CEMENT

2.) CONSTRUCT A SOIL REINFORCED WALL WITH EITHER GEOGRIDS OR LAYERS WITHIN THE SOIL TO PROVIDE STRENGTH.

EITHER OF THE ABOVE METHODS WOULD PROVIDE A STABLE SLOPE AT A 0.5:1 (H:V).

(1) A LABORATORY TEST SHOULD BE CONDUCTED WITH THE ON-SITE SOILS TO EVALUATE THE APPROPRIATE SOIL/CEMENT MIXTURE.



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# C.4 SETTLEMENT

# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 1/29/96 Subject Waste Settlements Sheet No. 0 of 33

Chkd. By EC Date 2/14/96 Proj. No. 95-284

POST-CLOSURE WASTE SETTLEMENTS

U.S. ECOLOGY HAZARDOUS WASTE LANDFILL

BEATTY, NV SITE

CELL 12



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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 1/23/96 Subject Waste Settlement Sheet No. 1A of 33

Chkd. By EC Date 2/14/96 Proj. No. 95-284

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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 1/31/96 Subject Waste Settlement Sheet No. 1B of 33

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5.0 SETTLEMENT CALCULATIONS (continued)


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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 1/29/96 Subject Waste Settlement Sheet No. 2 of 33

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1.0 Purpose: Estimate post-closure settlement to verify the slope gradient of the soil cap is sufficient to provide positive drainage after closure of the facility.



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By JH Date 1/29/96 Subject Waste Settlement Sheet No. 3 of 33
Chkd. By EC Date 2/14/96 Proj. No. 95-284

## 2.0 APPROACH

Three major components of the settlement are evaluated:

- Settlement of the Final Soil Cap (2.1)
- Settlement of Waste (2.2)
- Settlement of Foundation Soils (2.3)

## 2.1 Settlement of The Final Soil Cap

Settlement within the final soil cap is assumed to be negligible. The following two requirements will be specified in order to achieve the assumed condition:

- Sand and gravel will be used for construction of the final soil cap. Oversize particles (with greatest dimension larger than 6 inches) will be surrounded



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By JH Date 1/29/96 Subject Waste Settlement Sheet No. 4 of 33
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by finer particles to prevent forming of voids within the final soil cap.

- Soils will be placed in relatively thin lifts (no more than 12 inches in thickness prior to compaction), and compacted with appropriate construction equipment to achieve a minimum 90 percent of the maximum dry density determined per ASTM D1557 Test Method. Care will be taken to assure the cover geosynthetic layers are not damaged during cap construction.



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## 2.2. Settlement of waste

Settlement of waste is assumed to consist of four major components:

- Static settlement of soil and soil-like waste (2.2.1),
- seismically induced settlement of soil and soil-like waste (2.2.2),
- Settlement caused by collapse of voids within containerized waste (2.2.3), and
- Drum Waste Compression after degradation and collapse of waste containers.
- Assumed waste composition is as follows (see Ref. 1):
  - 85% soil and soil-like waste,
  - 10% containerized waste, and
  - 5% non-compressible solid waste

# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 2/12/96 Subject Waste Settlement Sheet No. 6 of 33
Chkd. By EC Date 2/14/96 Proj. No. 95-284

2.2.1 Static Settlement of Soil and Soil-like Waste

As stated in Reference 1, soil and soil-like waste in the Beatty, NV facility generally consist of native soils (primarily sand and gravels, see Grant Engineering, Cross-section of North Wall, Reference 8), contaminated clay-like soils, and stabilized sludges.

The actual composition of soil and soil-like waste can not be predicted. For this calculation, it is assumed up to 30% of the soil and soil-like waste (include clays and stabilized sludges) are compressible on a long term basis. The primary soil type is assumed to be native material types (sands and gravels).

Post-closure settlement of compressible materials (clays and sludges) are assumed to be 10% of the thickness of compressible material. It is assumed in the US Ecology Report (Ref. 1) that the soil and soil-like waste occupies 85% of total waste fill volume. As a result, the overall post-closure settlement of compressible materials are:

at D' $\Rightarrow$ $0.1 \times 0.3 \times 0.85 \times 150' = 3.8'$

at C' $\Rightarrow$ $0.1 \times 0.3 \times 0.85 \times 96' = 2.4'$

General Equation for Settlement in clay is: $0.1 \times 0.3 \times 0.85 \times$ Depth of Fill $= \boxed{0.0255 \times \text{Depth of Fill}}$

The self-weight induced settlement for sands and gravels are assumed to be completed by the time of cap construction. However, additional settlement can be induced by weight of final cap. Although this could complete shortly after the cap construction, it is conservative to consider the cap-weight induced settlement in sands and gravels as part of the post-cap settlement.



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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 2/12/96 Subject Waste Settlement Sheet No. 7 of 33
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The US Ecology Report (Ref. 1) stated "The compaction efforts and subsequent traffic loads generally result in a dense, compact configuration for the waste fill, which greatly reduces the potential for subsequent settlement".

In order to provide conservative settlement estimates, it is assumed that some voids or lightly compacted spots can exist between large construction debris, drums, or large objects (such as transformers). Because of this assumption, the overall soil matrix will behave like a medium dense sand (which is the primary assumed soil type) when it is newly placed. As time goes on, finer materials will migrate into the voids and consolidation occurs. At the time of cap construction, it is assumed that the waste soil would be dense at the bottom of the landfill, and medium dense at the top of the landfill.

Long term (post-closure) settlement of sandy soils are estimated using the above stated assumed soil profile. Schmertmann's method (Winterkorn and Fang, 1975, Reference 2) is used for this calculation.

The overall static settlement of soil and soil-like waste is the sum of those in the compressible materials and in sandy materials.



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2.2.2 Seismically Induced settlement of soil and soil-like waste

Seismic settlement is estimated using the same assumed waste soil profile (see 2.2.1), and the Tokimatsu and Seed (1987, Reference 3) procedure.

2.2.3 Settlement Caused by Collapse of Voids Within Containerized Waste

This calculation is based on the assumption that containerized wastes are stored in DOT approved stainless steel 55-gallon drums, with up to 10 percent voids (see US Ecology, 1994, Reference 1).

Based on a telephone conversation with a supplier and a manufacturer of drum crushing equipment. (see Reference 4), a 40,000-lb force (hydraulically applied)



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can crush a 33-inch-high, 55-gallon stainless steel drum to 5-6 inches in height. The initiation of the collapse occur at approximately 20,000 pounds (also see Reference 9).

For a standard drum with a outer radius of 11.175", the critical crushing pressure is

$$P_{critical} = \frac{20{,}000}{\pi \left(\frac{11.175}{12}\right)^2} \doteqdot 7341 \text{ psf}$$

Assuming a soil/waste unit weight of 100 pcf, the critical crushing pressure corresponds to an overburden pressure of 73.4 feet (say approximately 70 feet) of soil/waste.

If drums placed within the top 70 feet of soil/waste do not corrode before the completion of the final cap, then it is



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conservative to assume that the void space within these drums will eventually become part of the post closure settlement.

Considering waste composition assumed by US Ecology (Reference 1), the waste fill consist of:

- 85% soil and soil-like waste,
- 10% containerized waste, and
- 5% non-compressible solid waste.

The estimated settlement caused by collapse of voids within the containerized waste due to drum corrosion in the top 70 feet of the waste fill is:

$$0.1 \times 70' \times 12'' \times 0.1 = 8.4 \text{ inches.}$$

(General equation is $0.12 \times D$, where D is thickness of waste fill).



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2.2.4 Drum Waste Compression after Degradation and Collapse of Waste Containers

It was shown in 2.2.3 that containerized waste placed at a depth of approximately 70 feet or deeper can collapse before the construction of final cap. The collapse of the containers will initiate the process of consolidation of the waste stored inside the container. Time rate of consolidation of these materials is unknown. However, one can conservatively assume that majority of the compression will occur after landfill closure. In reality, waste located at the bottom of the landfill will reach much higher degree of consolidation than those in the upper layers at the time of final cap construction.

Settlement due to compression of containerized waste can be estimated using a method proposed by



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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 1/30/96 Subject Waste Settlement Sheet No. 12 of 33

Chkd. By EC Date 2/14/96 Proj. No. 95-284

Murphy and Gilbert (EPA, 1985, Reference 5):

$$\delta = \frac{\gamma L^2}{2}\left(\frac{1+\nu}{1-\nu}\right)(1-2\nu)\left(\frac{E_i - E_f}{E_i E_f}\right)$$

where $\gamma = 100$ pcf (soil/waste density assumed for this site, this is used to calculate overburden pressure)

(maximum) $L = 10\%$ of $150'$

$= 15'$

$\nu = 0.35$ (The value of $\nu$ is Not sensitive to settlement calculation, See Ref. 5 for discussion on $\nu$.)

$E_i = 3.86 \times 10^7$ psf (elastic modulus of a stainless steel, 55-gal drum standing upright)

$E_f = 40{,}000$ psf (elastic modulus of a commercially available non-calcined absorbent earth compound)

$$\Rightarrow \delta = \frac{100 \times 15^2}{2} \times 2.08 \times 0.3 \times 0.000025$$

$= 0.176' \doteq 2.1$ inches

[or $\delta = 7.8 \times 10^{-4} L^2$ (in feet) where $L = 10\%$ of depth of fill in feet.]



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actual settlement due to compression of containerized waste will be lower that 2.1" because parts of the compression would have occurred before final cap placement.



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2.3 Settlement of Foundation Soils.

Settlement of foundation soil is estimated using Schmertmann's method (Ref 2) with the assumption that the foundation soil is very dense.

3.0 Design Profile

Based on a review of the site closure plan (Figure 1), a profile was selected for settlement analysis (profile A-A', Figure 2).

This profile was selected to evaluate the maximum total and differential settlements that can occur at the site.



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4.0 References


1. US Ecology, Inc, Landfill Report, August 1, 1994.

2. Winterkorn and Fang, 1975, Foundation Engineering Handbook, Ch. 4.

3. K. Tokimatsu and H.B. Seed, Evaluation of Settlement in Sands Due To Earthquake Shaking. Journal of Geotechnical Engineering, Vol. 113, No. 8, August, 1987.

4. ESI, 1996, Telephone Conversation with Distributor and Manufacturer of Drum Crusher.

5. W.L. Murphy and P.A. Gilbert, April, 1985, Settlement and Cover Subsidence of Hazardous Waste Landfills, Report No. EPA/600/2-85/035.

6. ESI, 1996, Exhibit C.1, Stability Calculation, U.S. Ecology, Beatty, NV.

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7. Seed & Idriss, Ground Motions and Soil Liquefaction During Earthquake, 1982.

8. Grant Engineering, Cross-section of North Wall, 5/25/94.

9. McMaster-Carr Catalog, Drum Crusher Data.




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5.0 Settlement Calculations

Settlements were calculated at selected points along section A-A'. Total settlement was plotted along the horizontal axis. Differential settlements were expressed in terms of relative rotation between the point of interest and the neighbouring point located to the left (i.e., the slope of the total settlement profile to the left of the point).

5.1 Static Settlement of Soil and Soil-like Waste

The approach of this calculation is described in 2.2.1. The weight of the final soil cap is considered as an areal load applied to the top of the waste fill.

The final soil cap is assumed to consist of 2 feet of compacted soil/waste overlain by a layer of FML and a geocomposite, and a 2-ft-thick compacted soil over the FML system. Total weight applied

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To the underlying waste fill is approximately

$4' \times 125\ pcf = 500\ psf.$

Total static settlement in Sands can be estimated as:

$$\delta_1 = C_1\ C_2\ \Delta p \left(\frac{I_z}{E}\right) \cdot \Delta Z$$

- for surface areal load $C_1 \doteqdot 1$
- $C_2 = 1 + 0.2 \log_{10}\left(\frac{t}{0.1}\right)$, let $t = 100$ years

  $= 1.6$
- $I_z = 0.6$ (maximum value), assumed to apply for all depths due to large areal load.
- $E = 2\,\bar{q}_c$ tsf. (assumed $q_c = 50$ tsf for a medium dense sand at ground surface, and 120 tsf for a dense sand at the bottom of the landfill)
- $\Delta Z = 0.85 \times D$.
  (85% of the waste fill is assumed to be soil and soil-like waste. D is the waste fill thickness)
- $\bar{q}_c$ is taken as the average cone tip resistance at the mid-level of the waste fill, $\bar{q}_c = (50+120)/2 = 85$ tsf.
- $\delta_1 = 1 \times 1.6 \times \frac{500}{2000}\ tsf \times \frac{0.6}{2 \times 85} \times (0.85\ D)$

  $= 0.0012\ D$
- From 2.2.1, settlement in clays $(\delta_2) = 0.0255\ D$



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Total static settlement in soil and soil-like waste is

$$\delta_{STATIC} = \delta_1 + \delta_2 = (0.0012 + 0.0255)\,D = 0.0267\,D \quad (ft)$$

where D is the depth of the waste fill.

| Point | Depth of Waste fill (ft) | SETTLEMENT OF SOIL AND SOIL-LIKE WASTE (INCHES) WITH 30% CLAYEY SOILS | WITH NO CLAYEY SOILS |
|---|---|---|---|
| A | 0 | 0 | 0 |
| B' | 9 | 2.9 | 0.1 |
| C' | 96 | 30.8 | 1.4 |
| D' | 150 | 48.1 | 2.2 |
| E' | 130 | 41.7 | 1.9 |
| F' | 100 | 32 | 1.4 |
| G' | 0 | 0 | 0 |



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5.2 Seismically Induced Settlement of Soil and Soil-like Waste

From 5.1 and 2.2.1, the assumed soil/waste profile has a cone tip resistance of 50 tsf (for a medium dense sand) at the top of the landfill, and 120 tsf (dense sand) at the bottom of the landfill. The assumed average cone tip resistance is $\frac{50+120}{2}$ (= 85 tsf) at the middle of the fill.

From Reference No. 2, Table 4.5, the conversion factor ($q_c/N$) between cone tip resistance ($q_c$) and standard penetration blow count (N) vary from 2.0 to 6 depending on soil type. Because of the uncertainty of the actual soil types, an average conversion factor of $q_c/N = 4$ is used in this calculation.



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The average equivalent SPT-N is approximately

$\Rightarrow q_c/4 = 85/4 = 21$ (say 20 blows/ft).

The seismically induced settlement is estimated at each point by the assumption that the average blow count applies at all depths and the settlement is calculated for one uniform layer.

| Point | Waste Fill Thickness (ft) | Vertical Stress at mid-depth (psf) (1),(2) | $G_{max}$ (KSF) (3) | $\gamma_{eff}$ | $\frac{G_{eff}}{G_{max}}$ (4) | Stress Reduction Factor $r_d$ | $\gamma_{eff}$ (5) | $\varepsilon_{c,M=7.5}$ (%) (6) | $\varepsilon_{c,M=8}$ (%) (7) | $2\times\varepsilon_{c,M=8}$ (%) | Seismic Settlement (inches) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| A | 0 | 0 | 0 | 0 | | 0 | 0 | 0 | 0 | 0 | 0 |
| B' | 9 | 950 | 1,673 | $1.54\times10^{-4}$ | | 0.992 | $2.7\times10^{-4}$ | 0.027 | 0.03 | 0.06 | 0.06 |
| C' | 96 | 5300 | 3,952 | $2.82\times10^{-4}$ | | 0.771 | $4.1\times10^{-4}$ | 0.04 | 0.045 | 0.09 | 1.04 |
| D' | 150 | 8000 | 4,856 | $2.47\times10^{-4}$ | | 0.55 | $3.8\times10^{-4}$ | 0.04 | 0.045 | 0.09 | 1.62 |
| E' | 130 | 7000 | 4,542 | $2.61\times10^{-4}$ | | 0.62 | $4\times10^{-4}$ | 0.04 | 0.045 | 0.09 | 1.4 |
| F' | 100 | 5500 | 4,026 | $2.79\times10^{-4}$ | | 0.747 | $4.1\times10^{-4}$ | 0.04 | 0.045 | 0.09 | 1.08 |
| G' | 0 | NA | 0 | 0 | | 0 | 0 | 0 | 0 | 0 | 0 |

(1) Vertical stress at mid-depth is calculated as:

$\frac{D}{2} \times 100\text{ pcf} + 4' \times 125\text{ pcf}$

D: Waste Fill Thickness; 100 pcf: unit weight of waste fill; 125 pcf: unit weight of compacted cover soil

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(2) calculated using a unit weight of 125 pcf for compacted fill, and 100 pcf for waste fill.

(3) $G_{max} = 20 N^{1/3} \sqrt{\sigma_o} \times 1000$ (psf)

$= 20 N^{1/3} \sqrt{\sigma_o}$ (ksf) where $\sigma_o$ is in psf.

$\because N \doteq 20$ blows/ft

$\Rightarrow G_{max} = 20 (20)^{1/3} \sqrt{\sigma_o}$, ksf

$\doteq \underline{\underline{54.29 \sqrt{\sigma_o}}}$, ksf (Note: $\sigma_o$ is in psf)

where $\sigma_o$ is vertical effective stress at mid-height of the waste fill.

(4) $$\gamma_{eff} \frac{G_{eff}}{G_{max}} = \frac{0.65 \cdot a_{max} \cdot \sigma_o \cdot r_d}{g \cdot G_{max}} = \frac{0.273 \cdot \sigma_o \cdot r_d}{G_{max}}$$

where $r_d$ is the stress reduction factor (see Ref. 7), and

$a_{max} = 0.42g$ (Reference No. 6)

(5) See Figure 11 of Reference No. 3.

(6) See Fig 13 of Reference No. 3

(7) See Table 4 of Ref 3.

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5.3 Settlement Caused By Collapse of Voids Within Containerized Waste

The approach of this calculation is described in 2.2.3. From 2.2.3, settlement = 0.12 · D (")

| Point | Thickness of Collapsible Waste Fill (1) | Settlement due to Collapse of Voids within Container (11) |
|---|---|---|
| A | 0 | 0 |
| B' | 9 | 1.08 |
| C' | 70 | 8.4 |
| D' | 70 | 8.4 |
| E' | 70 | 8.4 |
| F' | 70 | 8.4 |
| G' | 0 | 0 |

(1) If more than 70 feet, use 70 feet for calculation. From discussions in 2.2.3, containerized waste placed below a depth of about 70 feet would have collapsed before the construction of final cap.



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5.4 Settlement Due To Waste Compression after Degradation and Collapse of Waste Containers.

The approach of this calculation is presented in 2.2.4.

| Point | Waste Fill Thickness (ft) | L (1) (ft) | Settlement Due To Waste Compression (2) (ft) |
|---|---|---|---|
| A | 0 | 0 | 0 |
| B' | 9 | 0.9 | 0.008 |
| C' | 96 | 9.6 | 0.86 |
| D' | 150 | 15 | 2.11 |
| E' | 130 | 13 | 1.58 |
| F' | 100 | 10 | 0.94 |
| G' | 0 | 0 | 0 |

(1) L = 10% of the thickness of waste fill.

(2) $\delta$ (feet) $= 7.8 \times 10^{-4} L^2$, where L is in feet.



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5.5 Settlement of Foundation Soils.

Settlement at the foundation level is calculated using Schmertmann's equation (Ref 2):

$$\delta = c_1 \cdot c_2 \cdot \Delta p \cdot \frac{I_z}{E} \cdot \Delta z$$

$c_1 \doteq 1$ for areal load

$c_2 = 1 + 0.2 \log_{10}\left(\frac{t}{0.1}\right)$, $t$ in years.

for $t = 10$ years, $c_2 = 1.4$

for $t = 100$ years, $c_2 = 1.6$

$I_z = 0.6$ (max)

$E = 2 q_c = 2 \times (6 \times N)$

Let $N = 100$ blows per foot (for a very dense sand) $\Rightarrow E = 1200$ tsf.

This elastic modulus is not high considering the foundation soils are being reloaded.

$\Delta z$ = depth of influence below the foundation level, assumed to be 50 feet,



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upon completion of waste placement, $t \doteq 10$ years,

$$\delta = 1 \times 1.4 \times \Delta p \times \frac{0.6}{1200} \times 50' \times 12''$$

$$= 0.42\ \Delta p, \text{ inches.}$$

(where $\Delta p$ is expressed in tsf. $\Delta p$ is the net stress increase at the foundation level, see page 27/33 for detailed calcs.)

at $t = 100$ years

$$\delta = 1 \times 1.6 \times \Delta p \times \frac{0.6}{1200} \times 50' \times 12''$$

$$= 0.48\ \Delta p$$

The potential increase of settlement over the first 90 years is about 14%, which is quite negligible considering the low differential settlements (see page 27/33 for details).



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Total settlement at $t = 10$ years is calculated as follows:

| Point | Waste Fill Thickness (ft) | Original Foundation Stress (tsf) (1) | $\Delta P$ Net Foundation Stress (tsf) (2) | $\delta$ Settlement (in) (3) |
|---|---|---|---|---|
| D' | 150 | 4.69 | 2.81 | 1.18 |
| E' | 130 | 4.69 | 1.81 | 0.76 |
| F' | 100 | 3.13 | 1.87 | 0.79 |
| G' | 0 | 0. | 2.38 | 1 |

(1) Original Foundation Stress is calculated as:

$$75' \times 125 \text{ pcf} / 2000 = 4.69 \text{ tsf}$$

(2) Net Foundation Stress is calculated as:

$$\Delta P = \frac{D \times 100 \text{ pcf}}{2000} - \text{original foundation stress}$$

where D = depth of waste fill.

(3) $\delta$ (inches) $= 0.42 \cdot \Delta P$

$\Rightarrow$ Maximum differential settlement of foundation soil due to weight of fill is on the order of 0.4 inches which is quite negligible



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comparing to other sources of settlements. Also, the settlements of foundation soil will essentially be completed at the end of waste placement. As a result, settlement of foundation soil is not considered as part of post-closure settlement.



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Chkd. By EC Date 2/14/96 5.6 Summary of Settlement Proj. No. 95-284

| POINT | STATIC SETTLEMENT (1) WITH 30% Clayey Soils | STATIC SETTLEMENT (1) No Clayey Soils | Seismic Settlement (1) | Settlement of Collapsed Voids in Drains (1) | Settlement Due To Waste Compression (1) | TOTAL POST-CLOSURE SETTLEMENT (1) WITH 30% Clayey Soils | TOTAL POST-CLOSURE SETTLEMENT (1) No Clayey Soils | Differential Settlement (1) w/30% clayey soils | Differential Settlement (1) No Clayey Soils | Rotation (radians) (2) w/30% clayey soils | Rotation (radians) (2) No Clayey Soils |
|---|---|---|---|---|---|---|---|---|---|---|---|
| A | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| B' | 2.9 | 0.1 | 0.06 | 1.08 | 0.01 | 4.05 | 1.25 | 4.05 | 1.25 | 0.0135 | 0.0042 |
| C' | 30.8 | 1.4 | 1.04 | 8.4 | 0.86 | 41.1 | 11.7 | 37.05 | 10.45 | 0.082 | 0.0232 |
| D'' (3) | | | | | | Exp. 23 | 14.33 | 19.13 | 2.63 | 0.0071 | 0.001 |
| D' | 48.1 | 2.2 | 1.62 | 8.4 | 2.11 | 60.23 | 14.33 | 0 | 0 | 0 | 0 |
| E' | 41.7 | 1.9 | 1.4 | 8.4 | 1.58 | 53.08 | 13.28 | -7.15 | -1.05 | -0.0095 | -0.0014 |
| F' | 32 | 1.4 | 1.08 | 8.4 | 0.94 | 42.42 | 11.82 | -10.66 | -1.46 | -0.071 | 0.0097 |
| G' | 0 | 0 | 0 | 0 | 0 | 0 | 0 | -42.42 | -11.82 | -0.1414 | -0.0394 |

NOTES:

(1) Comparing to the left neighbouring point.

(2) Slope of the total settlement profile to the left neighbouring point. The total post-closure settlement profile is presented on Figure 2, page 33.

(3) Same total settlement as D'.



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6.0 CONCLUSIONS

1. Maximum calculated total settlement at point D' is on the order of 60 inches.

2. Maximum differential settlement expressed in terms of rotation between neighbouring points is on the order of 7-14 % of slope reduction assuming waste containing 30% clayey materials, and 1-4 % assuming no clayey materials.

   Max. differential settlement occurs between B'-C', E'-F', and F'-G'.

3. Overall slope reduction between D' and G' is about 5 percent.

4. This calculation provides reasonable estimates of post-closure settlements based on the assumptions made.



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Actual settlement will depend on the composition of as-received waste stream which can not be predicted at this time. This calculation has conservatively assumed that up to 30% of the soil and soil-like waste consists of clay or clay-like sludge (stabilized) which can result up to 10% strain on a long term basis. Actual settlement can be significantly less if there are less clays or clay-like sludge, or if the clays are well-mixed with native sands and gravels, and compacted with heavy equipment during placement.

Calculations performed without considering clays (i.e. assume there is no clay or clay-like materials in the waste fill) have resulted a maximum estimated settlement of about 14 inches at D', which is about the same as the maximum post-closure settlement estimated by US Ecology (Ref. 1) using an independent approach of analysis.



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Figures



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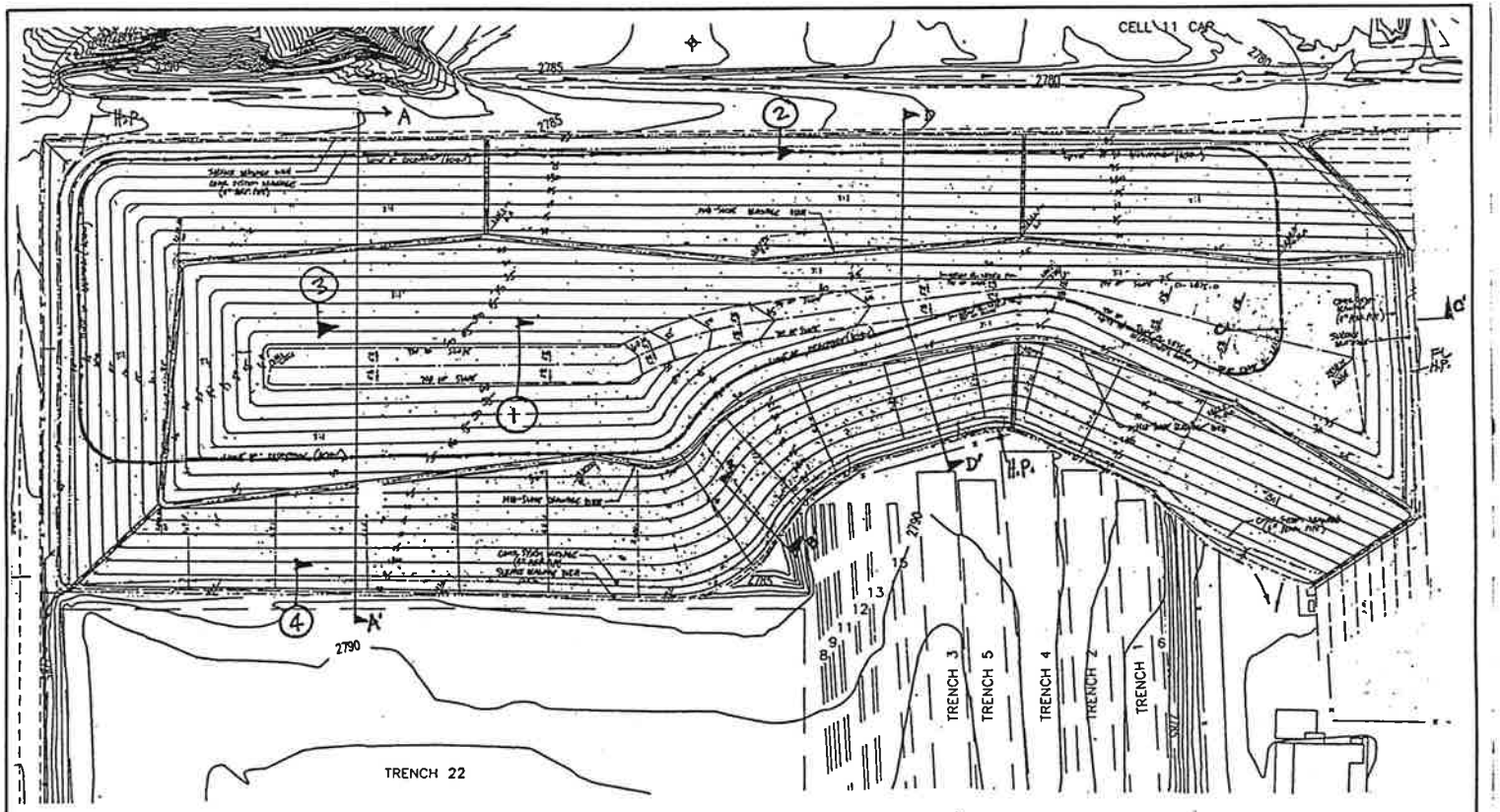



NOTES: CROSS SECTIONS A-A', B-B', C-C', D-D' — SEE FIGURES 3&4
DETAILS ①,②,③,④ — SEE FIGURE 5

0 50 100 200 FT
SCALE

FIGURE 1
CLOSURE PLAN
HAZARDOUS WASTE MANAGEMENT FACILITY
BEATTY • NEVADA
ENVIRONMENTAL SOLUTIONS, INC.

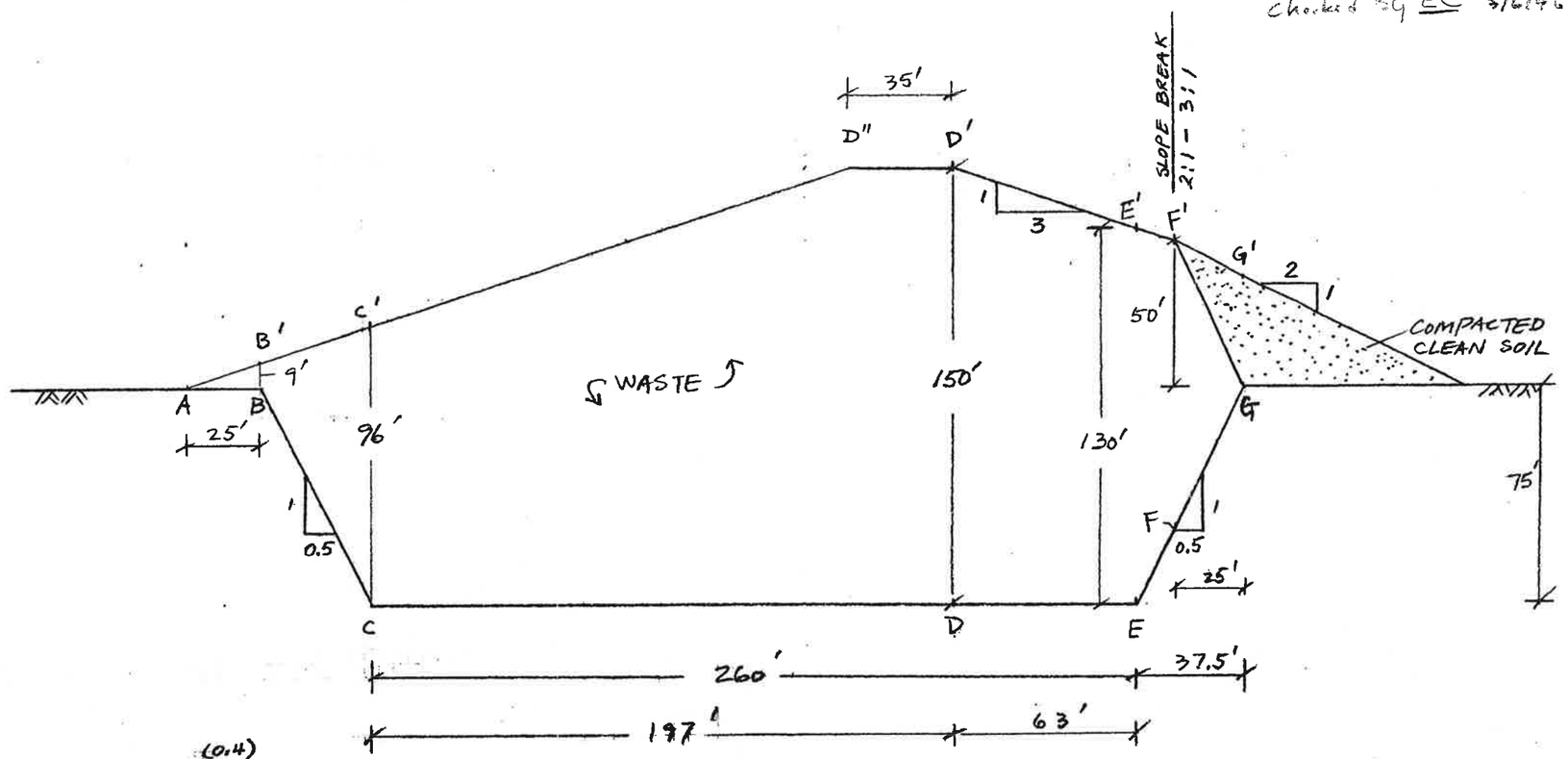
FIGURE 2 PROFILE A–A′

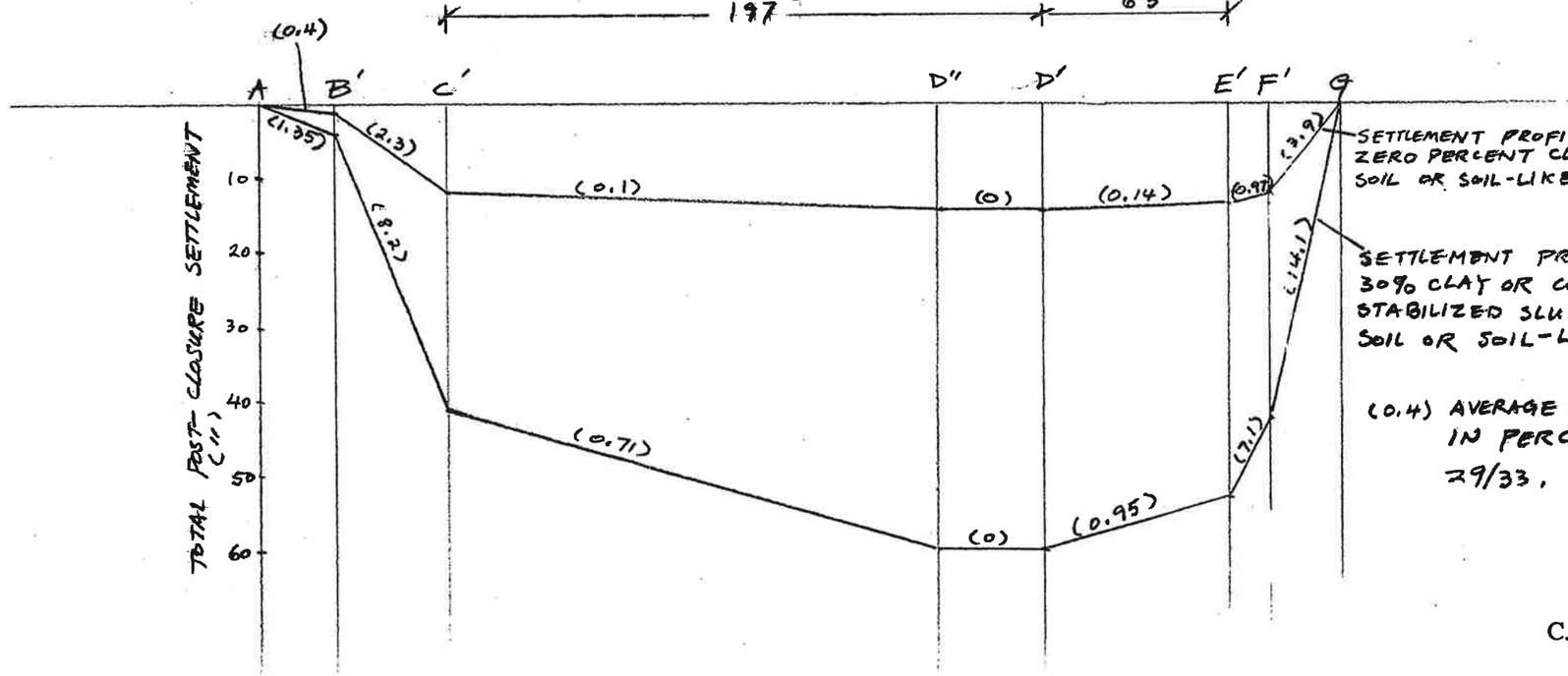
APPROXIMATE SCALE: 1″ = 50′

33/33

Prepared By JH 3/6/96

Checked By EC 3/6/96




(0.4) AVERAGE SLOPE REDUCTION IN PERCENT, SEE PAGE 29/33.

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Reference No. 3



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$L, L'$ = foundation length, $L \geqq B$;
$m$ = ratio $L'/B'$;
$N$ = standard penetration test blow count;
$n$ = ratio $H/B'$;
$q_c$ = cone penetration resistance;
$\Delta q$ = foundation-to-soil contact pressure (footing pressure);
$R, r$ = radius of round foundation;
$\epsilon_z$ = vertical strain;
$\mu$ = Poisson's ratio; and
$\sigma_i$ = stress, $i = 3$ coordinate axes $(x, y, z)$.



C.4-38

# Evaluation of Settlements in Sands due to Earthquake Shaking

Ref. 3

By Kohji Tokimatsu, A. M. ASCE, and H. Bolton Seed, Hon. M. ASCE

**Abstract:** Simplified methods of analysis are proposed for estimating probable settlements in either saturated or unsaturated sand deposits subjected to earthquake shaking. It is suggested that the primary factors controlling induced settlement are the cyclic stress ratio and maximum shear strain induced in saturated sands by the earthquake shaking and the cyclic strains induced in dry or partially saturated sands, together with the SPT N-value of the sand and the magnitude of the earthquake. Charts are presented for estimating settlements using these parameters, and the results are shown to compare favorably with settlements observed at six sites for which good data on settlements have been observed.

## Introduction

It has long been recognized that sands tend to settle and densify when they are subjected to earthquake shaking. If the sand is saturated and there is no possibility for drainage, so that constant volume conditions are maintained, the primary effect of the shaking is the generation of excess pore water pressures. Settlement then occurs as the excess pore pressures dissipate.

Depending on the characteristics of the soil and the length of the drainage path, the time required for all settlement to develop can vary considerably, from almost immediately to about a day. In dry sands, on the other hand, the settlement occurs during the earthquake shaking under conditions of constant effective vertical stress. In both cases, however, the final result of the shaking and the application of cyclic loading is settlement of the sand.

Although methods of evaluating the two types of settlement have been proposed by Lee and Albaisa (1974) and Silver and Seed (1971), there seems to be no recent work on the prediction of settlements in sands that incorporates findings since about 1975. Since even small settlements may sometimes have a significant effect on the performance of structures during earthquakes, it is desirable to review recent findings and field observations relating to the settlements of sands that may be induced by earthquake shaking.

The object of the present work, therefore, is to review previous studies, to summarize the available information concerning the settlements of sands during earthquakes, and to propose simplified methods of analysis to predict earthquake-induced settlement in both saturated and nonsaturated sands.

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[2]Prof. of Civ. Engrg., Univ. of California, Berkeley, CA 94720.

Note.—Discussion open until January 1, 1988. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on June 27, 1986. This paper is part of the *Journal of Geotechnical Engineering*, Vol. 113, No. 8, August, 1987. ©ASCE, ISSN 0733-9410/87/0008-0861/$01.00. Paper No. 21706.

On the basis of laboratory cyclic loading test data, Lee and Albaisa (1974) studied the settlement of sands resulting from the dissipation of excess pore water pressures developed during cyclic loading. They concluded that

> the amount of reconsolidation volumetric strain for non-liquefaction conditions increases with increasing grain size of soil, decreasing relative density, and increasing excess pore pressure generated during the undrained cyclic loading, but is almost independent of how this excess pore pressure was generated. No conclusions were drawn concerning the possible effect of induced strains on the postliquefaction volumetric strain.

Recently, Tatsuoka, et al. (1984) studied the volumetric strain after initial liquefaction (pore pressure ratio = 100%) and found that the amount of settlement can be significantly influenced by the maximum shear strain developed in the soil as well as the soil density, but that it is relatively insensitive to effective overburden pressure. Thus, the maximum shear strain is an important factor influencing the probable settlements after liquefaction has developed, even though there are no significant changes in the maximum pore pressure beyond this stage.

The relationships between relative density and volumetric strain after initial liquefaction observed in the studies by Tatsuoka, et al. (1984) are summarized in Fig. 1 in terms of the maximum shear strain occurring in the tests. Each data point shown is the average of several tests, which show very little scatter.

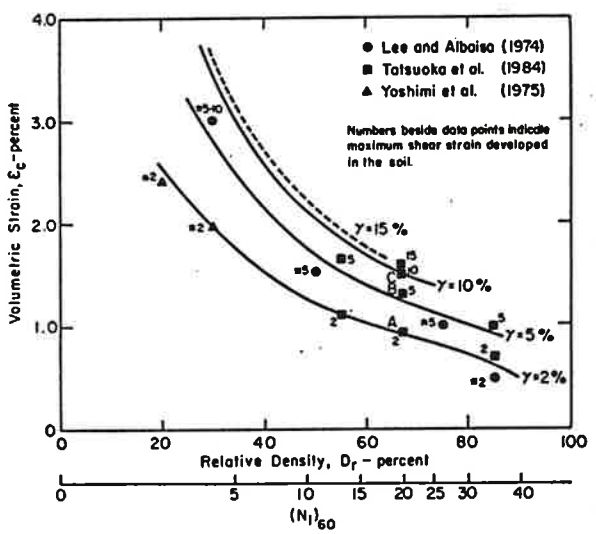



**FIG. 1.—Relationship between Volumetric Strain, Induced Strain, and Relative Density for Sands**



Although there were no direct measurements of the in shear strains in the studies by Lee and Albaisa, a review off their results permits estimates of the strain level likely to have developed in their tests and four data points. This estimated strain level represents reasonable average conditions based on about 30 tests performed in their investigation (see Fig. 1). An additional data point based on test data reported by Yoshimi, et al. (1975) is also shown. Although different sands were used in the three investigations for which data are plotted, the results show good general consistency. Thus, representative average curves incorporating postliquefaction volumetric strain, maximum induced shear strain, and relative density (based on all three studies) are shown in Fig. 1. Because of the sparsity of the data, the relationships shown may be in error by ±25% or so, but they provide the basis for expressing a preliminary relationship between volumetric strain following liquefaction and other parameters. It may be seen that the volumetric strain increases significantly with decreasing relative density and increasing induced strain in a soil.

The liquefaction resistance of a sand, usually expressed in terms of the cyclic stress ratio, $\tau_{av}/\sigma_o'$, where $\tau_{av}$ = the average induced cyclic stress, and $\sigma_o'$ = the initial effective overburden pressure, is strongly dependent on such factors as method of sample preparation and stress history effects. However, these factors are likely to become less significant when dealing with the volumetric strain after liquefaction due to the relatively large strains associated with this phenomenon. This aspect of soil behavior may well be influenced primarily by only the relative density and the maximum shear strain developed in a sand. Thus, it is not unreasonable to expect that the relationship shown in Fig. 1 can be used as an approximate basis for estimating the settlement of other saturated sands after liquefaction, provided that the soil density in the field can be estimated with a reasonable degree of accuracy.

The shear strain developed in situ during earthquakes may be estimated from Fig. 2 (after Seed, et al. 1984a), which shows tentative values of the shear strain potential for any combination of cyclic shear stress ratio and normalized SPT N-value for a magnitude 7.5 earthquake. A similar chart was presented by Tokimatsu and Yoshimi (1983) for N-values measured according to Japanese standards, where the effective energy delivered by the hammer is generally higher than that recorded using United States standards. A recent study by Tokimatsu and Yoshimi (1984) showed that the maximum strain potential is reasonably consistent with the limiting strain potentials proposed by Seed (1979) when the difference in energy ratios in SPT determinations in the two countries is taken into account.

The shear stress ratio in Fig. 2 is given by

$$\frac{\tau_{av}}{\sigma_o'} = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma_o'} \cdot r_d \quad \text{(1)}$$

in which $\tau_{av}$ = average cyclic shear stress induced by the earthquake shaking; $a_{max}$ = maximum horizontal acceleration at the ground surface; $\sigma_o$ = total overburden pressure at the depth considered; $\sigma_o'$ = effective overburden pressure; and $r_d$ = stress reduction factor, which varies from

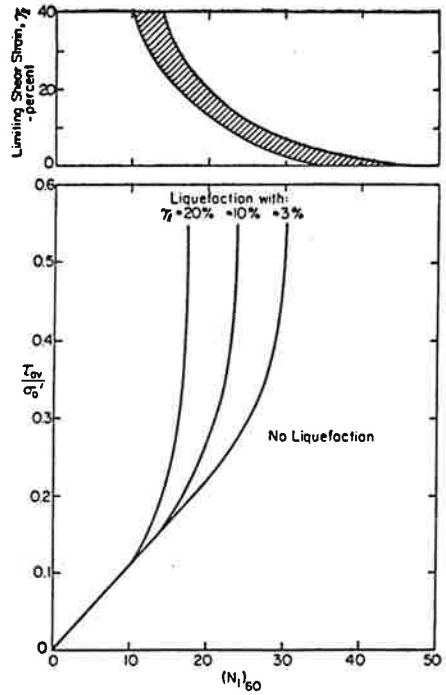



**FIG. 2.—Tentative Relationship between Cyclic Stress Ratio Causing Liquefaction, SPT N-Value, and Limiting Shear Strain (after Seed, et al. 1984)**

a value of 1 at the ground surface to a value of about 0.9 at a depth of 30 ft. The normalized SPT N-value (Kovacs, et al. 1984; Seed, et al. 1984a) may be determined by

$$(N_1)_{60} = C_{ER} \cdot C_N \cdot N \quad (2)$$

in which $(N_1)_{60}$ = SPT N-value normalized to an effective overburden pressure of 1 tsf and to an effective energy delivered to the drill rods equal to 60% of the theoretical free-fall energy; $N$ = measured SPT N-value; $C_N$ = a correction factor depending on the effective overburden pressure at the depth of the SPT determination; and $C_{ER}$ = a correction factor for the energy developed in the SPT determinations.

The results presented in Figs. 1 and 2 may be combined with the aid of a relationship between relative density and SPT $N_1$-values. It has been found in Japanese engineering practices (e.g., Tatsuoka, et al. 1978; Tokimatsu and Yoshimi 1983) that a good average relationship between these parameters is provided by the expression proposed by Meyerhof (1957), viz:

$$D_r = 21\sqrt{\frac{N_J}{\sigma_o' + 0.7}} \quad (3)$$



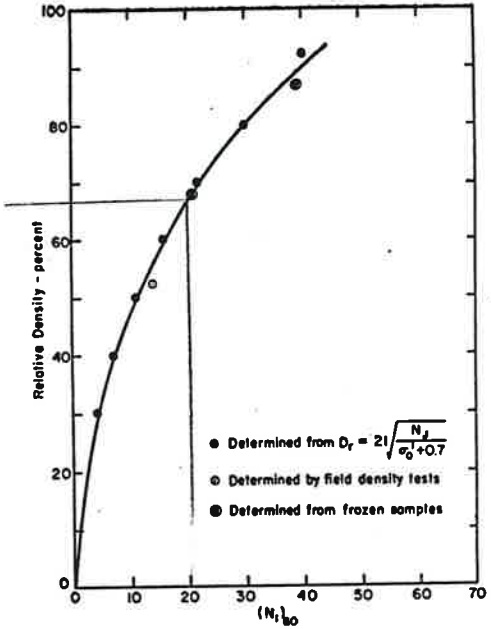



**FIG. 3.—Relationship between Relative Density and $(N_1)_{60}$**

in which $N_J$ = SPT N-value measured by Japanese standards; and $\sigma_o'$ = effective overburden pressure, in ksc. However, Japanese standards for the measurement of N-values involve the use of a higher energy ratio than that required by United States standards, in addition to other minor differences (Seed, et al. 1984a). Thus, the writers have chosen to convert the results expressed by Eq. 3 to corresponding values in terms of $N_{60}$, where $N_{60}$ = the SPT N-value determined by a method that provides 60% of the theoretical free-fall energy to the drill rods, as well as other standards of practice. Points expressing this relationship are plotted in Fig. 3.

Fig. 3 also shows several points determined by recent studies on natural deposits of sand where the relative density was measured using undisturbed samples obtained by in situ freezing (Yoshimi 1984); Yoshimi, et al. 1984; Katayama, et al. 1984) and a data point from an old deposit where the relative density was determined on the basis of field density measurements. It may be seen that all of the results shown are in good agreement, and the relationship shown in this figure therefore has been adopted for use in the present study. It should be noted that this relationship is somewhat different from that provided by laboratory studies since it takes into account the effect of aging on the STP N-value of sands (Mitchell 1984) and is thus likely to be indicative of the relationship that might be expected for natural or older deposits.

The relationship between relative density and $(N_1)_{60}$ shown in Fig. 3 is plotted along the abscissa in Fig. 1, thereby providing an approximate

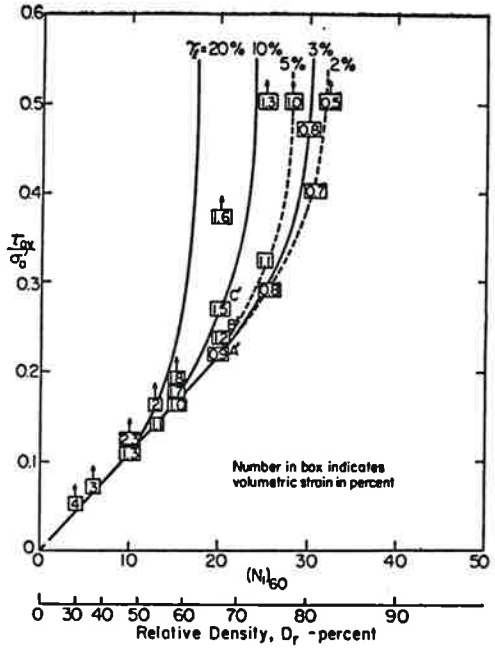
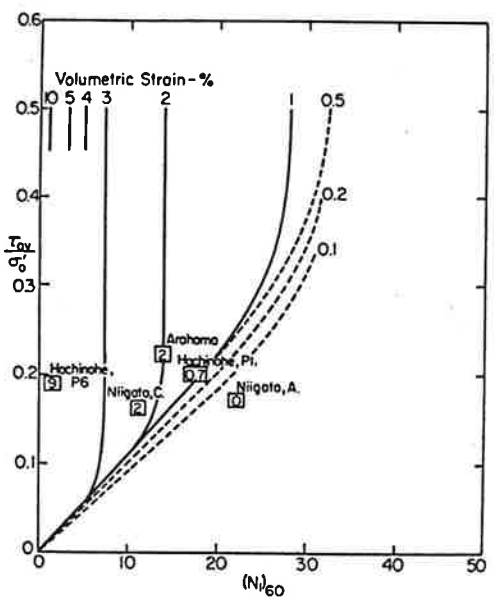



FIG. 4.—Relationship between Cyclic Stress Ratio, Volumetric Strain, and $(N_1)_{60}$




STEPS
1. Calc $(N_1)_{60}$ for equivalent clean sand
2. Calc. $\tau_{avg}/\sigma_o'$ from Eq. (1)
3. Read $\varepsilon_v$ from Fig. 5
4. Calc. $\delta$
$\delta = \varepsilon_v \cdot$ thickness

FIG. 5.—Proposed Tentative Relationship between Cyclic Stress Ratio, $(N_1)_{60}$, and Volumetric Strain for Saturated Clean Sands



relationship between volumetric strain after liquefaction and $(N_1)_{60}$-values expressed in terms of $(N_1)_{60}$.

The volumetric strains for different combinations of $(N_1)_{60}$-values and shear strain can be read from Fig. 1 and plotted at the corresponding points on Fig. 2, as shown in Fig. 4. For example, the points labeled A, B, and C in Fig. 1, which correspond to $(N_1)_{60}$ = 20 and shear strains of 2, 5, and 10%, would correspond to volumetric strains of 0.9, 1.2, and 1.5% and would be plotted as point A′, B′, and C′ in Fig. 4. It is now possible, based on the values of volumetric strains after liquefaction shown in Fig. 4, to draw equivolumetric shear strain lines on the stress ratio versus SPT $(N_1)_{60}$ chart, as shown by the solid lines in Fig. 5. It should be noted that the resulting volumetric strains after liquefaction may be as high as 2–3% for loose to medium sands and even higher for very loose sands.

## Volumetric Strain after Incomplete Liquefaction

Even though liquefaction may not occur, some pore pressure may be generated in sand deposits by earthquake shaking, and the dissipation of this pore pressure may result in small amounts of settlement. This condition corresponds to the zone below the boundary line for liquefaction shown in Fig. 2 and below the solid line marked volumetric strain = 1.0% in Fig. 5. The pore pressure generated under such conditions may be expressed in terms of the normalized stress ratio, i.e., the ratio of the actual shear stress ratio to the stress ratio just causing liquefaction. In Fig. 6, the relationship between the pore pressure ratio and the normalized stress ratio generally falls within the shaded area and for most sands, may be represented by the broken line (Tokimatsu and Yoshimi 1983).

Lee and Albaisa (1974) plotted volumetric strain as a function of induced pore pressure ratio (see Fig. 7) and concluded that, for pore pressure ratios less than about 0.6, the relationship between volumetric strain and peak pore pressure ratio could be represented for practical purposes by the dashed line in Fig. 7 for sands with relative densities in the range

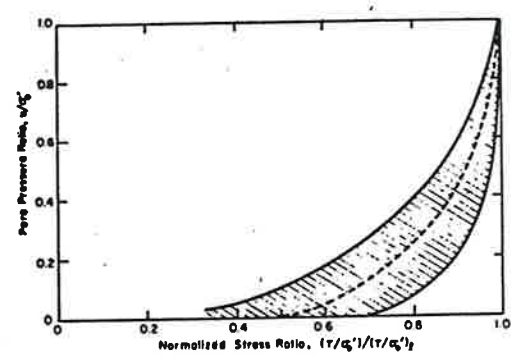



FIG. 6.—Relationship between Induced Pore Pressure Ratio and Normalized Stress Ratio for Clean Sands

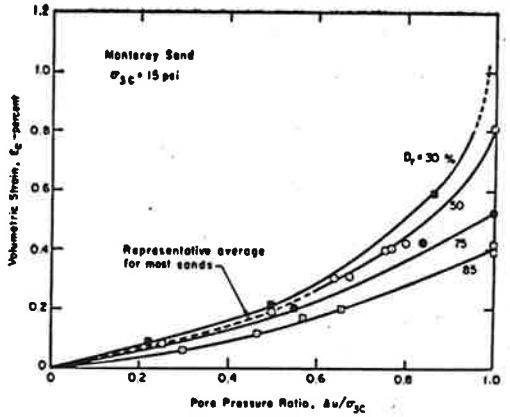



**FIG. 7.—Relationship between Volumetric Strain and Induced Pore Pressure Ratio (after Lee and Albaisa 1974)**

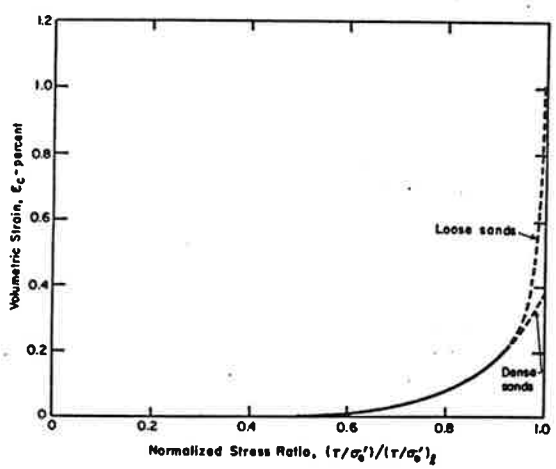



**FIG. 8.—Relationship between Volumetric Strain and Normalized Pore Pressure Ratio for Saturated Clean Sands**

of 30–80%, regardless of the confining pressure. Thus, by combining the average results shown in Figs. 6 and 7, Fig. 8 can readily be prepared to determine a relationship between volumetric strain and normalized stress ratio. It may be seen that even for a condition where the stress ratio is about 0.8 times that required to cause liquefaction (i.e., factor of safety against liquefaction is about 1.25), the resulting volumetric strain is only about 0.1%. The relationship shown in Fig. 8 is also represented by the dashed lines in Fig. 5.

It may also be noted that, if the normalized stress ratio is less than 0.7, the induced pore pressure ratio is likely to be less than 0.1 according



**TABLE 1.—Scaling Factors for Effect of Earthquake Magnitude ( ic Stress Ratio Causing Liquefaction**

| Earthquake magnitude, $M$ (1) | Number of representative cycles at $0.65_{max}$ (2) | Scaling factor for stress ratio, $r_m$ (3) |
|---|---|---|
| 8-1/2 | 26 | 0.89 |
| 7-1/2 | 15 | 1.0 |
| 6-3/4 | 10 | 1.13 |
| 6 | 5 | 1.32 |
| 5-1/4 | 2–3 | 1.5 |

to Fig. 6, and hence the effect of pore pressure generation on settlement is likely to be insignificant for most structures.

## Effects of Earthquake Magnitude on Settlement in Saturated Sands

The chart in Fig. 5 can be extended to other magnitude events by noting that the main difference between different magnitude earthquakes is the difference in number of cycles of stress they produce (e.g., Seed, et al. 1983). Table 1 summarizes the relative values of stress ratio required to cause liquefaction for earthquakes of different magnitudes to the stress ratio required to cause liquefaction for a $M = 7.5$ event, together with the approximate numbers of cycles induced by the earthquakes. Thus, by multiplying the values of the ordinate for each equivolumetric strain line in Fig. 5 by the scaling factors shown in Col. 3 of Table 1, volumetric strain charts can be obtained for earthquakes with different magnitudes. Alternatively, Fig. 5 can be used for any magnitude event simply by modifying the values of the cyclic stress ratios for those earthquakes into equivalent values for $M = 7.5$ earthquakes using the relationship

$$\left(\frac{\tau_{av}}{\sigma_o'}\right)_{M=7.5} = \left(\frac{\tau_{av}}{\sigma_o'}\right)_{M=M} \times \frac{1}{r_m} \quad (4)$$

where $r_m$ has the values shown in Col. 3 of Table 1.

## Observed Settlements in Saturated Sands

In order to compare the values of volumetric strain shown on the chart in Fig. 5 with field behavior, information concerning volumetric strains for several field cases of seismically induced settlement of saturated sands are summarized in Table 2, and the resulting volumetric strains are plotted on the chart in Fig. 9. Although the number of field cases for which data are available is quite limited, the field evidence is generally consistent with the lines drawn in the figure.

Typical examples of the computation of settlements for two cases reported by Ohsaki (1970), where settlements occurred in saturated sand during the Tokachi-oki earthquake of 1968, are shown in Table 3. The sites are designated P6 and P1, respectively. For site P6, the sand de-

TABLE 2.—Field Observations of Earthquake-Induced Settlements in Saturated Sands

| Earthquake (1) | Year (2) | Magnitude (3) | Site (4) | Maximum acceleration (5) | Thickness of layer causing major settlement (ft) (6) | Observed settlement (in.) (7) | Volumetric strain (%) (8) | Average stress ratio (9) | Average SPT $(N_1)_{60}$ (10) | Fines content (%) (11) | Reference (12) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Tokachioki | 1968 | 7.9 | Hachinohe P6 | 0.2 | 16 | 15–20 | 9 | 0.19 | 1.4 | 5 | Ohsaki (1970) |
| Tokachioki | 1968 | 7.9 | Hachinohe P1 | 0.23 | 7 | 0.5–0.7 | 0.7 | 0.2 | 17.5 | 5 | Ohsaki (1970) |
| Niigata | 1964 | 7.5 | Niigata C | 0.16 | 30 | 8 | 2 | 0.16 | 11 | 2 | Building Research Institute (1965) |
| Niigata | 1964 | 7.5 | Niigata A | 0.18 | 30 | 0 | 0 | 0.17 | 22 | 2 | |
| Miyagiken Oki | 1968 | 7.4 | Arahama | 0.2 | 30 | 8 | 2 | 0.22 | 14 | 0 | Tohno, et al. (1981) |

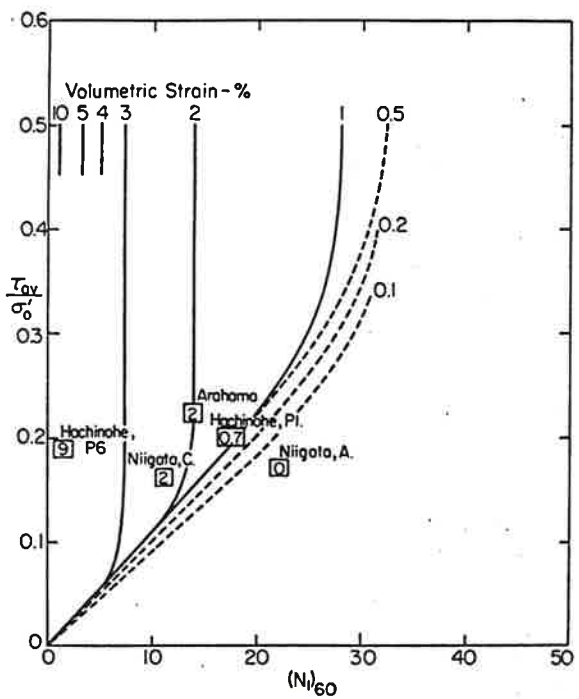



FIG. 9.—Comparison of Proposed Chart for Determination of Volumetric Strain with Field Performance of Saturated Sands



TABLE 3.—Computation of Settlement for Saturated Sand: Comp of Proposed Method with Field Observations Made by Ohsaki (1970)

| Site (1) | Layer number (2) | Thickness (ft) (3) | $N$ (4) | $C_{ER}$ (5) | $\sigma_o$ (psf) (6) | $\sigma_o'$ (psf) (7) | $C_N$ (8) | $(N_1)_{60}$ (9) | $(\tau/\sigma_o')_{ave}$ (10) | Volumetric strain (%) (11) | Settlement (in.) (12) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| P6[a] | 1 | 4 | 1 | 0.82[c] | 240 | 240 | | | | | |
| | 2 | 3.3 | 0.5 | 0.82[c] | 678 | 575 | 1.7 | 0.7 | 0.155 | 10 | 4 |
| | 3 | 3.3 | 0.5 | 0.82[c] | 1,074 | 764 | 1.57 | 0.6 | 0.185 | 10 | 4 |
| | 4 | 3.3 | 0.5 | 1.09 | 1,470 | 954 | 1.44 | 0.8 | 0.20 | 10 | 4 |
| | 5 | 3.3 | 2 | 1.09 | 1,866 | 1,144 | 1.34 | 2.9 | 0.21 | 5.5 | 2.2 |
| | 6 | 3.3 | 5 | 1.09 | 2,262 | 1,334 | 1.24 | 6.8 | 0.215 | 3.2 | 1.3 |
| | 7 | 3.3 | 23 | 1.21 | 2,658 | 1,523 | 1.16 | 32 | 0.22 | 0 | |
| | 8 | 3.3 | 33 | 1.21 | 3,054 | 1,713 | 1.09 | 44 | 0.225 | 0 | |
| | 9 | 3.3 | 28 | 1.21 | 3,450 | 1,903 | 1.03 | 35 | 0.225 | 0 | |
| | 10 | 3.3 | 33 | 1.21 | 3,846 | 2,093 | 0.97 | 39 | 0.225 | 0 | |
| Total | | | | | | | | | | | 15.5 |
| P1[b] | 1 | 3.3 | 12 | 0.82[c] | 198 | 198 | | | | | |
| | 2 | 3.3 | 12 | 0.82[c] | 594 | 490 | 1.8 | 18 | 0.185 | 0.3 | 0.1 |
| | 3 | 3.3 | 13 | 0.82[c] | 990 | 681 | 1.63 | 17 | 0.22 | 1.5 | 0.6 |
| | 4 | 3.3 | 20 | 1.21 | 1,386 | 870 | 1.50 | 36 | 0.24 | 0 | |
| | 5 | 3.3 | 36 | 1.21 | 1,782 | 1,060 | 1.38 | 60 | 0.24 | 0 | |
| Total | | | | | | | | | | | 0.7 |

[a]Water table = 4 ft; estimated maximum acceleration ≈ 0.20 g; actual settlement ≈ 20 in. (max).
[b]Water table = 3.3 ft; estimated maximum acceleration ≈ 0.23 g; actual settlement ≈ 0.5–0.7 in.
[c]Corrected by 0.75.

posit was extremely loose to a depth of about 20 ft and the observed maximum settlement was about 20 in. Based on values of the volumetric strain in different depth zones taken from the chart in Fig. 5, the computed settlement was 15.5 in., as shown in footnote a of Table 3. For site P1, the sand was medium dense and the observed settlement was only about 0.6 in. The computed value, as shown in footnote b of Table 3, was about 0.7 in. In both cases, the computed settlements are in good agreement with the reported values. This relatively good agreement between the measured and computed values indicates the possibility of using the chart as a basis for estimating probable settlements of saturated sands due to earthquake shaking.

## Review of Previous Studies on Dry Sands

Silver and Seed (1971) have shown that the settlement of dry sands due to cyclic loading is a function of: (1) The relative density of the soil; (2) the magnitude of the cyclic shear strain; and (3) the number of strain cycles. It was also found that, for a given density and number of cycles, settlement is not significantly affected by the value of the vertical stress and depends only on the shear strain amplitude in the soil. Based on these results, Seed and Silver (1972) suggested a procedure for estimating the probable settlement of dry sand that involves a response analysis for the sand deposit to determine the induced shear strains developed at different depths in the soil. The analysis was modified by Pyke, et al. (1975) to allow for multidirectional shaking effects, which were found to have an important influence on the magnitude of settlements.

1. ıod outlined later is a simplified version of the Seed and Silver m..ıod of analysis and offers the advantage that it can be performed without the need for a response analysis for the sand deposit.

### Shear Strain Developed in Ground during Earthquakes

As stated earlier, the primary factor controlling settlements in dry sands is the cyclic shear strain induced in the soil at various depths. Values of this strain may be estimated as follows. At any given depth in a soil deposit, the effective shear strain, $\gamma_{eff}$ induced by earthquake shaking may be estimated from the following relationship:

$$\gamma_{eff} = \frac{\tau_{av}}{G_{eff}} = \frac{\tau_{av}}{G_{max} \cdot \left(\frac{G_{eff}}{G_{max}}\right)} \quad \text{(5)}$$

in which $G_{max}$ = shear modulus at low strain level; $G_{eff}$ = effective shear modulus at induced strain level; and $\tau_{av}$ = average cyclic shear stress at the corresponding depth. $\tau_{av}$ may be computed from the following relationship (Seed and Idriss 1971):

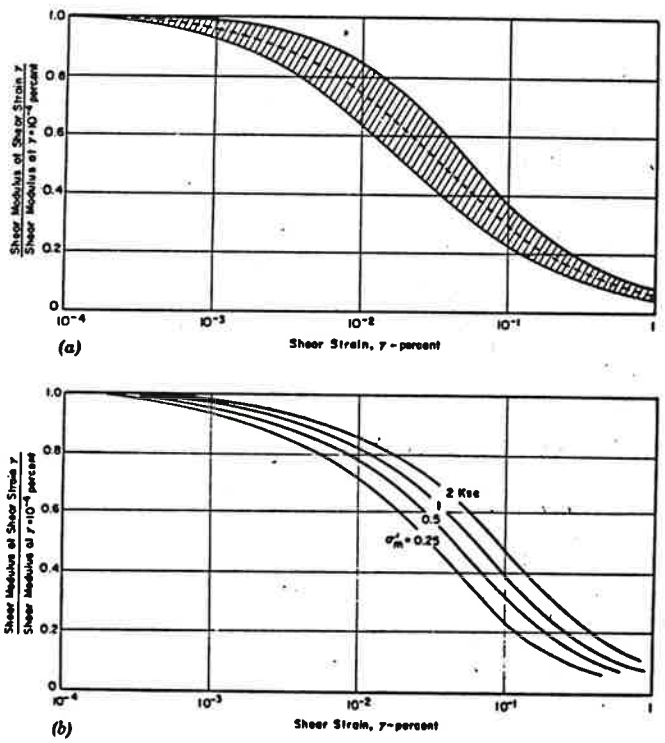



FIG. 10.—Relationship between Shear Modulus and Strain for Sands: (*a*) after Seed, et al. 1984; (*b*) after Iwasaki, et al. 1978



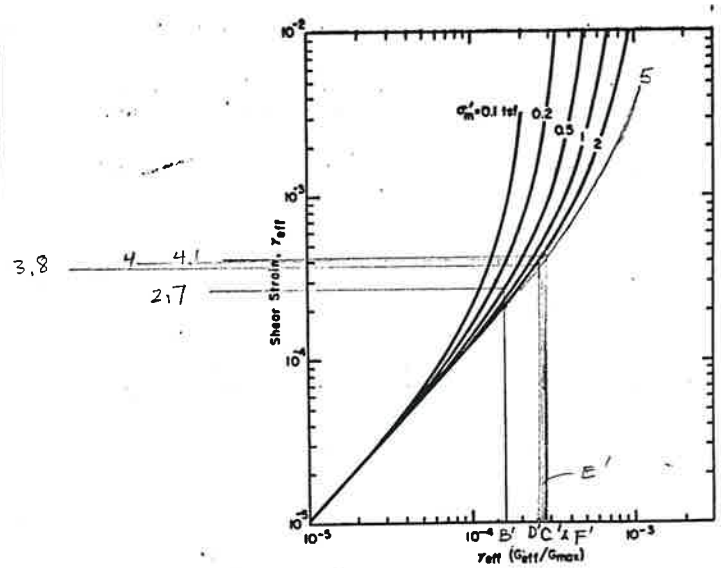



FIG. 11.—Plot for Determination of Induced Strain in Sand Deposits

$$\tau_{av} = 0.65 \cdot \frac{a_{max}}{g} \cdot \sigma_o \cdot r_d \quad \text{(6)}$$

Substituting Eq. 6 into Eq. 5 and rearranging the terms leads to

$$\gamma_{eff} \frac{G_{eff}}{G_{max}} = \frac{0.65 \cdot a_{max} \cdot \sigma_o \cdot r_d}{g \cdot G_{max}} \quad \text{(7)}$$

The right-hand side of this equation can readily be evaluated for any given depth, since $G_{max}$ may be determined from the following relationship (Seed and Idriss 1970):

$$G_{max} = 1{,}000 \cdot (K_2)_{max} \cdot (\sigma'_m)^{1/2} \quad \text{in psf units} \quad \text{(8a)}$$

$$\text{where} \quad (K_2)_{max} \simeq 20(N_1)^{1/3} \quad \text{(8b)}$$

Eq. 8*b* is based on the correlation proposed by Ohta and Goto (1976) as a result of numerous field shear wave measurements in Japan and subsequently modified to the form shown in Eq. 8*a* by Seed, et al. (1984b).

Having thus determined a value for the product $\gamma_{eff} \cdot (G_{eff}/G_{max})$, a value for $\gamma_{eff}$ can be determined as follows. A typical representative relationship between the ratio $G_{eff}/G_{max}$ and shear strain for sands, based on the work of many investigations, is shown in Fig. 10(*a*). Many investigators find relationships lying within the band shown in this figure (Seed, et al. 1984b). In detail, the relationship is also dependent on the confining pressure as shown by Hardin and Drnevich (1971), Shibata and Soelarno (1975), and Iwasaki, et al. (1978) and is perhaps best represented by the family of curves proposed by Iwasaki, et al. (1978), shown in Fig. 10(*b*). From these relationships, values of the product of $(G_{eff}/G_{max}) \cdot \gamma_{eff}$

can be computed and plotted against the effective shear strain, $\gamma_{eff}$, as shown in Fig. 11.

Thus, since the value of the product $(G_{eff}/G_{max}) \cdot \gamma_{eff}$ can be determined for a layer at any depth by means of Eq. 7, the corresponding value of $\gamma_{eff}$ can readily be read from the curves presented in Fig. 11. This procedure is not as accurate as performing a dynamic response analysis, but it is probably accurate enough for most settlement estimates in practice.

## Volumetric Strain versus Shear Strain for Dry Sands

Silver and Seed (1971) presented relationships between volumetric strain and shear strain for sands at different relative densities; Fig. 12 summarizes these relationships after 15 cycles. Combining the relationships presented in Figs. 12 and 3 leads to the relationships between volumetric strain and shear strain for sands with different SPT $N_1$-values, as shown in Fig. 13.

The relationships shown in Figs. 12 and 13 are applicable only for cases involving 15 equivalent uniform strain cycles, which are representative of a magnitude 7.5 earthquake. However, the results can be extended to different magnitude events by a procedure similar to that adopted for saturated sands. The number of cycles representative of different magnitude events and the corresponding volumetric strain ratios are shown in Table 4.

A review of previous studies (e.g., Silver and Seed, 1971) shows that the volumetric strain ratios for different numbers of cycles normalized to that for 15 cycles generally falls within the shaded zone shown in Fig.

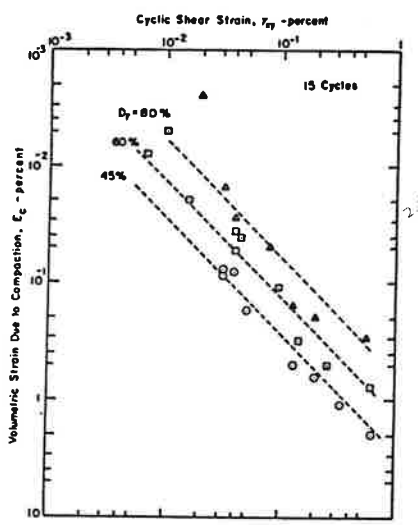



FIG. 12.—Relationship between Volumetric Strain and Shear Strain for Dry Sands (after Silver and Seed 1971)

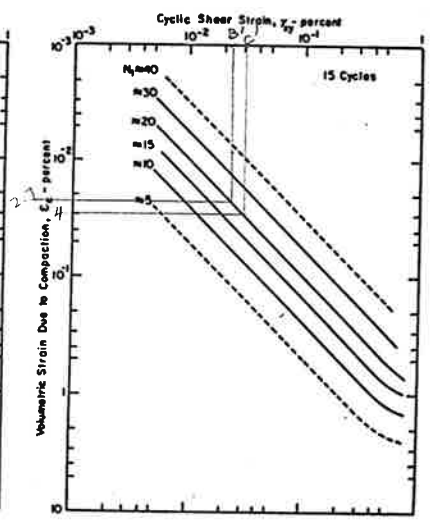



FIG. 13.—Relationship between Volumetric Strain, Shear Strain, and Penetration Resistance for Dry Sands





TABLE 4.—Influence of Earthquake Magnitude on Volumetric Stra[illegible] for Dry Sands

| Earthquake magnitude (1) | Number of representative cycles at 0.65 $\tau_{max}$ (2) | Volumetric strain ratio, $\varepsilon_{c,N}/\varepsilon_{c,N=15}$ (3) |
|---|---|---|
| 8-1/2 | 26 | 1.25 1.125 |
| 7-1/2 | 15 | 1.0 |
| 6-3/4 | 10 | 0.85 |
| 6 | 5 | 0.6 |
| 5-1/4 | 2–3 | 0.4 |

14 and may be represented by the average line shown in the figure. Thus, the ratio of values of volumetric strain in any number of cycles to that for 15 cycles can readily be determined from Fig. 14, and values are listed in Col. 3 of Table 4. By multiplying the volumetric strain from Fig. 13 by the factor shown in Col. 3 of Table 4, the volumetric strain can be determined for earthquakes with different magnitudes. It should be noted, however, that the correction factors are different from those listed in Table 1, since the correction is made here for volumetric strain rather than shear stress ratio.

Finally, it is necessary to note that the relations shown in Fig. 13 are based on unidirectional simple shear conditions, whereas under actual earthquake loading conditions, soils are subjected to multidirectional shaking. Tests by Pyke, et al. (1975), using multidirectional shear as well as unidirectional shear, led to the conclusion that "the settlements caused by combined horizontal motions are about equal to the sum of the settlements caused by the components acting alone." This means that the volumetric strain estimated from Fig. 14 should be doubled to take multidirectional shaking effects into account.

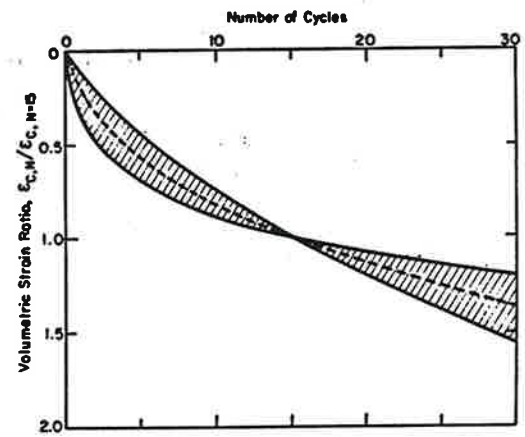



FIG. 14.—Relationship between Volumetric Strain Ratio and Number of Cycles for Dry Sands

Seed and Silver (1972) computed the settlement in a 50-ft thick deposit of sand with a relative density of 45%, which was subjected to a maximum surface acceleration of 0.45 $g$ (see Fig. 15). They concluded that the computed settlement of 2.5 in. was in fairly good agreement with the observed settlement during the San Fernando earthquake of 1971. The same soil profile has been evaluated using the simplified method described earlier. The results of the analyses are shown in Table 5, and the results are compared with those by Seed and Silver (1972) in Fig. 15. It may be noted that the strain distribution determined by the approximate method is in good agreement with the values computed by Seed and Silver and that the settlement estimated by the simplified procedure is in reasonable accordance with that determined previously.

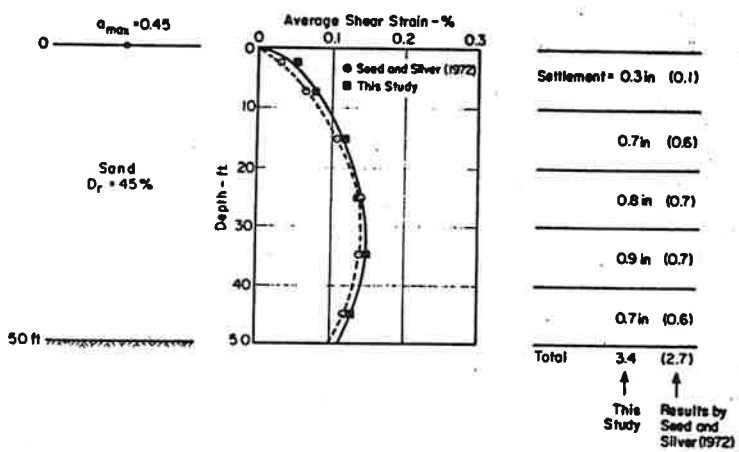



**FIG. 15.—Computation of Settlement for 50-ft Deep Sand Layer**

**TABLE 5.—Computation of Settlement for Deposit of Dry Sand**

| Layer number (1) | Thickness (ft) (2) | $\sigma_o = \sigma'_o$ (psf) (3) | Fig 3 $D_r$ (%) (4) | $N_1$ (5) | $G_{max}$[a] (ksf) (6) | Eq (7) $\gamma_{eff}(G_{eff}/G_{max})$ (7) | Fig 11 $\gamma_{eff}$ (8) | Fig 13 $\varepsilon_{C,M=7.5}$ (%) (9) | Table 4 $\varepsilon_{C,M=6.6}$[b] (%) (10) | $2\varepsilon_{C,M=6.6}$[c] (%) (11) | Settlement (in.) (12) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 5 | 240 | 45 | 9 | 520 | $1.3 \times 10^{-4}$ | $5 \times 10^{-4}$ | 0.14 | 0.11 | 0.22 | 0.13 |
| 2 | 5 | 714 | 45 | 9 | 900 | $2.3 \times 10^{-4}$ | $8 \times 10^{-4}$ | 0.23 | 0.18 | 0.36 | 0.22 |
| 3 | 10 | 1,425 | 45 | 9 | 1,270 | 3.2 " | 12 " | 0.35 | 0.28 | 0.56 | 0.67 |
| 4 | 10 | 2,375 | 45 | 9 | 1,630 | 4.0 " | 14 " | 0.40 | 0.32 | 0.64 | 0.77 |
| 5 | 10 | 3,325 | 45 | 9 | 1,930 | 4.5 " | 15 " | 0.45 | 0.36 | 0.72 | 0.86 |
| 6 | 10 | 4,275 | 45 | 9 | 2,190 | 4.6 " | 13 " | 0.38 | 0.30 | 0.60 | 0.72 |
| Total | | | | | | | | | | | 3.37 |

[a] $G_{max} = K_2 \cdot 1{,}000\,(\sigma'_m)^{1/2} = 20\,N_1^{1/3}(\sigma'_m)^{1/2} \times 1{,}000$.
[b] $\varepsilon_{C,M=6.6}/\varepsilon_{C,M=7.5} = 0.80$.
[c] Multidirectional effect.



## CONCLUSIONS

Simplified methods of analysis have been proposed for estimating probable settlements of either saturated or unsaturated sand deposits subjected to earthquake shaking. Based on a review of previous studies, it appears that the primary factors controlling earthquake-induced settlement are the cyclic stress ratio for saturated sands with pore pressure generation and the cyclic shear strain for dry or partially saturated sands, together with the N-value or relative density of the sand and the magnitude of this earthquake.

It should be recognized that, even under static loading conditions, the error associated with the estimation of settlements in sands is on the order of 25–50%. It is therefore reasonable to expect less accuracy in predicting settlements for the more complicated conditions associated with earthquake loading. However, comparison of the numerical results with several case histories indicates that the methods presented herein can be used, in many cases, as a first approximation for evaluating the volume changes and settlements of sands due to earthquake shaking. In the application of the methods, it is of course essential to check that the final results are reasonable in the light of available experience.

## ACKNOWLEDGMENTS

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C.4-47

# Unified Approach for Beams on Ela Foundations under Moving Loads

By A. M. Trochanis,[1] A. M. ASCE, R. Chelliah,[2] and J. Bielak,[3] M. ASCE

**Abstract:** This paper describes a simple unified procedure, based on the fast Fourier transform (FFT) technique, for analyzing the steady-state response of an infinitely long beam supported on an arbitrary linear elastic, damped foundation and subjected to moving oscillatory loads. As an application, the procedure is used to analyze the response of a simplified model of railroad track; the base is idealized either as a Kelvin model, or as a two-layer Vlasov model for an improved representation of the actual track. For the limiting static problem the results are compared with those from experiments and from a more realistic model (GEOTRACK); agreement between the various results is reasonable, despite the simplicity of the Kelvin and Vlasov models. A parametric study is conducted to assess the significance of system parameters such as the velocity and the frequency of the moving load and the stiffness and damping characteristics of the foundation on the system response. Response quantities evaluated include beam deflections, bending moments, and contact pressure as well as the energy dissipated within the foundation.

## Introduction

This work has two main objectives. The first is to present a unified, efficient, and easy-to-implement procedure for obtaining accurate results for the steady-state response of an infinitely long beam on an arbitrary elastic foundation due to oscillatory loads traveling along the beam. The second objective is to estimate analytically the corresponding energy losses occurring in the foundation due to material damping. Applications include railroad tracks, highway and airport concrete pavements, floating ice covers, and guideways for missile test sleds (Kerr 1981). The case of a railroad track resting on a soil foundation, which is represented by two different simple models, is used for demonstration of the proposed analysis; the track is subjected to a moving oscillatory load, modeling the wheel set of a train traveling at operational speeds and oscillating vertically at a constant frequency.

Many investigations of the response of beams on elastic foundations using various foundation models have been conducted in recent years, including several specific applications to railroad track response. A detailed survey of continuously supported beams and plates subjected to moving loads is given by Kerr (1981). Early work, such as that reported by Timoshenko and Langer (1932) in a pioneering paper on track dynamics, made use of the Winkler foundation to represent the subgrade. This model, which consists of a set of closely spaced linear spring ele-

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[3]Prof., Dept. of Civ. Engrg., Carnegie-Mellon Univ., Pittsburgh, PA 15213.
Note.—Discussion open until January 1, 1988. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on August 4, 1986. This paper is part of the *Journal of Geotechnical Engineering*, Vol. 113, No. 8, August, 1987. ©ASCE, ISSN 0733-9410/87/0008-0879/$01.00. Paper No. 21728.

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## RECORD OF TELEPHONE CONVERSATION

☐ INCOMING CALL ☑ OUTGOING CALL

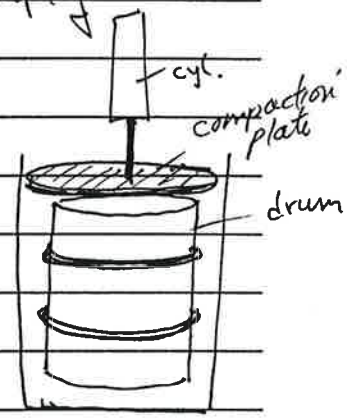
CONTACT INFO.* Mike, Dave Sanford
TELEPHONE NO. 1 800 272 7716
COMPANY NAME Rampactor
COMPANY ADDRESS 362 So. Main St. Clearfield, Utah 84015

STAFF INFO. Tom Witte
DATE 1/30/96
TIME 9:00 AM
PROJECT NO. 95-284
PROJECT NAME US Ecology Beatty NEV
PAGE 1 OF 1

SUBJECT Drum Compaction

Mike at ~~Versapak~~ Rampactor gave the following info.

Drum type - DOT 55 gal steel drum. - empty
hydraulic Ram type compactor




| lbs | final ht of compaction |
|---|---|
| 40,000 | 5-6" |
| 60,000 | 4-5" |
| 85,000 | 2½-3" |

I asked him what minimum force was necessary to start (begin) ~~abs~~ crushing (buckling). He said he didn't know but would find out.

He called back later to say he had asked a couple of the company engineers who estimated a force of 20,000 psi would be required to begin compaction and would ultimately result in a ~ 10-11" high compaction wafer.

Art Legros of VERS-A-PAK - called later and independently confirmed these estimates.

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VERS-A-PAK - COMPACTORS
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Fallbrook, CA 92028
619-728-1790 or Art Legros 714-827-1977

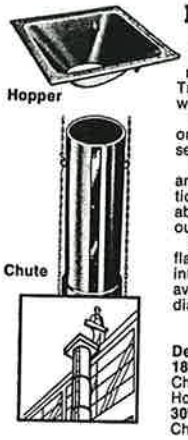


A-Gen Forms (12/8/94/bd)

Ref. (9)

# Disposal Equipment

## Rooftop Trash Chutes




**You can set up these tough, high-density polyethylene trash chutes in minutes for roofing, demolition and other job-site work.** Trash can go directly into your dumpster, without dust or the danger of flying debris.

The chutes can be used on buildings from one to ten stories high. The easy-to-handle sections are each 4 feet long.

The sections are fitted with support chains and tapered to telescope into any other section. To use, hook section chains to section above, hoist to roof level, and secure to the outrigger supports.

Chutes are easy to disassemble and may be flattened for storage. To help funnel trash into the 18" diameter chute, a PVC hopper is available—it measures 37" square with an 18" diameter hole.

| Description | Wt., Lbs. | No. | NET EACH |
|---|---|---|---|
| **18" DIAMETER** | | | |
| Chute Section | 20 | 4986T42 | $81.82 |
| Hopper Only | 25 | 4986T41 | 120.37 |
| **30" DIAMETER** | | | |
| Chute Section | 25 | 4986T43 | 87.72 |

## Glass Bottle Crusher

**This safe, sanitary, and efficient crusher can save you money on waste removal and handling costs.**

The crusher breaks glass bottles up to 1 gallon and jugs up to 7" in diameter. It can crush 600 glass bottles and jugs per hour.

The crusher has an aluminum head that fits on top of 30- and 55-gallon steel drums (not included).

Rubber flaps in the chute prevent broken glass from escaping. A window viewer shows when drum is full. The crusher has a hardened-steel impeller for maximum wear resistance.

Drum-head inside diameter is 24⅛". Height is 25½". Motor is totally enclosed ½-hp, 115/230 VAC, 60 Hz, 1-ph. with an 8-ft. cord and 3-prong plug.

**No. 5003T13** ..... **NET EACH $784.00**

## Can and Glass Crusher

**With a large crushing chamber that eliminates feed problems and ensures high-volume output, this unit is perfect for restaurant, institution, and municipal recycling operations.**

The unit flattens up to 10,000 12-oz. cans per hour or up to 25 1-gallon cans per minute. It crushes over 2 tons of glass per hour into recyclable cullets. Cans are reduced to ⅓ their original volume. Glass is crushed to 1/20 original volume.

The crusher is made of ¼" heavy-steel plate and includes a hopper and stand. A 55-gallon steel drum (not included) placed under the stand collects output. Size including stand is 50" L × 34" W × 79½" H. Unit has an overload-protected, 2-hp, 230/460 VAC, 60 Hz, 3-ph. motor with 8-ft. cord and 3-prong plug.

**No. 40485T1** ..... **NET EACH $1984.00**

## Steel-Strapping Disposer

**Cuts strapping and band saw blades into manageable size.** Disposer chops ⅜" to 1¼" strapping and thin kerf band saw blades into uniform 3" to 5" lengths. Use on top of a 55-gal. steel drum (not included) to serve as base and container. Cutting shears and automatic feeding device are powered by a ½-hp, 115 VAC, 60 Hz, 1-ph. motor with 1-ft. cord and 3-prong plug. Has a release control lever for use in case of a jam. Size is 38"L × 15"W × 16" H.

**No. 40295T11 NET EACH $2381.82**

## Manual Trash Compactors

**Compacting your trash saves space and money.** You don't have to use as many trash bags, cans, and containers. *And,* fewer collection pickups are needed.

These high-leverage, steel units have compaction rates up to five to one. They take up little floor area, use no fuel or power to operate, and have no electrical parts to maintain. Use them anywhere—on job sites or in the plant. Trash is compacted in a drum or other container (not supplied) at up to 6500 pounds force and ram retracts with spring loading.

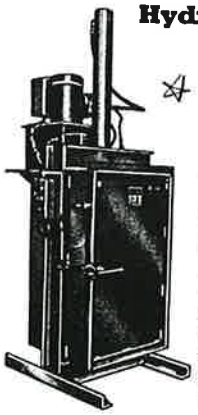
| Cap. Gal. | Ram Full Stroke | Overall W × H | No. | NET EACH |
|---|---|---|---|---|
| 15/20 | 19¼" | 26" × 34" | 40245T11 | $486.67 |
| 30 | 19¼" | 28" × 38" | 40245T12 | 491.67 |
| 55 | 24½" | 28" × 44" | 40245T13 | 516.67 |

## Dual Open Storage Enclosure

**This rugged steel enclosure will neatly hold two 55-gallon steel drums, and can be securely locked.** Drums are not included. Enclosure construction is all-welded 14-gauge steel with green enamel finish. The hinged top lid has a rod-style support. Two hinged front doors have a staple hasp inside for padlocking (lock not included). Bottom is open with four full- length steel angles welded to sides for drum support. Overall size is 52" wide × 27" deep × 42" high. Weight is 195 lbs. Enclosure is fully assembled and ready to use.

| No. | NET EACH |
|---|---|
| 9865T52 | $366.47 |

## Hydraulic Drum Crusher



**You don't have to use valuable space to store damaged and corroded 55-gallon drums until they can be trucked away.** This unit's 24,000 pounds of force crushes 33" high drums down to about 6" in just 25 seconds.

All moving parts are completely enclosed. The crusher automatically shuts down if the door is unlocked or power is lost. When the door is locked, the unit is repowered and the ram automatically recycles itself to its highest position. A system relief valve prevents overload. Pilot lights show direction of ram travel.

The 2000-psi hydraulic system has a 10-gallon reservoir and meets JIC standards. Crusher is made of sturdy 10-gauge steel. Unit has a 5-hp, 230/460 VAC, 60 Hz, 3-ph. motor with an 8-ft. cord, 3-prong plug, and keyed on/off switch. Up/down controls are waterproof pushbuttons. Overall dimensions are 36" wide × 44" deep × 90" high. Weight is 1476 lbs.

**No. 44565T11** ..... **NET EACH $5828.67**

## Plastic-Strapping Disposer

**The disposer cuts up to four plastic straps at one time into 2" long pieces and gently drops them into the 55-gal. steel drum below (not included).** It operates at 150 feet per minute as the rotating dual knife makes 933 cuts per minute. You can adjust the knife holder for most efficient cutting of your materials. The disposer is powered by a ½-hp, 115 VAC, 60 Hz, 1-ph. motor with 3-ft. cord and 3-prong plug. Size is 35" L × 18" W × 13" H. The unit has handles for easy carrying.

**No. 40405T21** ..... **NET EACH $1694.69**



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| 16" × 20" | 3984 |
| 18" × 24" | 3984 |
| 18" × 30" | 3984 |
| 24" × 37" | 3984 |

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**These handy holde hold mail bags as w bags.** Spring action k

Holders are availabl the base for sliding ove casters with rubber wh 18" long × 12" wide × 3

**Regulation post wide × 36½" deep mail** is made of heavy whit draw cord, and rope lo

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| Bag Holder with Swive | |
| Mail Bag | |

## Stand

**Save storage space- folded until it's needed ber drum or a box.** The cally square, it assumes lbs. of contaminated m burned along with the w the inner liner and oute

The inner liner is 3-m wire tie that is includec polypropylene fabric wi tie and two lifting loops

**No. 8765T21** .....

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EPA/600/2-85/035
April 1985

# SETTLEMENT AND COVER SUBSIDENCE OF HAZARDOUS WASTE LANDFILLS

(FINAL REPORT)

by

W. L. Murphy and P. A. Gilbert
US Army Engineer Waterways Experiment Station
Vicksburg, Mississippi 39180

Interagency Agreement No. AD-96-F-2-A079

Project Officer

Robert P. Hartley
Solid and Hazardous Waste Research Division
Municipal Environmental Research Laboratory
Cincinnati, Ohio 45268

HAZARDOUS WASTE ENGINEERING RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
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15. SUPPLEMENTARY NOTES

16. ABSTRACT

Numerical models using equations for linearly elastic deformation were developed to predict the maximum expected amount of settlement and cover subsidence and potential cracking of the cover by differential settlement in uniformly, horizontally layered hazardous waste landfills. The hazardous waste landfill models represented landfills in which unsaturated wastes were contained in steel drums that were assumed to deteriorate ultimately. The model landfill consisted of alternating layers of drummed wastes and intermediate clay cover. The maximum expected subsidence was calculated to be about 12% of the total landfill thickness. Finite element analysis of differential settlement across the landfill indicates that tensile stresses do not develop in the model case, and therefore that cracking by settlement does not occur. The findings represent only one set of conditions, however, and should not be construed as representative of landfill phenomena in general.

17. KEY WORDS AND DOCUMENT ANALYSIS

| a. DESCRIPTORS | b. IDENTIFIERS/OPEN ENDED TERMS | c. COSATI Field/Group |
|---|---|---|
| | | |

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$$S_T = S_{ISL} + S_{LV} + S_{DV} + S_E \quad (8)$$

where

$S_{ISL}$ = subsidence due to consolidation of the intermediate cover layers

$S_{LV}$ = subsidence due to closing of the inherent geometric layer void space

$S_{DV}$ = subsidence attributed to void space inside the drums. This component is assumed to be zero, but should be quantified and accounted for in a general approach

$S_{\Delta E}$ = subsidence due to change in stiffness of the waste layers

The maximum volume of void, or cavity, that can be included in the landfill by drum placement may be calculated geometrically for drums placed on their sides and for drums placed on end. The maximum cavity volume for the former is 10.73 percent of the total landfill volume and for the latter is 9.31 percent. If the cavities close completely, the maximum total subsidence must be the sum of cavity closure plus subsidence by compression of the landfill contents. However, actual landfill operations will reduce maximum cavity volume because the operations do not achieve perfect drum placement and because filling of interdrum space by sifting of cover soils during filling generally occurs to some extent. Conversely, any unexpected void within the drums would contribute to the volume of inherent cavity.

Deformation by Degradation of Waste Layer Modulus

For the sake of simplicity and so that time does not enter the analysis, the treatment given here will be that of linear elasticity. The end product of this treatment will be the amount of subsidence or the potential for subsidence caused by a decreasing waste-layer modulus, given in terms of the pertinent parameters, but the work will not address how the subsidence occurs with time.

Assume again a central column (Figure 17) of length $L$ and consisting only of waste material. The cover layers have been removed. The distance from the top of the layer to a differential element $dy$ is $y$. The edges of the column ride on rollers so that the area of the column does not change. Plane strain conditions exist in the column. Therefore

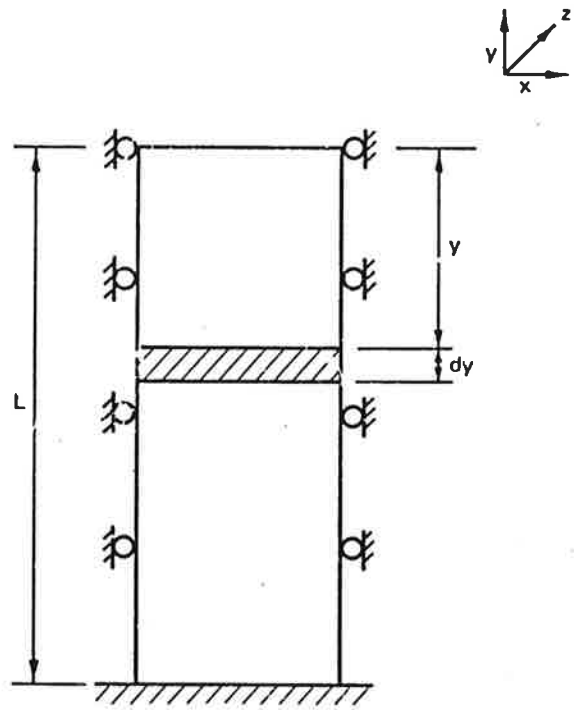


Figure 17. Schematic of central column and coordinate system for subsidence model.

$$\varepsilon_y = \frac{1 + \nu}{E} (\sigma_y (1 - \nu) - \nu\sigma_x) \tag{9}$$

$$\varepsilon_x = \frac{1 + \nu}{E} (\sigma_x (1 - \nu) - \nu\sigma_y) \tag{10}$$

$$\varepsilon_z = 0 \tag{11}$$

where $\varepsilon_x$, $\varepsilon_y$, $\varepsilon_z$ = strain in the x, y, and z directions, respectively.

$\sigma_x$, $\sigma_y$ = stress in the x, y directions, respectively

E = Young's Modulus

$\nu$ = Poisson's Ratio

The lateral stress $\sigma_x$ is the product of the coefficient of earth pressure at rest, $K_o$, and the vertical stress, or

$$\sigma_x = K_o \sigma_y \tag{12}$$

which can be shown to be

$$\sigma_x = \frac{\nu}{1 - \nu} \sigma_y \tag{13}$$

Combining Equations 9 and 13 yields

$$\varepsilon_y = \sigma_y \frac{1 + \nu}{1 - \nu} \frac{1 - 2\nu}{E} \tag{14}$$

Combining Equations 10 and 13 yields

$$\varepsilon_x = 0 \tag{15}$$

The vertical stress

$$\sigma_y = \gamma y \tag{16}$$

where

$\gamma$ = the unit weight (density) of the medium.

By definition, strain, $\varepsilon$, is the ratio of the change in length to the total length of the body or

$$\varepsilon = \frac{\Delta L}{L} \tag{17}$$

Therefore the strain in the differential element, dy , of Figure 17 is the change in length divided by the original length which, expressed in differential form, becomes

$$\epsilon_y = \frac{d\ (dy)}{dy} \tag{18}$$

Combining Equations 14, 16, and 18,

$$\frac{d\ (dy)}{dy} = \gamma\ y \left(\frac{1+\nu}{1-\nu}\right)\left(\frac{1-2\nu}{E}\right) \tag{19}$$

Therefore,

$$d\ (dy) = \gamma \left(\frac{1+\nu}{1-\nu}\right)\left(\frac{1-2\nu}{E}\right)\ y\ dy \tag{20}$$

Integration of Equation 20 results in the summation of the deformation in all differential elements over the length L of the column. Therefore,

$$\int_0^L d\ (dy) = \Delta L \tag{21}$$

where

$\Delta L$ = the total change in length of the column.

Combining Equations 20 and 21

$$\Delta L = \int_0^L \gamma \left(\frac{1+\nu}{1-\nu}\right)\left(\frac{1-2\nu}{E}\right) y\ dy \tag{22}$$

and

$$\Delta L = \frac{\gamma\ L^2}{2} \left(\frac{1+\nu}{1-\nu}\right)\left(\frac{1-2\nu}{E}\right) \tag{23}$$

Note that Equation 23 contains no stresses and suggests that the subsidence that is at the cell center and therefore maximum, varies directly with the density of the waste fill and with the square of the waste depth and inversely as the stiffness, E . This means that a column of material with density $\gamma$ , length L and elastic properties E and $\nu$ will deform an amount $\Delta L$ if suddenly "released to gravity" from a completely weightless state. Now from this point, if the elastic modulus E begins to decrease and deteriorates from an initial value $E_i$ to a final value $E_f$ then the resulting subsidence will be

$$S_{max} = \frac{\gamma L^2}{2} \left(\frac{1 + \nu}{1 - \nu}\right) (1 - 2\nu) \left(\frac{E_i - E_f}{E_i E_f}\right) \qquad (24)$$

and if the change in elastic modulus is known, $S_{max}$ may be computed directly from Equation 24. $S_{max} = \Delta L_f - \Delta L_i$ and Equation 24 is derived by direct substitution of $\Delta L_f$ and $\Delta L_i$ into Equation 23.

Effect of Poisson's Ratio

Poisson's ratio ($\nu$) logically changes and influences the results as subsidence occurs, but the manner in which it changes is not clearly understood. Lambe and Whitman[40] suggest that the value of $\nu$ can usually be estimated with satisfactory accuracy as 0.35 for soils of low saturation and 0.5 for fully saturated soils. The value of 0.5 will enforce material incompressibility and consequently no subsidence will occur at that Poisson's ratio. At the opposite extreme, a Poisson's ratio of zero indicates behavior where a vertical stress causes vertical strain but no tendency for horizontal strain. A material such as loose snow has a Poisson's ratio approaching zero. Practically, waste in a hazardous waste landfill will be neither of these extremes but something in between. Lambe and Whitman[40] recognized the difficulty of making an exact evaluation of Poisson's ratio for use in any problem, but suggested that the value usually has a relatively small effect on engineering predictions. Craig[41] states that during consolidation Poisson's ratio decreases from the undrained value to the drained value at the end of consolidation. The decrease does not affect the vertical stress and slightly decreases the horizontal stress, but this decrease is neglected in the Skempton-Bjerrum method of consolidation settlement prediction.

Al-Hussaini[42] performed $K_o$ tests on sand under conditions of triaxial compression. During the $K_o$ tests, radial strain was not permitted and the horizontal and vertical stresses were measured and vertical strain occurred. The ratio of horizontal to vertical stress under these conditions is, by definition, $K_o$. These tests showed that $K_o$ remained constant during loading up to axial pressures of 300 psi. Since $K_o$ and Poisson's ratio are related in linear elasticity by the equation

$$\nu = \frac{K_o}{1 + K_o} \qquad (25)$$

materials. Both materials were soaked in water for 48 hours to allow for the maximum possible absorption of water. The materials were then spooned into an oedometer (one-dimensional consolidation device) to simulate the probable low density achieved by the field practice of pouring the material into a drum half filled with liquid. The specimens were then loaded up to 55 psi in the oedometer in five or six increments allowing complete drainage under each increment. The results of the loading are shown on Figure 21.

The first material is sold commercially as a compound to absorb oil spills. It is basically a calcium bentonite claystone which was removed from a quarry, crushed, then kiln-dried at 1100°F (calcined). The resulting material has the appearance of a coarse angular sand with hard granules which remained hard and intact after soaking. The constrained secant modulus of this material was measured to be about 1,000 psi or 144,000 psf (see Figure 21).

The second material is sold commercially as an absorbent for use commonly as "kitty litter." The material is identical in composition to the first material except that after crushing, it is kiln-dried at only 300°F (i.e., it is noncalcined). The resulting material had the same physical appearance as the first material but after soaking became soft and plastic and the resulting modulus was measured to be about 275 psi or 40,000 psf. Since the objective was to estimate maximum subsidence, the lower modulus of 40,000 psf (the noncalcined claystone) was used in the calculations.

Referring again to Figure 20, the final configuration was analyzed in exactly the same manner as the intact drum configuration except that for the final configuration the drum volume is replaced by elastic material with a modulus of 40,000 psf (the modulus of the noncalcined claystone). If a similar analysis is performed assuming that the two materials will compress as if loaded by an isotropic pressure then the equivalent elastic modulus may be shown to be

$$E_{Tf} = \frac{4E_c E_s}{\pi E_s + 4E_c (1/2 - \pi/8)} \qquad (28)$$

where $E_{Tf}$ = Final elastic modulus of composite system, in psf
$E_s$ = Elastic modulus of intermediate clay cover in psf
$E_C$ = Elastic modulus of drum contents in psf

* See derivation in Appendix B.

If the values of 40,000 psf for modulus of drum contents and 104,000 psf for intermediate cover modulus are substituted into Equation 28, then the equivalent composite modulus is computed to be 48,000 psf. The corresponding subsidence for this value is 7 in. taken from Figure 18. Now if the initial value of 0.4 in. is subtracted, then the resulting subsidence is 6.6 in., which corresponds to 0.89 percent strain in a total height of 744 in. (62 ft). But this is not all the subsidence that can occur. The air void (cavity) beneath the drum can be shown to be 10.73 percent of the total volume. If this space closes completely during subsidence, then the total value will be the sum of the two or 10.73 + 0.89 = 11.62 percent (approximately 87 in.) for the maximum expected subsidence. If all the waste drums are assumed to contain 10 percent (allowable) void space, the additional 10 percent represents an additional 7.9 percent waste layer cavity volume (drums occupy 79 percent of the waste layer for drums or their sides). The total estimated subsidence considering 10 percent void space in the containers would then be 11.62 + 7.9 = 19.52 percent or 145 in. (12.1 ft). However, it seems unlikely that all the drums would contain the maximum 10 percent allowable void space.

Drums on End. Similarly, if drums that are buried on end are represented by hollow steel cylinders that carry the axial compressive load without buckling then the effective elastic modulus of such a structure can be shown to be

$$E_D = 3.86 \times 10^7 \text{ psf} \quad * \tag{29}$$

which corresponds approximately to zero initial subsidence on Figure 19.

When the drum has completely corroded and the remaining material is taken to have a modulus of 40,000 psf then the final value of subsidence taken from Figure 19 is 11.7 in. This corresponds to 1.56 percent strain in a height of 750 in. It can be shown that the cavity space between drums in the densest possible stacking configuration comprises 9.31 percent of the total volume. If this space closes completely during settlement then the total value will be 1.56 + 9.31 = 10.87 percent (approximately 82 in.) for the maximum expected subsidence (drums-on-end). If all the waste drums are assumed to contain 10 percent (allowable) void space, the additional 10 percent

---

* See derivation in Appendix B.

## APPENDIX B

## DERIVATIONS OF EQUATIONS FOR YOUNG'S MODULUS

(1) Young's modulus of drum subjected to uniform external pressure.

The assumption is made that a drum lying on its side will be subjected to a uniform pressure $\Delta P$ and that

$$(U)_{r=b} = \frac{-b\,\Delta P}{E}\left[\frac{a^2 + b^2}{b^2 - a^2} - \nu\right] \tag{1}$$

where

$(U)_{r=b}$ = radial deformation of the drum at radius b.

b = outer radius of the drum (= 11.175 in).

a = inner radius of the drum (= 11.125 in).

E = Young's modulus of the drum material (steel, $E = 30 \times 10^6$ psi).

$\nu$ = Poisson's ratio of drum material (steel, $\nu = 0.29$).

A unit thickness will be chosen so that the initial volume $V_i = \pi r^2$. If the final volume is $V_f = \pi(r + \Delta r)^2$ then the change in volume of the system neglecting higher order terms is

$$\Delta V = V_f - V_i = \pi\,(2r\,\Delta r) \tag{2}$$

and

$$\frac{\Delta V}{V} = \frac{2\pi\, r\Delta r}{\pi\, r^2} = \frac{2\Delta r}{r} \tag{3}$$

which becomes by substituting in equation (1)

$$\frac{\Delta V}{V} = \frac{2\Delta P}{E}\left[\frac{a^2 + b^2}{b^2 - a^2} - \nu\right] \tag{4}$$

and if one dimensional compression is assumed, the modulus of the drum. $E_D$ is

$$E_D = \frac{\Delta P}{\left[\frac{\Delta V}{V}\right]} = \frac{E}{2\left[\frac{a^2 + b^2}{b^2 - a^2} - \nu\right]} \tag{5}$$

which becomes upon substitution

$$E_D = 67352 \text{ psi} = 9.7 \times 10^6 \text{ psf} \tag{6}$$

(2) Composite Young's modulus of soil/drum configuration for drums on their sides.

The assumption is made that the two-component system (the drum and soil, see Figure 20) will compress hydrostatically under pressure, i.e., the bulk modulus of the system can be computed from

$$K = \frac{\Delta P}{\left[\frac{\Delta V}{V}\right]} \tag{7}$$

where

$\Delta P$ = the increase in pressure applied to the system.

$\frac{\Delta V}{V}$ = volumetric strain of the system.

For the assumed condition of one dimensional compression, the bulk modulus, K, is equal to the Young's modulus,

$$K = E = \frac{\Delta P}{\left[\frac{\Delta V}{V}\right]} \tag{8}$$

The change in volume of the drum as a result of a change in pressure is equal to

$$\Delta V_D = \frac{V_D\ (\Delta P)}{E_D} \tag{9}$$

Similarly, the change in volume of the affected soil is

$$\Delta V_s = \frac{D^2\ (1/2 - \pi/8)\ \Delta P}{E_s} \tag{10}$$

where

D = drum diameter.

$E_s$ = Young's modulus of the soil.

The total change in volume of the system is therefore

$$\Delta V_{TOTAL} = \Delta V_s + \Delta V_D \tag{11}$$

and

$$\Delta V_{TOTAL} = \frac{V_D\ \Delta P}{E_D} + \frac{D^2\ (1/2 - \pi/8)\ \Delta P}{E_s} \tag{12}$$

The volume of the drum is $\pi D^2/4$ and the total volume of the system is $D^2$ for unit thickness. Therefore, equation (12) reduces to

$$\frac{\Delta V_{TOTAL}}{V_{TOTAL}} = \Delta P \left[ \frac{\pi}{4E_D} + \frac{(1/2 - \pi/8)}{E_s} \right] \tag{13}$$

which when combined with equation (8) yields

$$E_{composite} = E_T = \frac{4E_D\ E_s}{\pi\ E_s + 4E_D\ (1/2 - \pi/8)} \tag{14}$$

(3) Composite Young's modulus of soil/drum configuration for drums on end.

The assumption is made that for the drums standing upright (on end), a uniform pressure $\Delta P$ is applied over the top surface of the drum and that the contents will initially contribute no stiffness to the system. Therefore the total force applied to a drum is

$$F = A_T\ \Delta P \tag{15}$$

where $A_T$ is the total area of the top of the drum and equal to $\pi b^2$. The deflection, $\delta$, of the drum is

$$\delta = \frac{FL}{A_s\ E} \tag{16}$$

where

$L$ = drum length

$E$ = Young's modulus of drum material (for steel, $E = 30 \times 10^6$ psi)

$A_s$ = area of the material of drum = $\pi (b^2 - a^2)$

$b$ = outer drum radius = 11.175 in.

$c$ = inner drum radius = 11.125 in.

Therefore

$$\delta = \frac{A_T\ \Delta PL}{A_s\ E} \tag{17}$$

But the strain in the drum is the ratio $\frac{\delta}{L}$ and the applied stress is $\Delta P$. Therefore the effective Young's modulus of the drum is the ratio

$$E_D = \frac{\Delta P}{(\delta/L)} = \frac{A_s\ E}{A_T} \tag{18}$$

$\pi (b^2 - a^2)$

$\pi b^2$

which yields

$$= 2.68 \times 10^5 \text{ psi}$$

$$E_D = 3.86 \times 10^7 \text{ ~~psi~~ psf} \tag{19}$$

when appropriate values are substituted.

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By _____ Date _____ Subject Waste Settlement Sheet No. ____ of ____

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Reference No. 7



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## Simp Procedure for Evaluating Stresses Induced by Earthquakes

The shear stresses developed at any point in a soil deposit luring an earthquake appear to be due primarily to the vertical ropagation of shear waves in the deposit. This leads to a sim-lified procedure for evaluating the induced shear stresses (Seed nd Idriss, 1971). If the soil column above a soil element at epth h behaved as a rigid body, the maximum shear stress on ie soil element would be

$$(\tau_{max})_r = \frac{\gamma h}{g} \cdot a_{max} \qquad (1)$$

here $a_{max}$ is the maximum ground surface acceleration and is the unit weight of the soil; see Fig. 39(a). Because the oil column behaves as a deformable body, the actual shear ress at depth h, $(\tau_{max})_d$, as determined by a ground response ialysis will be less than $(\tau_{max})_r$ and might be expressed by

$$(\tau_{max})_d = r_d \cdot (\tau_{max})_r \qquad (2)$$

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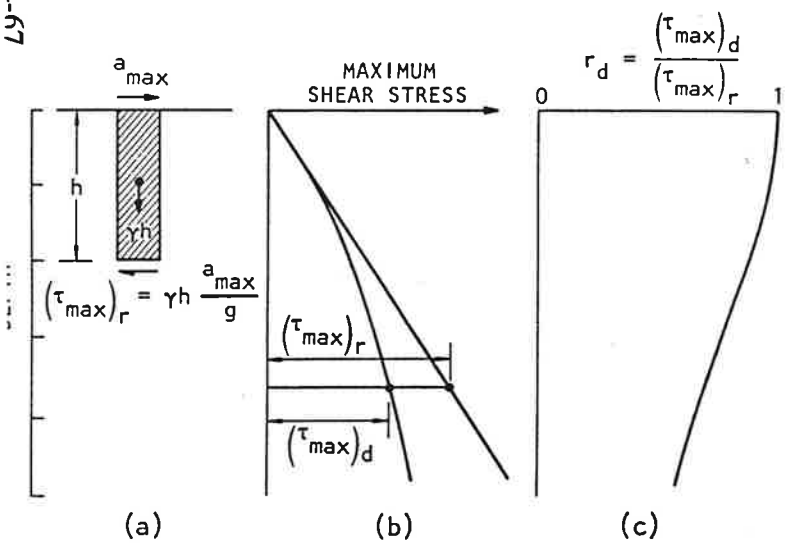


*Figure 39.* Procedure for determining maximum shear stress, $(\tau_{max})_r$.



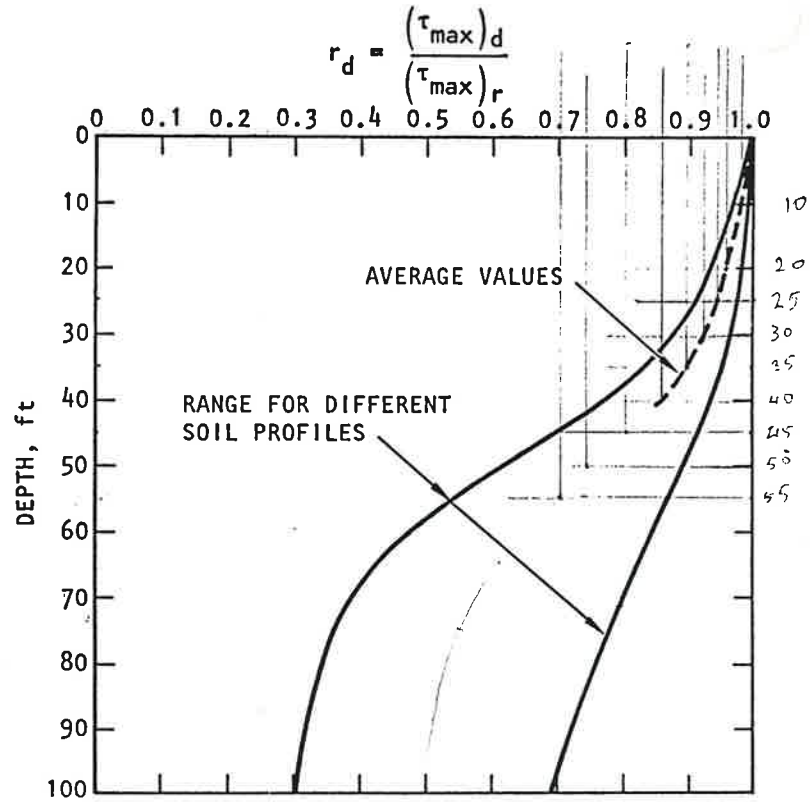


*Figure 40.* Range of values of $r_d$ for different soil profiles.

where $r_d$ is a stress reduction coefficient with a value less than 1. The variations of $(\tau_{max})_r$ and $(\tau_{max})_d$ will typically have the form shown in Fig. 39(b) and, in any given deposit, the value of $r_d$ will decrease from a value of 1 at the ground surface to much lower values at large depths, as shown in Fig. 39(c).

Computations of the value of $r_d$ for a wide variety of earthquake motions and soil conditions having sand in the upper 50 ft. have shown that $r_d$ generally falls within the range of values shown in Fig. 40. It may be seen that in the upper 30 or 40 ft., the scatter of the results is not great and, for any of the deposits, the error involved in using the average values shown by the dashed line would generally be less than about 5%. Thus



Ref 7 Seed & Idriss, 1982

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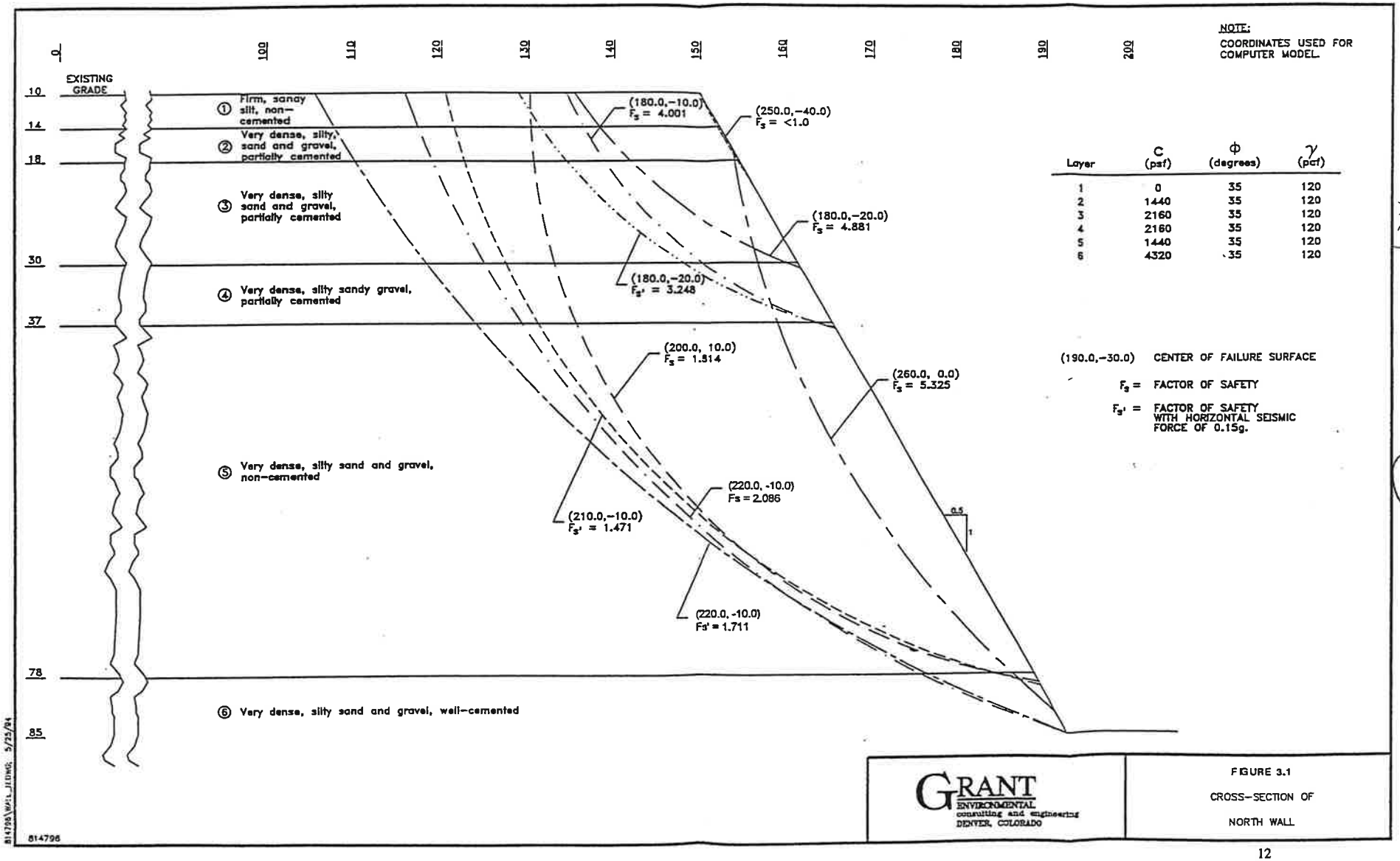


Reference No. 8

# C.5 LINER STRESSES

# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 3/6/96 Subject Liner Stresses Sheet No. 0 of 20

Chkd. By WB Date 3/7/96 Proj. No. 95-284

LINER STRESSES

US ECOLOGY HAZARDOUS WASTE LANDFILL

BEATTY, NV



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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 3/6/96 Subject Liner stresses Sheet No. 1 of 20

Chkd. By WB Date 3/7/96 Proj. No. 95-284

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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 3/6/96 Subject Liner stresses Sheet No. 2 of 20
Chkd. By VB Date 3/7/96 Proj. No. 95-284

## 1.0 PURPOSE

Evaluate tension/elongation of the design slope liner system to prevent rupture or damage to liner system due to gravity, thermal expansion/contraction, wind uplift, seismic deformation, and waste settlement.

## 2.0 APPROACH

The following causes of liner stresses are considered:

- Load due to gravity
- Thermal expansion/contraction
- wind uplift
- Seismic deformation
- Settlement of waste fill

<u>Load Due To Gravity</u> (Assume stable slopes)

Gravity load is calculated as follows:

$$T_1 = W \sin\theta + F_u - F_L$$

where $W$ = weight of liner component;

$\theta$ = slope angle [63.4° FOR 0.5:1 (H:V) slope]

$F_u$ = Friction Force above the liner component.

$F_L$ = Friction Force below the liner component.



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# ENVIRONMENTAL SOLUTIONS, INC.

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Strain caused by self weight is denoted as $\varepsilon_1$.

## Thermal Expansion/Contraction

Thermal expansion is considered for HDPE geomembrane and geonet. A coefficient of linear thermal expansion of $1.2 \times 10^{-4}$ cm/cm-°c is selected (See Ref. 3). Assume a temperature variation of 40°c (Ref. 4), the induced thermal strain is

$$\varepsilon_2 = 1.2 \times 10^{-4} \times 40 \doteqdot 0.5\%$$

## Wind Uplift

Sand bags and suction devices will be used to hold down the liner system during installation and waste placement. As a result, wind uplift will be balanced by sand bags and suction devices and will not generate stresses in liner system.

## Seismic Deformation

Seismic deformation during waste placement will be small (less than 1.6", see Ref. 13). Consider a slope length of 84', seismic deformation is $\doteqdot 0.2\%$, which will generate negligible stresses.

$$\Rightarrow \quad \varepsilon_3 = 0.2\%$$

Although negligible, this strain is included in the calculation for completeness. This strain will not occur below Interface No. 3, see Figure 1 and discussions below.



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## Settlement of Waste fill

From Ref. 8, waste fill settlement will be less than 4% which includes settlements during waste placement and after closure of landfill. Waste fill settlement will induce upto 4% strains in the upper layers (non-woven geotextile and geonet) of the liner system, See Figure 1.

The liner system is designed such that slippage will occur between geonet and the smooth side of HDPE membrane (i.e., slippage will occur at Interface No. 3).

As a result, waste fill settlement will not affect the liner system below Interface No. 3. (Similarly, strain induced by seismic deformation will not be transferred below this interface.)

## Induced Stresses

The overall strains caused by the above factors are added and the corresponding liner component stresses are determined from stress-strain curves or the elastic (secant) modulus of the material.

## Allowable Stresses of Geotextile

The geotextile is used for separation purposes and the following partial factors of safety are used to calculate allowable tensile stress:

$$T_{allow} = T_{ult} \left( \frac{1}{FS_{ID} \times FS_{CR} \times FS_{CD} \times FS_{BD}} \right) \qquad \text{(Ref. 10)}$$

where $FS_{ID} = 2.5$, $FS_{CR} = 1.2$, $FS_{CD} = 1.5$, and $FS_{BD} = 1.2$.



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By JH Date 2/1/96 Subject TENSION/ELONGATION Sheet No. 5 of
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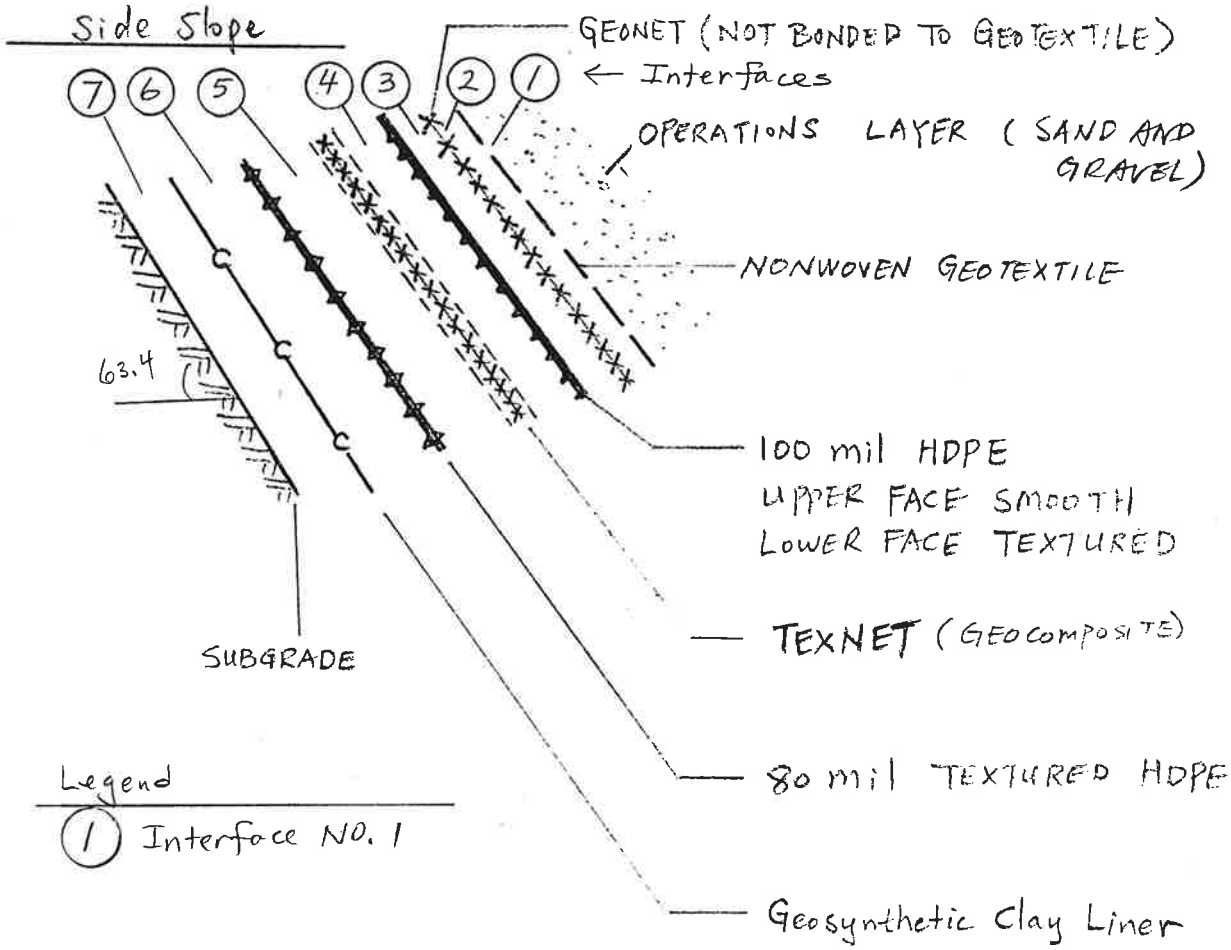


Legend
(1) Interface NO. 1

Figure 1 Side Slope Liner System

Note: See Tables 1, 2, and 3 for material properties of liner components.



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Ref (10) 6/20



**TABLE 2.12** RECOMMENDED PARTIAL FACTOR OF SAFETY VALUES FOR USE IN EQUATION 2.18

| Application area | Various partial factors of safety | | | |
|---|---|---|---|---|
| | Installation damage | Creep* | Chemical degradation | Biological degradation |
| separation | 1.1 to 2.5 | 1.0 to 1.2 | 1.0 to 1.5 | 1.0 to 1.2 |
| cushioning | 1.1 to 2.0 | 1.2 to 1.5 | 1.0 to 2.0 | 1.0 to 1.2 |
| unpaved roads | 1.1 to 2.0 | 1.5 to 2.5 | 1.0 to 1.5 | 1.0 to 1.2 |
| walls | 1.1 to 2.0 | 2.0 to 4.0 | 1.0 to 1.5 | 1.0 to 1.3 |
| embankments | 1.1 to 2.0 | 2.0 to 3.0 | 1.0 to 1.5 | 1.0 to 1.3 |
| bearing capacity | 1.1 to 2.0 | 2.0 to 4.0 | 1.0 to 1.5 | 1.0 to 1.3 |
| slope stabilization | 1.1 to 1.5 | 1.5 to 2.0 | 1.0 to 1.5 | 1.0 to 1.3 |
| pavement overlays | 1.1 to 1.5 | 1.0 to 1.2 | 1.0 to 1.5 | 1.0 to 1.1 |
| railroads | 1.5 to 3.0 | 1.0 to 1.5 | 1.5 to 2.0 | 1.0 to 1.2 |
| flexible forms | 1.1 to 1.5 | 1.5 to 3.0 | 1.0 to 1.5 | 1.0 to 1.1 |
| silt fences | 1.1 to 1.5 | 1.5 to 2.5 | 1.0 to 1.5 | 1.0 to 1.1 |

*The low end of the range refers to results that have been compensated for creep in the performance of the tests.

$$T_{\text{allow}} = T_{\text{ult}} \left( \frac{1}{FS_{ID} \times FS_{CR} \times FS_{CD} \times FS_{BD}} \right) \tag{2.18}$$

where

$T_{\text{allow}}$ = the allowable tensile strength,
$T_{\text{ult}}$ = the ultimate tensile strength,
$FS_{ID}$ = the factor of safety for installation damage,
$FS_{CR}$ = the factor of safety for creep,
$FS_{CD}$ = the factor of safety for chemical degradation, and
$FS_{BD}$ = the factor of safety for biological degradation.

Note that this equation could just as well have been formulated as fractional multipliers ($\leq 1.0$) and placed in the numerator of the equation. It is placed in this form because other studies have done likewise (e.g., Voskamp and Risseeuw [37]). While the equation indicates tensile strength, it can equally well be applied to burst strength, tear strength, puncture strength, seam strength, etc.

*For problems dealing with flow through or within a geotextile*, the formulation takes the following form with typical values given in Table 2.13. Note that these values must be tempered with site-specific conditions as was the case with the previous table.

$$q_{\text{allow}} = q_{\text{ult}} \left( \frac{1}{FS_{SCB} \times FS_{CR} \times FS_{IN} \times FS_{CC} \times FS_{BC}} \right) \tag{2.19}$$

# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 3/7/96 Subject Liner Stresses Sheet No. 7 of 20

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allowable Stresses of Geotextile

$\therefore$ Tallow = Tult / (2.5 × 1.2 × 1.5 × 1.2) , Tult = 150 lbs/in.

= 150/5.4 = 27.8 lbs/in

Allowable Stresses of Geonet

To avoid rupture of geonet, a factor of safety of 4 is used to calculate allowable stresses in geonet.

Tallow = Tult/4

= 50/4 = 12.5 lbs/in , Tult = 50 lbs/in

allowable Stresses of HDPE

To avoid rupture of HDPE, a factor of safety of 40 is used (See Ref. 10) to calculate allowable stresses in geomembrane. This factor of safety is considered very conservative.

Tallow = Tult/40

where Tult = 500 lbs/" for 100-mil HDPE and 400 lbs/" for 80-mil HDPE (see Ref. 11)

For 100-mil HDPE,
Tallow = 500/40 = 12.5 lbs/in

For 80-mil HDPE
Tallow = 400/40 = 10 lbs/in

allowable Stresses of Geocomposite

The geocomposite is composed of a geonet heat bonded to non-woven geotextile on both the upper and lower face of the geonet. As a result, the allowable stresses can be significantly higher than a single layer of geotextile or a geonet. For convenience, the allowable stresses of a geotextile can be used conservatively as the allowable stress of the geocomposite.



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Ref (10)

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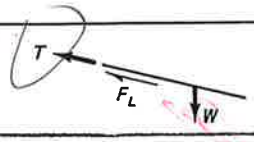
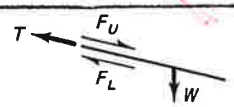
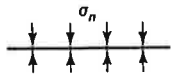
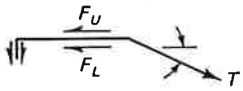
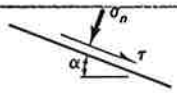
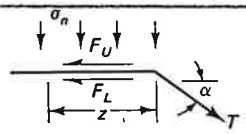
**TABLE 5.17** VARIOUS DESIGN MODELS FOR GEOMEMBRANES IN WASTE DISPOSAL SITUATIONS (REF: KOERNER AND RICHARDSON, 48)

| Problem | Liner stress | Free body diagram | Required properties: Geomembrane | Required properties: Landfill | Typical factor of safety |
|---|---|---|---|---|---|
| 1. liner self weight | tensile |  | $G, t, \sigma_{allow}, \delta_L$ | $\beta, H$ | 10 to 100 |
| 2. weight of filling | tensile |  | $t, \sigma_{allow}, \delta_U, \delta_L$ | $\beta, h, \gamma, H$ | 0.5 to 10 |
| 3. impact during construction | impact | | $I$ | $d, w$ | 0.1 to 5 |
| 4. weight of landfill | compression |  | $\sigma_{allow}$ | $\gamma, H$ | 10 to 50 |
| 5. puncture | puncture | | $\sigma_p$ | $\gamma, H, P, A_p$ | 0.5 to 10 |
| 6. anchorage | tensile |  | $t, \sigma_{allow}, \delta_U, \delta_L$ | $\beta, \gamma, \phi$ | 0.7 to 5 |
| 7. settlement of landfill | shear |  | $\tau, \delta_U$ | $\beta, \gamma, H$ | 10 to 100 |
| 8. subsidence under landfill | tensile |  | $t, \sigma_{allow}, \delta_U, \delta_L, \chi$ | $\alpha, \gamma, H$ | 0.3 to 10 |

Notes:

Geomembrane properties

- $G$ = specific gravity
- $t$ = thickness
- $\sigma_{allow}$ = allowable stress (yield or break)
- $\tau$ = shear stress
- $I$ = impact energy
- $\sigma_p$ = puncture stress
- $\delta_U$ = friction with material above
- $\delta_L$ = friction with material below
- $\chi$ = mobilization distance

Landfill properties

- $\beta$ = slope angle
- $H$ = height
- $\gamma$ = unit weight
- $h$ = lift height
- $\alpha$ = subsidence angle
- $\phi$ = friction angle
- $d$ = drop height
- $W$ = weight
- $p$ = puncture force
- $A_p$ = puncture area

# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 3/7/96 Subject Liner Stresses Sheet No. 9 of 20

Chkd. By ___ Date ___ Proj. No. 95-284

For this analysis, an allowable stress of 27.8 lbs/in is used for geocomposite.

Allowable Stresses of GCL

To avoid rupture of GCL, a conservative factor of safety of 10 is used in this calculation.

$$T_{allow} = T_{ult}/10 = 45/10 = 4.5 \text{ lbs/in}$$

3.0 Calculations (see Tables 1, 2, and 3)



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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 3/6/96 Subject Liner Stresses Sheet No. 10 of 20

Chkd. By WB Date 3/7/96 3.0 Calculations Proj. No. 95-284

Table 1

| COMPONENT | UNIT WEIGHT (lbs/ft²) | Tensile Load Due To Self WT (lbs/ft)[2] | Strain Due To Self Weight, $\varepsilon_1$ (%) | Strain Due To Thermal Expansion $\varepsilon_2$ (%) | Strain Due To Seismic Deformation $\varepsilon_3$ (%) | Strain Due To Settlement of Waste Fill $\varepsilon_4$ (%) | Total Strain $\varepsilon_T$ (%) | Total Induced Stresses (lbs/ft) | Tult (lbs/ft) | Tyield (lbs/ft) | Tallow (lbs/ft) | Ref. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Non-Woven GEOTEXTILE (TREVIRA SPUNBOND 1125) | 0.052 | 0.3 | 0[7] | 0.5 | 0.2 | 4 | 4.7 | 12[3] | 150 | NR[1] | 27.8 | 9 |
| GEONET (POLY-NET PN 300) | 0.18 | 1.1 | 0.51[4] | 0.5 | 0.2 | 4 | 5.2 | 11.3[4] | NA[9] | 50[8] | 12.5 | 10,14 |
| 100-mil HDPE (Textured Hyperflex) | 0.54 | 2.6 | 0.16[5] | 0.5 | 0 | 0 | 0.66 | 10.6[5] | 500 | 240 | 12.5 | 11 |
| GEOCOMPOSITE (TEXTNET TN3002/1125) | 0.26 | 1.3 | 0[7] | 0 | 0 | 0 | 0 | 0 | 150 | NR | 27.8 | 9 |
| 80-mil HDPE (Textured Hyperflex) | 0.44 | 2.6 | 0.2[5] | 0.5 | 0 | 0 | 0.7 | 9[5] | 400 | 192 | 10 | 11 |
| GCL (CLAYMAX 500SP) | 1.2 | 2.4 | 0.8[6] | 0 | 0 | 0 | 0.8 | 2.4[6] | NA | 45[10] | 4.5 | 2 |

NOTES

(1) "NR" represents Not Reported. Stress-strain curve does not indicate a pronounced yield point.

(2) See Calculations in Table 2.

(3) See stress-strain curve, Ref 9.

(4) Calculated using an assumed elongation at yield of about 23% (Ref. 10)

(5) Calculated using a tensile elongation at yield of 15% (Ref. 11)

(6) Calculated using tensile elongation at rupture of 15% (Ref. 2)

(7) See Stress-strain curve attached (Ref 9).

(8) Ref. 14

(9) Not Available.

(10) Calculated using the GCL grab strength of 180 lbs (Ref. 2) devided by the width (4") of the test specimen, 180/4 = 45 lbs/in.



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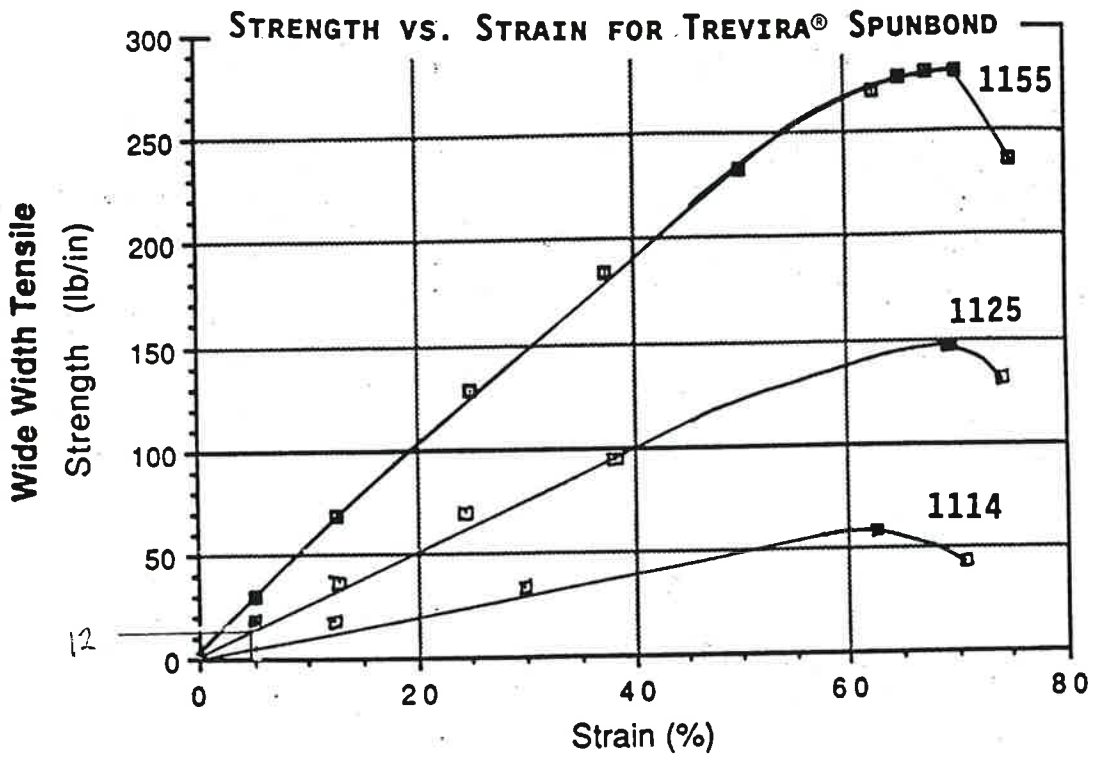
Ref. 9

11 /20

TREVIRA SPUNBOND

## ~~TABLE 1~~: STRENGTH CHARACTERISTICS OF TREVIRA® SPUNBOND GEOTEXTILES

| | CBR Puncture Strength (lb) | CBR Puncture Elongation (in) | Measured Wide Width Tensile (lb/ft) | | | Measured Wide Width Strain % | | |
|---|---|---|---|---|---|---|---|---|
| | | | Machine | Cross-M | Average | Machine | Cross-M | Average |
| Trevira 1114 | 341 | 1.9 | 674 | 529 | 602 | 63% | 61% | 62% |
| Trevira 1125 | 701 | 1.95 | 1770 | 1450 | 1610 | 69% | 70% | 70% |
| Trevira 1155 | 1420 | 1.99 | 3360 | 2670 | 3015 | 68% | 77% | 73% |




## ~~FIGURE 1~~: WIDE WIDTH TENSILE STRENGTH/STRAIN CURVES FOR TREVIRA® SPUNBOND GEOTEXTILES (MACHINE DIRECTION)

Ref (10)

12/20



## 4.1 GEONET PROPERTIES AND TEST METHODS

Since the primary function of a geonet is to convey liquid within the plane of its structure, the in-plane hydraulic flow rate, or transmissivity, is of major importance. However, other features, which may influence this value over the service lifetime of the geonet, are also of importance. Thus a number of properties will be presented in this section.

### 4.1.1 Physical Properties

The primary physical property needed to characterize a geonet is its thickness, which can be determined using ASTM D1777. In this test, it is recommended that geotextile thickness be measured under a normal pressure of 42 lb./ft.$^2$ (2.0 kPa). There are no current guidelines for geonets. The pressures used in obtaining the values listed in Table 4.1 are not known, but it should be noted that geonets are not nearly as sensitive to thickness variation under normal pressure as are geotextiles.

Mass per unit area can be determined using ASTM D1910. For a 200-mil (5.1-mm)-thick, extruded, solid-rib geonet, the mass per unit area is usually in the range of 20 to 30 oz./yd.$^2$ (680 to 1020 g/m$^2$). It is not a design property per se, but it is informative from a manufacturer's point of view.

Other physical properties such as rib dimensions, planar angles made by the intersecting ribs, vertical angles made at the juncture point, aperture size and shape, color, hardness, etc., can be measured directly and are relatively straightforward to obtain.

### 4.1.2 Mechanical Properties

A number of strength aspects of geonets are important to consider.

#### 4.1.2.1 Tensile Strength

The wide-strip tensile properties of geonets are sometimes called for in various specifications. Using ASTM D4595 which uses a test specimen 8.0 in.(200 mm) in width by 4.0 in. (100 mm) in length, the curves of Figure 4.4 resulted for a 200-mil(5.1-mm) thick solid-rib geonet in the machine and cross machine directions. Note the differences in behavior, suggesting that there is a preferential direction in strength between the machine and cross-machine directions. If site-specific conditions warrant a high-strength direction, the geonet should be oriented with its machine direction positioned accordingly. Also to be noted is that geonets do indeed possess a measurable tensile strength. For the solid-rib geonet illustrated in Figure 4.1 and shown in Figure 4.4, the average of a series of wide-strip tensile tests gave the following information:

- machine direction:
  Peak strength = 680 lb./ft. (120 kN/m) = 680/12 lb./in = 56.7 lbs/in
  Strain at peak = 23%
- Strain at failure = 290%

Ref (10)

13/20



- cross-machine direction:
  Peak strength = 390 lb./ft. (68 kN/m)
  Strain at peak = 170%
  Strain at failure = 240%

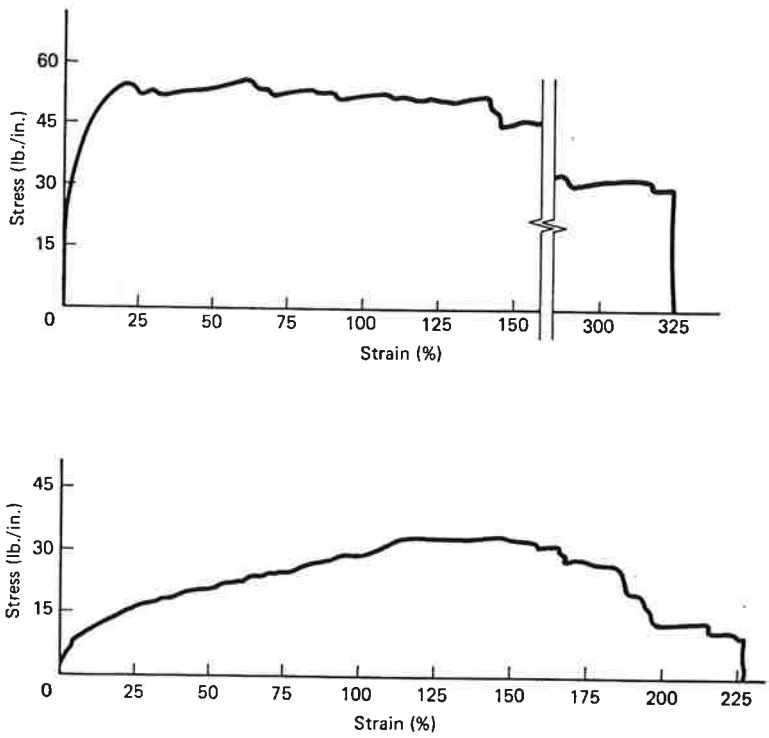



**Figure 4.4.** Tensile strength behavior of a 200-mil (5.1-mm) solid-rib extruded geonet. Upper curve, machine direction; lower curve, cross-machine direction.

### 4.1.2.2 Compressive Strength

Of greater importance than the above described in-plane tensile strength of geonets is the cross-plane compressive strength. This is due to the influence that compressive yielding or collapse has on the ability of the geonet to conduct liquid within its open spaces. There are a number of approaches to measuring a geonet's compressive strength, and the ASTM has a task commitee working on the topic. Using 6.0-in. (150-mm)-square test specimens normally loaded under a constant strain rate load of 0.002 in./min. (= 0.05 mm/min.), the curves of Figure 4.5 resulted. The upper figure illustrates deformation behavior to applied normal stress, while the lower figure presents strain versus applied normal stress. Here it is seen that both the foamed rib extruded geonet and the solid rib extruded geonet are initially very stiff but begin to deform at approximately 15,000 lb./ft.$^2$ ($\cong$ 720 kPa). The solid extruded ribs deform into the valleys created by their intersecting

# HDPE DRAINAGE NET — GEOTEXTILE GEOCOMPOSITE

14/20

## Recommended TEX-NET® Specifications

Ref. (14)

## 1. DESCRIPTION

These specifications describe a high density polyethylene (HDPE) drainage net - geotextile geocomposite. The supply and installation of this material shall be in strict accordance with these specifications and contract drawings.

## 2. MATERIALS AND PRODUCT CONSTRUCTION

The geocomposite drainage material shall be TEX-NET® by Fluid Systems, Inc. and shall meet or exceed the following criteria:

### 2.01

Drainage net shall be manufactured by extruding two sets of strands to form a three (3) dimensional structure to provide planar water flow.

The drainage net shall contain stabilizers to prevent ultraviolet light degradation.

### 2.02

The drainage net shall be POLY-NET® PN-3000 or approved equal. In addition to the material properties listed above, the drainage net shall conform to the following properties detailed in the table below.

**POLY-NET® PN-3000**

| PROPERTY | TEST METHOD | UNIT | QUALIFIER | VALUE |
|---|---|---|---|---|
| POLYMER DENSITY | ASTM D-1505 | g/cm³ | range | .937 ± .002 |
| POLYMER MELT INDEX | ASTM D-1238 | g/10 min | maximum | 1.0 |
| CARBON BLACK CONTENT | ASTM D-1603 | % | range | 2-3% |
| NOMINAL THICKNESS | ASTM D-1777 | inches | range | 0.220 ± 0.022 |
| NOMINAL MASS PER UNIT AREA | ASTM D-3776 | lbs/1000 ft² | range | 180 ± 18 |
| TRANSMISSIVITY at 15,000 psf | *ASTM D-4716 | m²/sec | minimum | $1 \times 10^{-3}$ |
| TENSILE STRENGTH MACHINE DIRECTION | ASTM D-1682 | lbs/in | range | 50 ± 10 |

*Per ASTM D4716-87. The transmissivity was measured using water @ 20°C (68°F) with a gradient of one, between two steel plates, after one hour. Value may vary, based on dimensions of the transmissivity specimen and specific laboratory.

### 2.03

The geotextile filter fabric shall meet or exceed the following:

**TREVIRA® 1120 OR EQUAL MEETING THE PROPERTIES LISTED BELOW**

| FABRIC PROPERTY | UNIT | TEST METHOD | *MINIMUM REQUIREMENT |
|---|---|---|---|
| FABRIC WEIGHT | oz/yd² | ASTM D-3776 | 5.7 |
| THICKNESS, t | mils | ASTM D-1777 | 75 |
| GRAB STRENGTH | lbs | ASTM D-4632 | 160 |
| GRAB ELONGATION | % | ASTM D-4632 | 60 |
| TRAPEZOID TEAR STRENGTH | lbs | ASTM D-4533 | 60 |
| PUNCTURE RESISTANCE | lbs | ASTM D-4833 | 80 |
| MULLEN BURST STRENGTH | psi | ASTM D-3786 | 275 |
| WATER FLOW RATE | gpm/ft² | ASTM D-4491 | 130 |
| PERMITTIVITY, $\Psi$ | sec⁻¹ | ASTM D-4491 | 1.74 |
| PERMEABILITY, $k = \Psi t$ | cm/sec | ASTM D-4491 | .33 |
| AOS | Sieve Size mm | ASTM D-4751 | 70 .210 |

*These minimum values represent minimum test values determined from Q.C. testing on all lots produced in 1989.

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Ref (11) 15/20

# FrictionFlex® Textured HyperFlex® Lining Material

SLT's FrictionFlex patented texturing process provides the industry's leading textured liners. Uniquely produced from a specially formulated virgin HDPE geomembrane resin, **FrictionFlex** has outstanding chemical resistance, mechanical properties, environmental stress crack resistance, dimensional stability and thermal aging characteristics. **HyperFlex**, textured with the **FrictionFlex** process, contains approximately 97.5% polymer and 2.5% carbon black, anti-oxidants and heat stabilizers, and contains no additives, fillers or extenders. SLT textured liners have excellent resistance to UV radiation and are suitable for exposed conditions.

| PROPERTY | TEST METHOD | NOMINAL VALUES | | | |
|---|---|---|---|---|---|
| Thickness, mils | ASTM D751/374/1593 | 40 | 60 | 80 | 100 |
| Density (g/cc) | ASTM D792/1505 | 0.944 | 0.944 | 0.944 | 0.944 |
| Melt Flow Index (g/10 minutes) | ASTM D1238-E | ≤ 1.0 | ≤ 1.0 | ≤ 1.0 | ≤ 1.0 |
| Tensile Properties Either Direction | ASTM D638 Type IV Dumbell, 2 ipm Gauge length per N.S.F. Std. 54 | | | | |
| Tensile Strength at Break (lbs/in, width) | | 200 | 300 | 400 | 500 |
| Tensile Strength at Yield (lbs/in, width) | | 96 | 144 | 192 | 240 |
| Elongation at Break (percent) | | 600 | 600 | 600 | 600 |
| Elongation at Yield (percent) | | 15 | 15 | 15 | 15 |
| Tear Resistance Initiation (lbs) | ASTM D1004 Die C | 33 | 50 | 66 | 83 |
| Low Temperature Brittleness | ASTM D746 B | <-120°F | <-120°F | <-120°F | <-120°F |
| Dimensional Stability Each Direction (percent) | ASTM D1204 248°F 1 hr. | ± 1 | ± 1 | ± 1 | ± 1 |
| Ozone Resistance | ASTM D1149 7 days 100 ppm 104°F | No Cracks | No Cracks | No Cracks | No Cracks |
| Environmental Stress Crack Resistance (hrs) | ASTM D1693 Cond. C | >2000 | > 2000 | > 2000 | > 2000 |
| Puncture Resistance (lbs) | FTMS 101C Method 2065 | 60 | 90 | 120 | 150 |
| Water Absorption (percent weight change) | ASTM D570 | <0.01 | <0.01 | <0.01 | <0.01 |
| Moisture Vapor Transmission (g/m²day) | ASTM E96 | <0.001 | <0.001 | <0.0009 | <0.00085 |
| Oxidative Induction Time (minutes) Pure $O_2$ at 1 Atmosphere | ASTM D3895 200°C | 100 | 100 | 100 | 100 |
| Tensile Impact Strength (ft-lbs/in²) | ASTM D1822 | 381 | 381 | 381 | 381 |
| Carbon Black Content (percent) | ASTM D1603 | 2-3 | 2-3 | 2-3 | 2-3 |
| Carbon Black Dispersion | ASTM D3015 | A1/A2 | A1/A2 | A1/A2 | A1/A2 |

**SLT FrictionFlex** is available in 24-foot widths and up to 8,000 lb. rolls. Special length rolls and other material thicknesses are available upon request.

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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 3/6/96 Subject Liner Stresses Sheet No. 17 of 20

Chkd. By MB Date 3/7/96 Proj. No. 95-284

Table 2

| material | Self weight (1) (lb/ft) | Accumulated weight (lb/ft) | $F_u$ (2) (lb/ft) | $F_L$ (2) (lb/ft) | Tensile Load Due To Gravity (3) (lbs/") |
|---|---|---|---|---|---|
| Geotextile | 4.4 | 4.4 | 0 | 0.7 | 0.3 |
| Geonet | 15.1 | 19.5 | 0.7 | 1.2 | 1.1 |
| 100-mil HDPE | 45.4 | 63.9 | 1.2 | 11 | 2.6 |
| Geocomposite | 21.8 | 85.7 | 11 | 14.7 | 1.3 |
| 80-mil HDPE | 37 | 122.7 | 14.7 | 16.8 | 2.6 |
| GCL | 100.8 | 223.5 | 16.8 | 78.2 | 2.4 |

NOTES

(1) Self Weight = unit weight (in lbs/ft$^2$) * Slope Length

Slope Length = 84 feet.

unit weight is listed in Table 1.

(2) Friction Forces ($F_u$ & $F_L$) are calculated as follows:

$$F = \text{Accumulated Wt.} \times \cos 63.4 \times \tan \delta$$

where $\delta$ = residual interface friction angle (see Table 3).

(3) Tensile Load Due To Gravity (in lbs/") $= (\text{Self Wt.} * \sin 63.4 + F_u - F_L)/12''$



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By JH Date 3/6/96 Subject Liner stresses Sheet No. 18 of 20

Chkd. By WB Date 3/7/96 Proj. No. 95-284

Table 3

| Interface No. | Residual Friction angle (degree) | Reference |
|---|---|---|
| 2 | 19 | 1 |
| 3 | 8 | 1 |
| 4 | 21 | 1 |
| 5 | 21 | 1 |
| 6 | 17 | 2 |
| 7 | 38 | 2 |

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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 3/6/96 Subject Liner Stresses Sheet No. 19 of 20

Chkd. By WB Date 3/7/96 Proj. No. 95-286

4.0 CONCLUSIONS

1. The outer layers of the liner system will withstand the relatively large strains (up to ≈ 4.7%) primarily caused by waste fill settlement. Settlement induced stresses will occur gradually as the waste fill is being placed. This slow loading process will allow the upper materials (geotextile and geonet) to deform without inducing significant stresses. The calculated stresses are conservative and are within the allowable limits. This slow load application, creep relaxation of stresses should be measured in the laboratory to verify design parameters (see item 6).

2. The design slip plane between geonet and the smooth side of the 100-mil HDPE will prevent the settlement induced strains (and stresses) from being transferred to the lower layers of the liner system.

3. Self weight of liner components are not an issue. Up to one percent of strain can occur in the GCL material. The calculated stress in GCL is less than the allowable stress.

4. Thermal stresses are negligible. Based on our experience, we do not expect wrinkles to occur in the liner system when textured HDPE is used. However, in case minor wrinkles do occur, care should be taken to avoid placing fill (soil or waste) over wrinkles such that thermal stresses (and strains) are "locked in" and thus could accumulate from the bottom to the top.

5. Wind uplift is balanced by using sand bags and suction devices, and are not included in the liner stress calculations.

6. Actual materials used in construction should be tested for stress-strain properties and interface shear strength parameters.



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By JH Date 2/9/96 Subject Tension / Elongation Sheet No. 20 of 20
Chkd. By KB Date 2/19/96 Proj. No. 95-284

5.0 References


1. ESI, Internal Data Base.

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8. ESI, February, 1996, Calculations, Post-closure Waste Settlements.

9. Hoechst Celanese Corp, Catalog, Trevira SpunBond, 1990.

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12. This reference intentionally left blank.

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14. Fluid Systems, Inc, Catalog, 1991.




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# C.6 ANCHOR TRENCH

# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 2/11/96 Subject Anchor Trench Sheet No. 0 of ____

Chkd. By HB Date 2/19/96 Proj. No. 95-284

ANCHOR TRENCH DESIGN

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By JH Date 2/11/96 Subject Anchor Trench Sheet No. 1 of 10

Chkd. By [illegible] Date 2/19/96 Proj. No. 95-284

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| 2.0 | Approach | 2 |
| 3.0 | Calculations | 5 |
| 4.0 | Summary | 8 |
| 5.0 | References | 10 |
| FIGURE 1 | | 3 |
| TABLE 1 | | 9 |




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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 2/16/96 Subject Anchor Trench Sheet No. 2 of 10

Chkd. By WB Date 2/19/96 Proj. No. 95-284

1.0 Purpose Determine required trench dimensions.

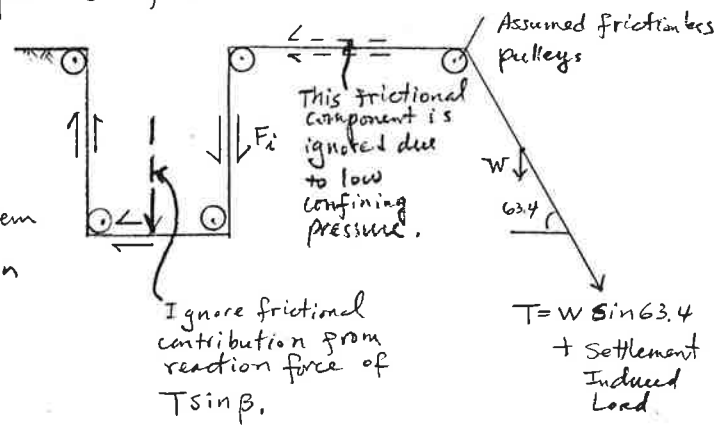
2.0 Approach

- Assume tension in liner system is taken by the sum of frictional resistance in anchor trench.

- From the model shown below, frictionless pulleys are assumed at angle points to simplify the analysis (Ref. 5).

- Downdrag Load
  T = Liner Stresses (Ref. 12)

- $F_i$ = resistance along soil/liner system interfaces (see Fig. 1 for design geometry).




MODEL

- Trench sizes are selected such that

$$\Sigma F_i \geq T \cdot F.S.$$

where F.S. is a factor of safety selected to be 4.

- The integrity of the liner components are checked in Ref. 12 with sufficient factor of safety against rupture.

- A 30-mil sacraficial HDPE membrane will be placed over the non-woven geotextile (see Figure 1) and will be anchored in Zone 1. However, this sacraficial membrane will pose very minor load. The unit weight is 0.11 lbs/ft², or 9.24 lbs/ft for 84-ft-long, 1-ft-wide strip. The downdrag load is

$$T = W \sin 63.4 = 9.24 \times \sin 63.4 = 8.26 \text{ lbs/ft} \doteq 0.7 \text{ lbs/in.}$$

Because of the negligibe load and embedment length of 10 feet, the layer will be adequately anchored.



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**ENVIRONMENTAL SOLUTIONS, INC.**

By JH Date 2/11/96 Subject ANCHOR TRENCH Sheet No. 3 of
Chkd. By WB Date 2/18/96 Figure 1 Proj. No. 95-284

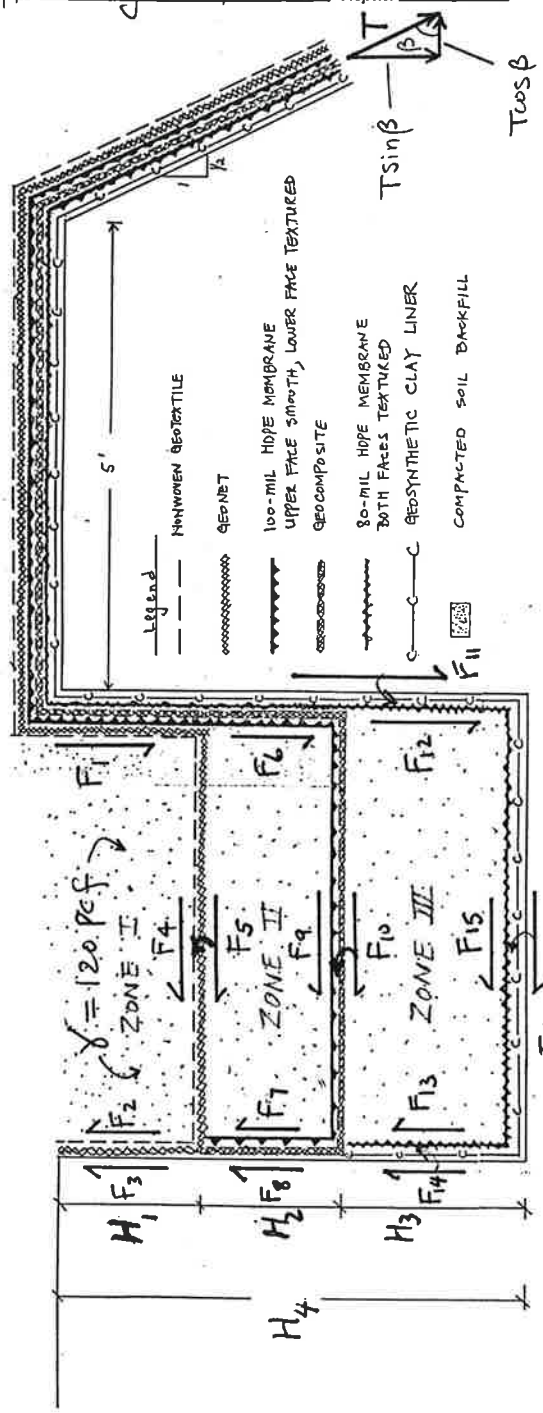


Figure 1

Vertical Frictional Component $= \frac{1}{2}\gamma H_i^2 K_o \tan\delta + a H_i$

Horizontal Frictional Component $= (\sigma_v \tan\delta + a) L$

$\delta$ = interface friction angle

$a$ = interface adhesion

$\sigma_v$ = vertical stress due to overburden.

$L = 10'$
$H_1 = 1'$
$H_2 = 1'$
$H_3 = 1'$
$H_4 = 3'$

# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 2/16/96 Subject Anchor Trench Sheet No. 4 of 10

Chkd. By WB Date 2/19/96 Proj. No. 95-284

2.0 Approach (cont'd)

- From Ref. 12, the total induced liner stresses due to gravity load, thermal expansion/contraction, seismic deformation, and settlement of waste fill are as follows for the liner components:

| Liner Components | Liner stresses (lbs/") | |
|---|---|---|
| geotextile | 12 | Zone 1 |
| geonet | 11.3 | |
| 100-mil HDPE | 10.6 | Zone 2 |
| Geocomposite | 0 | |
| 80-mil HDPE | 9 | Zone 3 |
| GCL | 2.4 | |

Let anchor capacity be at least 4 times the total liner stresses in each zone.

| Zone | Total Liner stresses (lbs/") | minimum Required Anchor Capacity (lbs/") |
|---|---|---|
| 1 | 12+11.3 = 23.3 | 23.3×4 = 93.2 |
| 2 | 10.6 | 10.6×4 = 42.4 |
| 3 | 9+2.4 = 11.4 | 11.4×4 = 45.6 |

The anchor capacity is also designed to exceed the yield strength of the stronger material in each zone such that the liner components will yield before being pulled out.



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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 2/11/96 Subject Anchor Trench Sheet No. 5 of 10

Chkd. By WB Date 2/19/96 Proj. No. 95-284

3.0 Calculations

Zone 1, Geotextile and Geonet Anchor Length

Min. Required anchor capacity is 93.2 lbs/ft

or

$$93.2 \times 12 = 1118.4 \text{ lbs/ft}$$

From Figure 1, $\Sigma F_i = F_1 + F_2 + F_3 + F_4 + F_5$

where $F_1 = F_2$ = Frictional resistance between sand and geotextile

$= \frac{1}{2} \gamma H_1^2 K_o \tan 21 + 25 H_1$ ($K_o = 0.5$ assumed, $\gamma = 120$ pcf)

$= 11.5 H_1^2 + 25 H_1$

$F_3$ = Frictional resistance between geonet and sand

$= \frac{1}{2} \gamma H_1^2 K_o \tan 30$

$= 17.3 H_1^2$

$F_4$ = Frictional resistance between geotextile and sand

$= (\gamma H_1 \tan 21 + 25) L$

$= (46 H_1 + 25) L$

$F_5$ = Frictional resistance between geotextile and geonet

$= (\gamma H_1 \tan 19 + 65) L = (41.3 H_1 + 65) L$

Let $H_1 = 1'$, $L = 10' \Rightarrow \Sigma F_i = (2 \times 36.5) + 17.3 + 710 + 1063 = 1863$ lbs/ft $> 1118$ lbs/ft (O.K.)

The yield strength of Trevira 1125 is 150 lbs/in $= 1800$ lbs/ft. $< 1863$ lbs/ft (O.K.)



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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 2/18/96 Subject Anchor Trench Sheet No. 6 of 10

Chkd. By WB Date 3/5/96 Proj. No. 95-284

Zone 2 100 mil HDPE and Geocomposite Anchor Length

Min. Required anchor capacity is 42.4 lbs/" or

$42.4 \times 12 = 509$ lbs/ft

From Figure 1, $\Sigma F_i = F_6 + F_7 + F_8 + F_9 + F_{10}$

where $F_6 = F_7 =$ Friction resistance between sand and smooth side of HDPE.

$= \frac{1}{2}\gamma\left(H_1 + \frac{H_2}{2}\right)^2 K_o \tan 18$

$= 9.75\left(1 + \frac{H_2}{2}\right)^2$

$F_8 = \frac{1}{2}\gamma\left(H_1 + \frac{H_2}{2}\right)^2 K_o \tan 21 + a H_2$ (Geocomposite / Sand)

$= 11.5\left(1 + \frac{H_2}{2}\right)^2 + 25 H_2$

$F_9 = \gamma(H_1 + H_2)\tan 18 \cdot L$ (smooth HDPE / sand)

$= 39(1 + H_2)L$

$F_{10} = [\gamma(H_1 + H_2)\tan 7 + 280] \cdot L$

$= [14.7(1 + H_2) + 280]L$

Let $H_2 = 1'$, $L = 10'$

$\Rightarrow \Sigma F_i = 22 \times 2 + 51 + 780 + 3094 = 3969 \gg 509$ lbs/ft, O.K.

The yield strength of the 100-mil HDPE is 240* lbs/" = 2880 lbs/ft which is less than the anchor capacity, O.K.

* From Ref. 4

# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 2/18/96 Subject Anchor Trench Sheet No. 7 of 10

Chkd. By WB Date 2/19/96 Proj. No. 95-284

Zone 3 80 mil HDPE and GCL

Min. Required anchor capacity is 45.6 lbs/ft

or $45.6 \times 12 = 547$ lbs/ft.

From Figure 1, $\Sigma F_i = F_{11} + F_{12} + \cdots + F_{16}$

where $F_{11}$ = GCL internal shear strength

$$= \frac{1}{2}\gamma\left(\frac{2+H_3}{2}\right)^2 K_o \tan 5 + 550\cdot(2+H_3)$$

$$= 2.6\left(\frac{2+H_3}{2}\right)^2 + 550(2+H_3)$$

$$F_{12} = F_{13} = \frac{1}{2}\gamma\left(2+\frac{H_3}{2}\right)^2 K_o \tan 26 \qquad \text{(textured HDPE/Sand)}$$

$$= 14.6\left(2+\frac{H_3}{2}\right)^2$$

$$F_{14} = \frac{1}{2}\gamma\left(2+\frac{H_3}{2}\right)^2 K_o \tan 5 + 550\, H_3 \qquad \text{(GCL Internal)}$$

$$= 2.6\left(2+\frac{H_3}{2}\right)^2 + 550\, H_3$$

$$F_{15} = \gamma(2+H_3)\tan 26 \cdot L \qquad \text{(textured HDPE/Sand)}$$

$$= 58.5(2+H_3)L$$

$$F_{16} = [\gamma(2+H_3)\tan 5 + 550]L \qquad \text{(GCL internal)}$$

$$= [10.5(2+H_3)+550]L$$

Let $H_3 = 1'$, $L = 10'$ $\Rightarrow \Sigma F_i = 1656 + 2\times 32 + 566 + 1755 + 5815$

$= 9856$ lbs/ft $\gg 547$ lbs/ft, O.K.

The yield strength of the 80-mil HDPE is 192 lbs/ft (Ref. 4) which is $192 \times 12 = 2304$ lbs/ft. ($\ll 9856$ lbs/ft, O.K.)



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# ENVIRONMENTAL SOLUTIONS, INC.

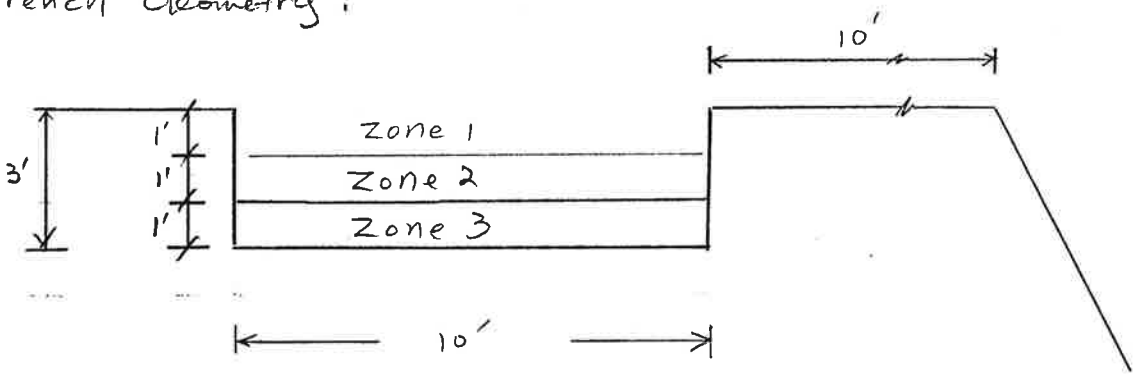
By JH Date 2/18/96 Subject Anchor Trench Sheet No. 8 of 10

Chkd. By KB Date 2/19/96 Proj. No. 95-284

## 4.0 Summary

- 10' wide trench is required to accomodate the downdrag load from zone 1. The anchor trench is designed such that the liner components will yield before being pulled out.

- Anchor Trench Geometry:




- Design interface, friction angle, adhesion, and equations are summarized in Table 1.

- Actual materials used in construction should be tested to obtain stress-strain properties and interface shear strength parameters.

- a 30-mil sacraficial HDPE membrane will be placed over the non-woven geotextile and will be anchored in Zone 1. Considering the negligible load applied to the anchor trench by the weight of this membrane, the inclusion of this sacraficial membrane in the anchor trench will have no effect to trench dimensions.



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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 2/11/96 Subject Anchor Trench Sheet No. 9 of 10

Chkd. By MB Date 2/19/96 Table 1 Proj. No. 95-284

| Friction Force No. | Interface | Interface Friction angle (degree) | Interface Adhesion (psf) | Reference No. | Equation (1),(2) |
|---|---|---|---|---|---|
| $F_1, F_2$ | Geotextile / Sand | 21 | 25 | 1 | $11.5\,H_1^2 + 25\,H_1$ |
| $F_3$ | Geonet / Sand | 30 | 0 | Assumed | $17.3\,H_1^2$ |
| $F_4$ | Geotextile / Sand | 21 | 25 | 1 | $(46\,H_1 + 25)\,L$ |
| $F_5$ | Geotextile / geonet | 19 | 65 | 1 | $(41.3\,H_1 + 65)\,L$ |
| $F_6, F_7$ | Smooth HDPE / Sand | 18 | 0 | 2 | $9.75\,(1 + \frac{H_2}{2})^2$ |
| $F_8$ | Geocomposite / Sand | 21 | 25 | 1 | $11.5\,(1 + \frac{H_2}{2})^2 + 25\,H_2$ |
| $F_9$ | Smooth HDPE / Sand | 18 | 0 | 2 | $39\,(1 + H_2)\,L$ |
| $F_{10}$ | Textured HDPE / Geocomposite | 7 | 280 | 1 | $[14.7\,(1 + H_2) + 280]\,L$ |
| $F_{11}$ | GCL internal | 5 | 550 | 9, 10 | $2.6\,(\frac{3 + H_3}{2})^2 + 550\,(3 + H_3)$ |
| $F_{12}, F_{13}$ | Textured HDPE / Sand | 26 | 0 | 4 | $14.6\,(3 + \frac{H_3}{2})^2$ |
| $F_{14}$ | GCL internal | 5 | 550 | 9, 10 | $2.6\,(3 + \frac{H_3}{2})^2 + 550\,H_3$ |
| $F_{15}$ | Textured HDPE / Sand | 26 | 0 | 4 | $58.5\,(3 + H_3)\,L$ |
| $F_{16}$ | GCL internal (4) | 5 | 550 | 9, 10 | $[10.5\,(3 + H_3) + 550] \cdot L$ |

Notes: (1) From Ref. 5 $\Rightarrow T = \Sigma F_i$ (3) $a$ = adhesion, $\delta$ = friction angle.
(2) $H_i$ = Depth of Embedment, $L$ = Trench Width. (4) Assumed shearing occur at internal face of GCL.



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# ENVIRONMENTAL SOLUTIONS, INC.

By JH Date 2/11/96 Subject _____ Sheet No. 10 of 10

Chkd. By WB Date 2/19/96 _____ Proj. No. 95-284

5.0 References


1. ESI, Internal Data Base.

2. Mitchell, Seed, Kettleman Hills Waste Landfill Slope Failure. I: Liner System Properties. Vol 116, No. 4, April, 1990

3. Geosyntec Consultants, Final Report, Interface Direct Shear Testing, Select Claymax Shear-Pro Bentonite Composite Interfaces, Prepared for James Clem Corp., 24 Nov 92.

4. Gundle, Technical Data Sheet, 1995.

5. Koerner, Designing with Geosynthetics, 3rd Ed.

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7. Hoechst Celanese Corp., Technical Data, 1990.

8. Fluid Systems, Inc., Catalog, 1991.

9. CETCO, Technical Data sheet, Oct-13-1995.

10. CETCO, Technical Note No. 5, Oct, 1995

11. Geosyntec Consultants, Geomechanics and Environmental Laboratory, Final Report, Interface Direct Shear Testing, Select Claymax Shearpro Bentonite Composite Interfaces, Prepared for James Clem Corp, Nov. 24, 1992.

12. ESI, March, 1996, Calculations, Liner Stresses.




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# C.7 SUMP AND RISER DESIGN

# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/24/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 1 of 22

Chkd. By EC Date 2/13/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284

PURPOSE: Design of the LCRS & LDS riser systems.

METHOD:

1) Prepare preliminary layouts of riser systems
2) Identify stresses & loads on pipe systems.
3) Evaluate ability of systems to accomodate stresses
   - Wall Crushing
   - Buckling
   - Downdrag
   - Deflection
   - Longitudinal Stresses
   - Shear Stresses.
4) Provide Preliminary Design of foundation blocks.
5) Evaluate stresses on foundation block - ckt. by structural eng.
6) Evaluate stresses on cell foundation.



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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/24/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 2 of 22

Chkd. By EC Date 2/13/96 AMERICAN ECOLOGY CORP Proj. No. 95-284

ANALYSIS

1) Shown below is the typical section of the sumps & sidewall

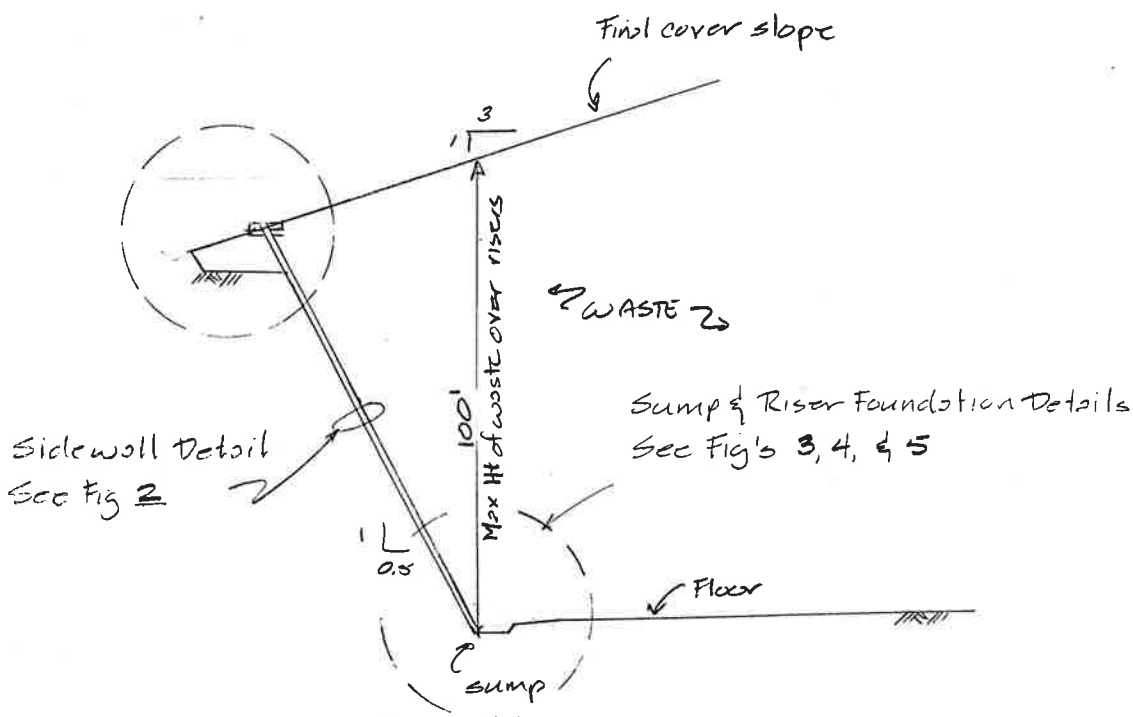


FIGURE 1
Typical Section of Cell 12

2) The purpose of the risers is to provide access to the sumps for removing liquids. Stresses resulting from the riser system that are transferred to the liner system must be accounted for in the liner design. Details of this system are shown in the following figures



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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/25/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 3 of 22

Chkd. By EC Date 2/13/96 AMERICAN ECOLOGY CORP. Proj. No. 95-234

ANALYSIS (CONT.)

3) The LCRS riser pipe will be installed directly on the 0.5:1 slope over the Primary Liner System. The LDS riser pipe will be installed in a trench cut into the side wall. The LDS riser pipe will be above the Secondary Liner System, but below the Primary Liner System (see Figure 2).

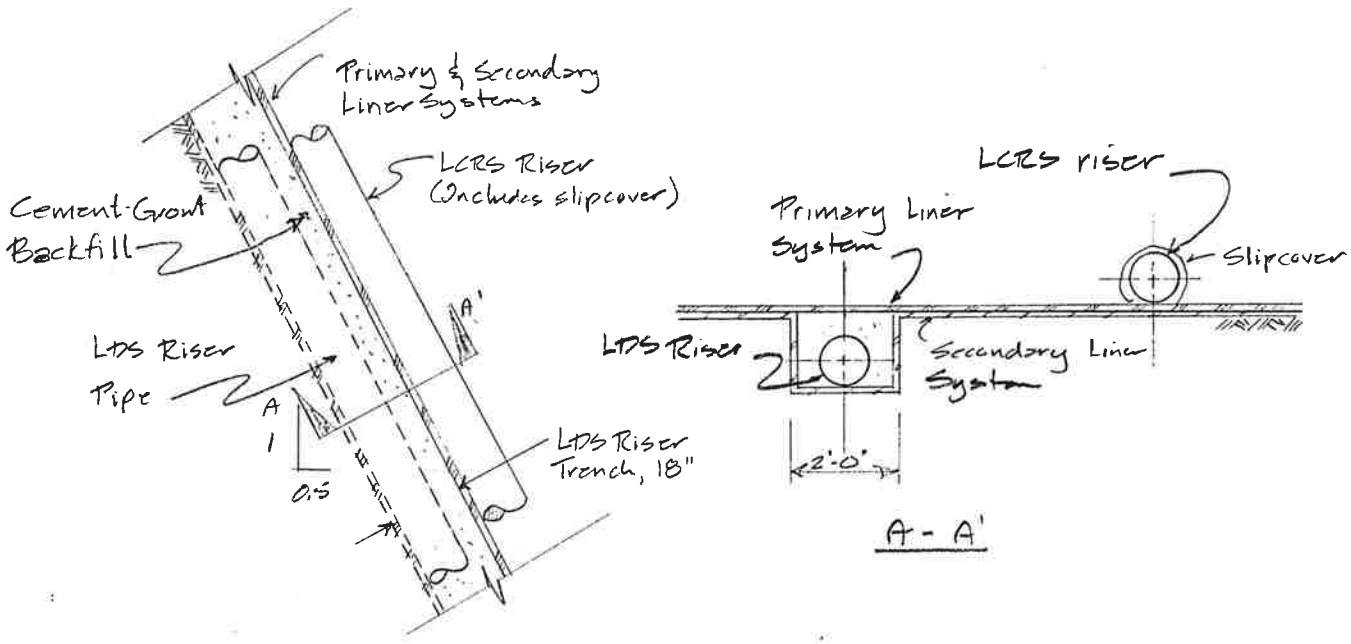


Figure 2
Detail of risers on Slope

Preliminary Selection of matl's is as follows:

LCRS Riser: 12" dia., Sch. 40 Carbon Steel Pipe
LDS Riser: 12" dia, SDR 17, HDPE Pipe
Slip Cover: Split Corrugated PE pipe - 12" dia.



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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/25/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 4 of 22

Chkd. By EC Date 2/13/96 AMERICAN ECOLOGY CORP. Proj. No. 95-234

ANALYSIS (cont.)

4) The preliminary plan view of the sumps is show below in Figure 3.

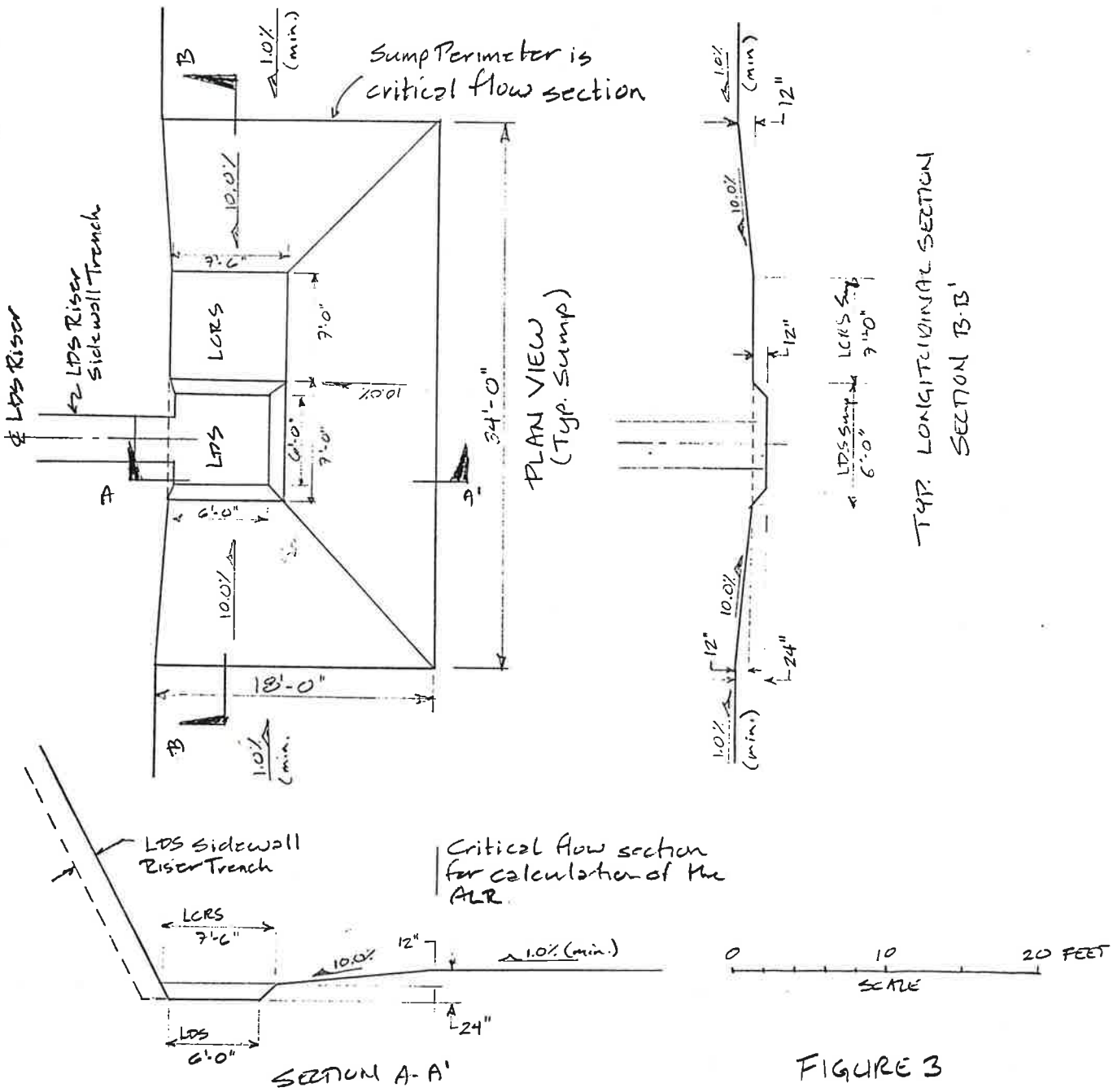


FIGURE 3



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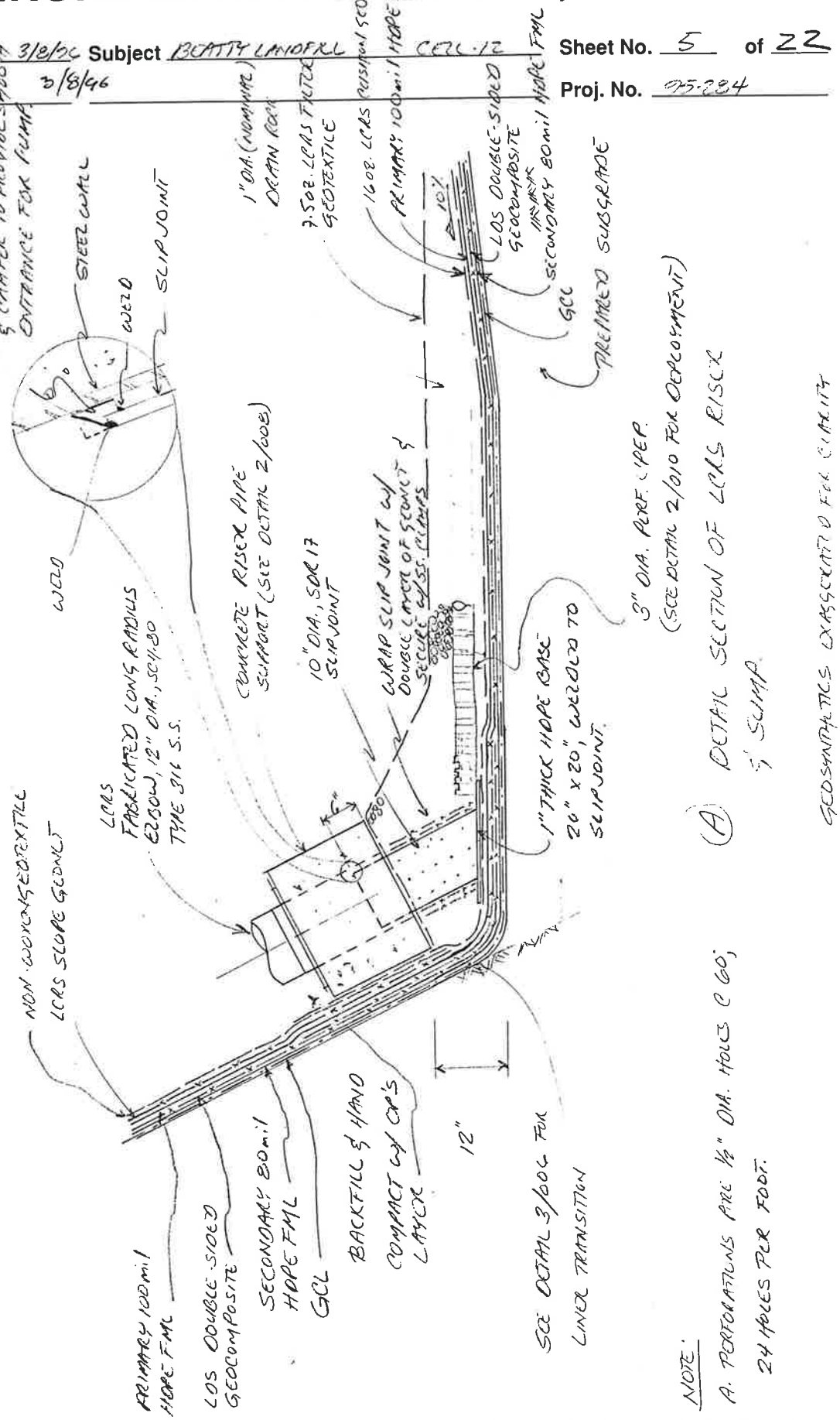
By RVH Date 3/8/96 Subject BEATTY LANDFILL CELL-12 Sheet No. 5 of 22

Chkd. By EC Date 3/8/96 Proj. No. 95-284



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# ENVIRONMENTAL SOLUTIONS, INC.

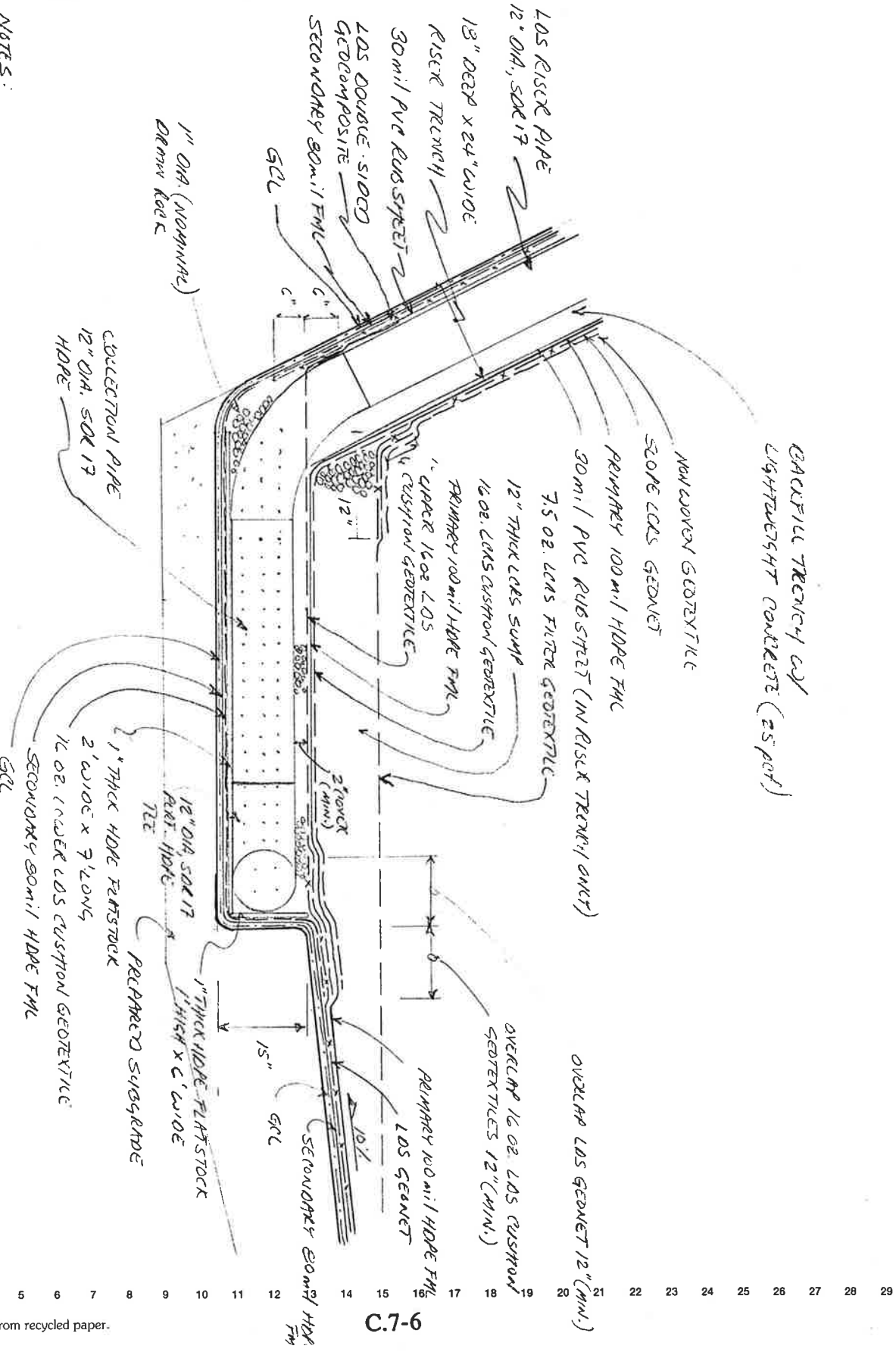
By RVH Date 3/8/96 Subject BEATTY LANDFILL CELL-12 Sheet No. 6 of 22

Chkd. By EC Date 3/8/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284

NOTES:

A) PERFORATIONS ARE 1/2" DIA. HOLES AT 60°, 24 HOLES PER FOOT.

B) WRAP PERFORATED SECTIONS OF DOUBLE LAYER OF GEONET & SECURE W/ STAINLESS STEEL CLAMPS.




B) DETAIL SECTION OF LOS SUMP & RISER

1" = 2'-0"

GEOSYNTHETICS EXAGGERATED FOR CLARITY

PIPE SHOWN SMA THAN SCALE FOR CLARITY.



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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/26/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 7 of 22

Chkd. By EC Date 2/13/96 AMERICAN ECOLOGY CORP. Proj. No. 05-284

ANALYSIS (cont.):

7) Detailed analysis of the loads for each riser will be handled separately. Both the foundation & anchors for each riser will be capable of independently supporting the riser's weight.

LCRS Riser Loads

(REF. 1)

Riser Weight - Sch. 40 Carbon Steel weighs 53.5 lbs/ft. Based on the typical section the riser is approx. 85' in length. It's weight is therefore:

$$53.5\ lb/ft \times 85\ ft = 4{,}550\ lbs = 4.6\ Kips$$

Overburden Load on riser foundation - Dry Density of the waste is $\gamma_d = 100\ lb/ft^3$, (REF 2A) Per Figure 1 the average height of waste above the sump foundation is 100'. Overburden stress is therefore

$$\sigma_v = 100' \times 100\ lb/ft^3 = 10{,}000\ lb/ft^2 \approx 10.0\ Kips/ft^2$$

Frictional Downdrag (DUE TO WASTE SETTLEMENT) - Downdrag is calculated by the following formula: for the general vertical case:

$$F_D = (P_o)\tan(\delta)$$ (REF. 3) (No Allowance is taken for reduction of settlement w/ depth - conservative).

where

$F_D$ = Force due to down drag

$P_o$ = Normal force on Riser, The force is calculated from the vertical stress distribution on the riser, converting to a lateral force, & determining the normal component of that force.

$\delta$ = Friction angle between the stl. riser & the HDPE slip cover.



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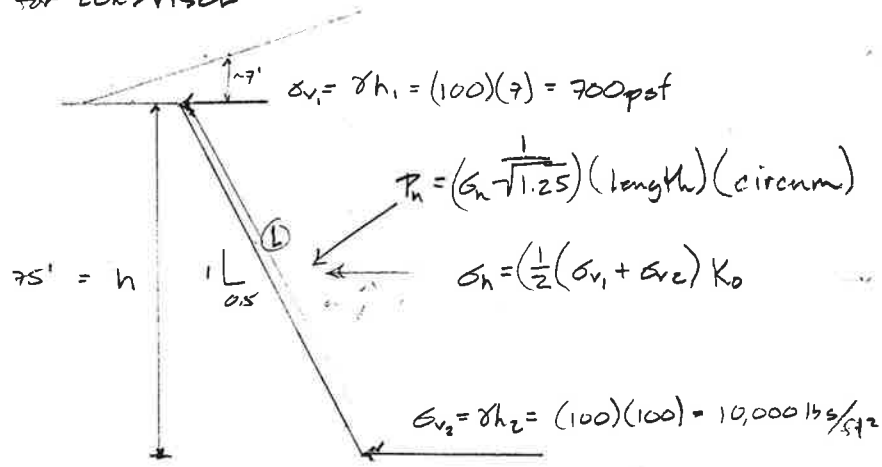
# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/26/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 8 of 22

Chkd. By EC Date 2/13/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284

ANALYSIS (cont):

$P_o$ for LCRS riser




$\sigma_h = \frac{1}{2}(\sigma_{v_1} + \sigma_{v_2}) h K_o$

$= \frac{1}{2}(700 + 10{,}000)\underbrace{(1 - \sin\phi_\omega)}_{K_o}$ (REF. 3B) , $\phi_\omega = 35°$ (REF. 2B)

$= 5{,}350 \text{ lb/ft}^2 \; (1 - \sin(35°)) = 2{,}281 \text{ lb/ft}^2$

$P_h = (2{,}281 \text{ lb/ft}^2)(\frac{1}{\sqrt{1.25}})(85')(\underbrace{\pi \times 1'}_{\text{circum.}}) = 545$ Kips

$F_D = 545$ Kips $\tan\phi$ ; $\phi = 10°$ (Assume HDPE on steel is similar to HDPE on HDPE)

$F_D = (545 \text{ Kips}) \tan(10°) = 96$ Kips

Total Longitudinal Loads = 4.6 Kips + 10.0 Kips + 96.0 Kips = $\boxed{110.6 \text{ Kips}}$

Max. Transverse Load = $(10{,}000 \text{ lb/ft}^2)(1')(\frac{1}{\sqrt{1.25}}) = 8{,}944$ lb $\simeq 9.0$ Kips
(Nominal one foot length)
(At bottom of Excavation)



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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/25/06 Subject BEATTY LANDFILL - CELL 12 Sheet No. 9 of 22

Chkd. By EC Date 2/14/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284

ANALYSIS (cont.)

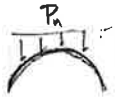
The steel riser pipe is evaluated as a flexible pipe system.

• Wall Crushing:

Ring Compression is given by the formula

$$RC = \frac{\sigma_n D}{(2A)}$$ , which for the Nom. 12" dia. pipe (REF 4A)

$D = 12.75"$




$$\frac{10{,}000\ lb}{ft^2} \times \frac{12.75"}{12"/ft} = 10{,}625\ lb/ft$$

$$\frac{10{,}625\ lb}{ft} \cdot \left(\frac{ft}{12\ in}\right) = 885\ lb/in$$

A = wall thickness = 0.406" (Ref. 1)

$$\text{Wall stress} = \left(\frac{885\ lb}{in}\right)\left(\frac{1}{2(.406\ in)}\right)$$

$$= \underline{1.1\ ksi} << (0.6)(30\ ksi) = 18\ ksi$$

MIN. YIELD POINT OF TYPE 316 SS. (REF 1B)

• Wall Buckling:

$$P_{cr} = 1.15\sqrt{E' \times P_c}$$ (REF 4B)

$P_{cr}$ = Critical buckling pressure

$P_c$ = Hydrostatic critical collapse differential pressure

$$P_c = \frac{2E}{1-\nu^2}\left(\frac{t^3}{D}\right)$$

E = Young's Modulus ($29 \times 10^6$ psi)

$\nu$ = Poisson's Ratio (0.25)

t = wall thickness (0.406")

D = diameter (12.75")

$$P_c = \frac{2(29 \times 10^6)}{1-(0.25)^2}\left(\frac{(0.406)^3}{12.75}\right)$$

$$= 324{,}731\ lb/in^2$$

E' = Soil Modulus (100 psi) (REF 4C)

$$\nu = \frac{E}{2G} - 1$$

$$P_{cr} = 1.15\sqrt{(100)(324{,}731)}$$

G = Modulus of Rigidity ($11.6 \times 10^6$ psi)

$$\nu = \frac{29}{2(11.6)} - 1 = 0.25$$

MAX ALLOW. $= 6.6\ ksi >> 10{,}000\ lb/ft^2 = \frac{70\ lb}{in^2}$



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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/26/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 10 of 22

Chkd. By EC Date 2/13/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284

ANALYSIS (cont.)

- Ring Deflection[1]

Based on $\frac{\Delta y}{D} = \epsilon$ (REF 4D), w/ $\epsilon \leq 5\%$ for steel pipe

By Spangler's equation (REF. 4E)

$$\frac{\Delta y}{D} = \frac{D_L K (\gamma H)}{\frac{EI}{r^3} + 0.061 E'}$$

Where:
- $\Delta y$ = ring deflection
- $D$ = diam (12.75") ← 12" dia. Stl. Pipe
- $D_L$ = Log Factor (1.0) (REF 4F)
- $K$ = Bedding Constant (0.11)
- $E$ = Young's Modulus ($29 \times 10^6$ psi)
- $I$ = Moment of Inertia (300 in$^4$)
- $r$ = Mean pipe radius (6.38 in)
- $E'$ = 100 psi

$$\frac{\Delta y}{D} = \frac{(1.0)(0.11)(100\ lb/ft^3)\left(\frac{ft}{12 in}\right)^3(100\ ft)\left(\frac{12 in}{ft}\right)}{\frac{(29 \times 10^6)(300)}{(6.38)^3} + (0.061)(100)}$$

$$= 2.28 \times 10^{-7} \ll 0.05$$

CONSERVATIVE & STILL < 5%

---

(1) THE LOAD APPLIED ON THE DEFLECTION FORMULA IS THE MAXIMUM SOIL LOAD, WHICH IS CONSERVATIVE FOR THIS CALCULATION. THE MORE APPROPRIATE LOAD IS THE LATERAL, OR NORMAL TO THE PIPE, SOIL PRESSURE.



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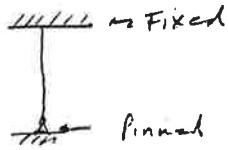
ANALYSIS (cont.):

8) The LCRS riser shall be checked longitudinally as a tension & compression member, since the design is intended to be able to support the riser under both conditions. Schematically the riser is this condition:




(REF 5)
$K = 0.7$

The riser will experience the following tensile forces:

- Self weight
- Downdrag

The anchor system at the top of slope will be designed w/ a slip joint to prevent loads induced by settlement from being transferred to the riser.

Per AISC specification maximum stress on a tensile member is $F_t = 0.6 F_y$

where $F_t$ = Tensile Stress

$F_y$ = Yield Stress of mat'l (36 ksi for carbon steel)
30 ksi for Type 316 SS

$\Rightarrow F_t = 0.6(36) = 21.6$ ksi

$= 0.6(30) = 18.0$ ksi ← use SS (MORE CONSERVATIVE)

Stress in Riser is

$$F_t = \frac{P}{A}$$

$P = 99.4$ Kips, from pgs 8 & 9.
$A = 15.74$ in$^2$ (REF 1A)

$$F_t = \frac{99.4}{15.74} = 6.32 \text{ ksi} \leq 18.0 \quad \underline{\underline{OK}}$$

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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/27/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 12 of 22
Chkd. By EC Date 2/14/96 AMERICAN ECOLOGY CORP Proj. No. 95-284

ANALYSIS (cont.):

Check of the LCRS Riser Longitudinally in compression.
The most critical time for the riser, relative to compression is when the system is first installed, & is at its greatest unsupported length. As the foundation & anchor will be provided w/ slip joints the only load is the pipe self weight = 4.6 Kips.

The slenderness ratio for this pipe is given by $Kl/r$ (REF 5)
where K = Effective Length Factor (use 0.7)
l = unsupported length (use 85')
r = radius of gyration (use 4.37) (REF. 1)
Although the riser is supported by the slope, it is not restrained from lateral deflections

$$Kl/r = \frac{(0.7)(85)\left(\frac{12\,in}{ft}\right)}{4.37} = 163.4$$

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = 126.1 \quad \text{(REF. 5B)}$$

Since $Kl/r > C_c$ use AISC eq. 1.5-2 for Allowable stress REF. 5B

$$F_a = \frac{\pi^2 E}{(Kl/r)^2}\left(\frac{12}{23}\right)$$

$$= \frac{\pi^2(29{,}000\,ksi)}{(163.4)^2}(0.52) = 5.59\ ksi > \frac{4.6}{15.74} = 0.29\ ksi$$

Although this formula is very conservative since it is intended for primary structural members the riser still performs satisfactorily.



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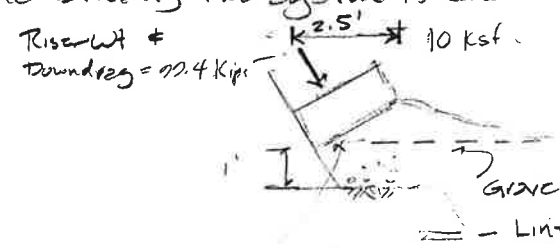
# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/27/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 13 of 22

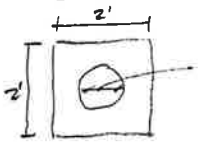
Chkd. By EC Date 2/14/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284

ANALYSIS (cont.):

9) LCRS Riser Foundation – is intended to transfer the loads from the riser pipe, & the overburden to the LCRS gravel & to the subgrade. Schematically the system is shown below:



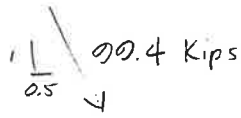

Bottom Area of Conc. Block




$(2' \times 2') - \left(\frac{\pi}{4}\right)\left(\frac{12.75}{12}\right)^2$

$= 4 - 0.89 = 3.11\ ft^2$

(closed later)




$P_v = \frac{2}{\sqrt{5}}\ 99.4 = 88.91$

Total Load = 88.9 + (10 ksf × 2 ft² × 2.5 ft²)
= 138.9 Kips

Ultimate bearing capacity is given by

$$q_{ult} = cN_c(s_c, i_c, b_c) + \tfrac{1}{2} B \gamma_1 N_\gamma (s_\gamma, i_\gamma, b_\gamma) + \gamma_2 D_f (N_q - 1)(s_q, i_q, b_q)$$ (REF 3C)

Definitions of the terms are provided in the References



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# ENVIRONMENTAL SOLUTIONS, INC.

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ANALYSIS (cont.)

10) Values used for the parameters in the ultimate bearing capacity calculation are shown below:

$c = 2,160$ psf

$N_q = \tan^2(45° + \phi/2)\, e^{\pi \tan\phi}$

$= \tan^2(62.5)\, e^{\pi \tan(35°)} = 33.30$

$N_c = (N_q - 1)\cot\phi = (33.3 - 1)\frac{1}{\tan\phi} = 46.2$

$N_\gamma = 2(N_q + 1)\tan\phi = 2(33.3 + 1)\tan(35°) = 47.75$

$N_q/N_c = \frac{33.30}{46.2} = 0.72$

$s_c = 1 + (B/L)(N_q/N_c) = 1.72$

$s_\gamma = 1 - 0.4(B/L) = 0.6$

$s_q = 1 + B/L(\tan\phi) = 1 + (\tan 35°) = 1.70$

$D_f = 100$

$\gamma_1 = 120$ pcf

$\gamma_2 = 100$ pcf

$i_q = \left(1 - \frac{Qh}{Q + BLc\cot\phi}\right)^m$

$= \left(1 - \frac{172}{177 + 9(2160)\cot 35°}\right)^{5/2} = 0.99$

$i_\gamma = \left[1 - \frac{Qh}{Q + BLc\cot(35°)}\right]^{1+m}$

$= (0.99)^{5/2} = 0.98$

$i_c = i_q - \left(\frac{1 - i_q}{N_c \tan\phi}\right) = 0.99 - \left(\frac{1 - 0.99}{(46.2)(\tan 35°)}\right) = 0.98$

$b_\gamma = \left(1 - \frac{\alpha}{57.3}\tan\phi\right)$

$= 1 - \frac{29.57}{57.3}\tan(35°) = 0.64$

$b_c = b_\gamma = 0.64$

$m = \frac{2 + L/B}{1 + L/B} = 3/2$

$b_c = b_q - \frac{(1 - b_q)}{N_c \tan\phi}$

$= 0.64 - \frac{(1 - 0.64)}{(46.2)\tan 35°}$

$= 0.63$

$q_{ult} = (2160)(46.2)(1.72)(0.99)^{0.64} + \frac{1}{2}(3)(120)(47.75)(0.6)(0.98)^{0.64} + (100)(100)(33.3 - 1)(1.70)(0.98)(0.6$

$= 451$ Ksf ← This reflects the strength of the cemented sandy soil

CISE FS = 3 ⇒ $q_{ult} = \frac{451}{3} = 150$ ksf



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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/27/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 15 of 22

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ANALYSIS (cont.)

LCRS Riser Foundation (cont)

Since the stress at the base of the block is no greater than

$138.9 \text{ Kips}/2.23 \text{ ft}^2 = 62.21 \text{ Ksf} < 150 \text{ ksf}$ so OK

Elastic Deformation of Primary (12") gravel base

$\epsilon = \frac{\sigma_v}{E_G}$ $E_{gravel} = 14 \text{ Ksi} \left(\frac{144 \text{ in}^2}{\text{ft}^2}\right) = 2{,}016 \text{ Ksf}$

$\epsilon = \frac{62.21}{2{,}016} = 3.09\%$

$\Rightarrow$ settlement in gravel = 0.37"

Check for lateral slip - critical condition is w/no overburden

LATERAL FORCE = $\frac{1}{\sqrt{5}}$ SELF WEIGHT

$= \frac{1}{\sqrt{5}} (4.6 \text{ Kips}) = 2.06 \text{ Kips}$

RESISTING FORCE = $T_D \tan \delta$

$= \left[\frac{2}{\sqrt{5}} (4.6) + 3.11 \text{ ft}^2 \left((2 \text{ ft})(150 \text{ lb/ft}^3) \frac{1 \text{ Kip}}{1000 \text{ lb}}\right)\right] \tan 28°$

for $\delta = 0.8\phi$ ; $\phi_w = 35°$

$= 2.68$ $\delta = 28°$

$FS = \frac{2.68}{2.06} = 1.30$

TEMPORARY SUPPORT OF THE RISER WILL BE NECESSARY UNTIL THE 2' OPERATIONS LAYER IS PLACED. (e.g. CRANE AT TOP OF SLOPE)



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# ENVIRONMENTAL SOLUTIONS, INC.

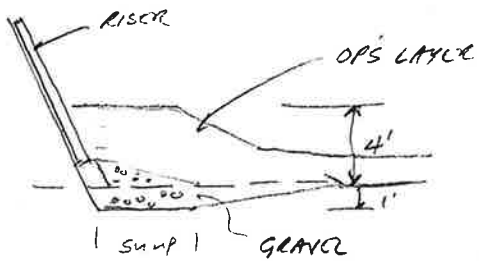
By RVH Date 2/26/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 16 of 22

Chkd. By EC Date 2/26/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284

ANALYSIS (cont.)

ADDITIONAL LATERAL RESISTING FORCE:

AT THE START OF WASTE FILLING, IT IS DESIRABLE TO HAVE A FS=2 AGAINST LATERAL SLIPPAGE. A FOUR FOOT THICK OPS LAYER SHALL BE PLACED OVER THE SUMP AREA.




THE ADDITIONAL LATERAL RESISTANCE IS CALCULATED AS A PASSIVE BLOCK

$$P_b = \sigma_v \tan^2(45 + \phi/2)$$

$$= (4)(125\,pcf)\tan^2\left(45 + \frac{35}{2}\right) = 1845\,lbs = 1.8\,Kips$$

$$FS = \frac{2.06 + 1.85}{2.06} = 2.20 > 2.0 \quad \text{so } \underline{OK}$$



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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/30/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 17 of 22
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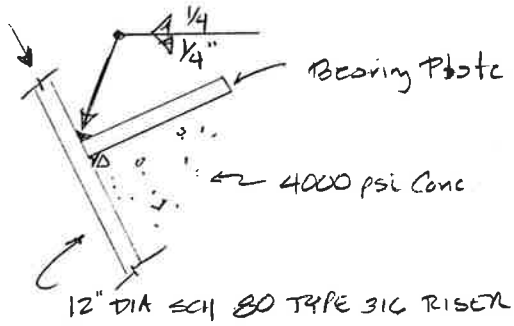
ANALYSIS (cont.):

LCRS Riser Foundation - Bearing Connection

10) The weld between the riser & bearing plate will be subjected to shear along the longitudinal access of the riser. Transverse shear will be resisted by the riser.

Load = 99.4 kips




$$f_v = \frac{99.4 \text{ kips}}{2(0.71)(0.25")(40")} = 7.1 \text{ ksi} < 0.3(70 \text{ ksi}) = 21 \text{ ksi (REF 6)}$$

Use 70 ksi Stainless Stl. Type Electrode.

11) Anchor Bolts in Shear

The total normal load on the pipe is approx 545 kips. However, Most of that load is transferred to the side wall, since the riser is in almost continuous contact. ~~H~~ Where the 3° Elbow the riser is no longer in direct contact w/ the wall. Assume 1/3 of load is transferred; 545/3 = 182 kips. The Concrete will also take up a portion of this load, conservatively assume all shear is taken by bolts (use 1" dia)

$$f_v = \frac{P}{A_B \text{ No Bolts}} = \frac{179 \text{ kips}}{\frac{\pi}{4}(1)^2(12)} = 19.0 \text{ ksi}$$

$F_v = 21.0$ (for A325 bolts - Assume 316 is similar)

$f_v < F_v$ ok



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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/30/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 18 of 22

Chkd. By EC Date 2/13/96 AMERICAN ECOLOGY CORP. Proj. No. 95-238

ANALYSIS (cont.)

LCRS Riser Foundation - Brg. Plate Shear

$F_v = 0.4 F_y = 0.4(30\ ksi) = 12\ ksi$ ← For TYPE 316 SS.

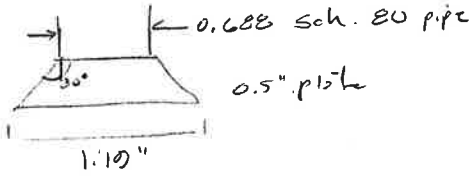
$f_v = \frac{99.4\ Kips}{15.75\ in^2} = 6.31\ ksi < 12\ ksi$ OK -

↑ GROSS AREA OF RISER (SHT. 10)

Brg Plate on Concrete

Max allowable stress $= \phi\, 0.85 f'_c = (0.7)(0.85)\,4000\ psi = 2.38\ ksi$

Load = 99.4 Kips

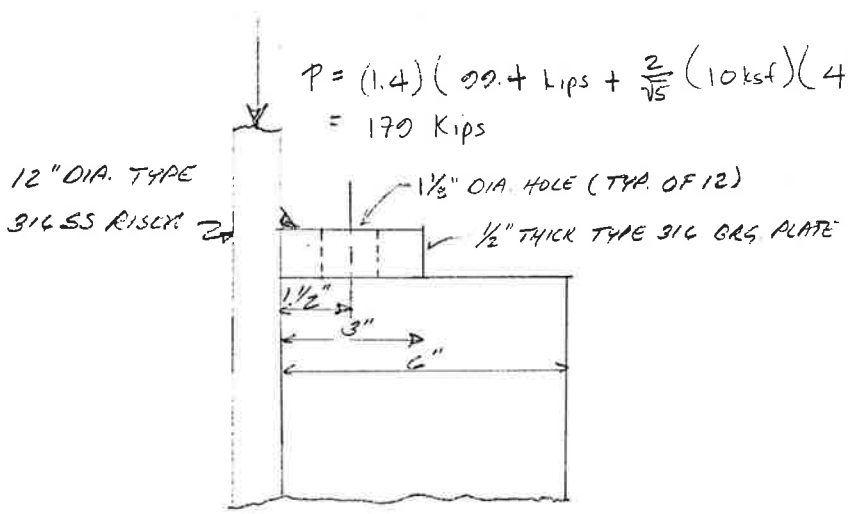



$\text{Stress} = \frac{99.4\ Kips}{(40)(1.10)} = 2.09\ ksi < 2.38$ ok

↑ Avg. Circumference of Brg Plate.

Wall of Riser Foundation - Design as Brg. wall (REF 7A)

$P = (1.4)\left(99.4\ Kips + \frac{2}{\sqrt{5}}(10\ ksf)(4\ ft^2 - 0.79\ ft^2)\right)$
$= 170\ Kips$



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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/30/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 19 of 22

Chkd. By EC Date 2/13/96 AMERICAN ECOLOGY CORP Proj. No. 95-284

ANALYSIS (CONT.)

LCRS RISER FOUNDATION - CONC. DESIGN

BEARING STRENGTH OF CONC. UNDER BAS. PLATE.

$$\phi\, 0.85\, f_c' A_1 = (0.7)(0.85)(4000)(5'' \times 40'')$$
$$= 285 \text{ Kips} > 170 \text{ Kips} \quad \text{SO } \underline{\text{OK}}$$

CAPACITY OF WALL — ALTHOUGH NOT DIRECTLY APPLICABLE - PROVIDES A CONSERVATIVE ESTIMATE OF CONC. STRENGTH IN THIS CONDITION.

$$\phi P_{nw} = 0.55\, \phi\, f_c' A_g \left(1 - \left(\frac{K L_c}{32 h}\right)^2\right)$$ (TERMS ARE DEFINED IN REF. 7A, pp. 241)

$\phi = 0.70$

$f_c' = 4000$ psi

$A_g = (5 + 4(6))(6) = 162 \text{ in}^2$ SEE REF, p. 242 — THERE IS NO UNSUPPORTED DISTANCE

$K = 0.8$ REF. —, p. 241

$L_c = 16$ in

$h = 6''$

$$\phi P_{nw} = (0.55)(0.7)(4)(162)\left(1 - \left(\frac{(0.8)(16)}{(32)(6)}\right)^2\right) = 240 \text{ Kips} > 170 \text{ Kips}$$

VERTICAL STEEL (REF 7A), (7B FOR AREA OF STL. BARS)

$$A_{sv} = 0.0012\, bh = 0.0012(12)(6) = 0.086 \text{ in}^2/\text{ft of length}$$

USE No. 4 BAR EVERY 18"

HORIZONTAL STEEL

$$A_{sh} = 0.002\, bh = 0.002(12)(6) = 0.144 \text{ in}^2/\text{ft of height}$$

USE No. 4 BARS TOP & BOTTOM.



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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/30/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 20 of 22

Chkd. By EC Date 2/13/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284

ANALYSIS (CONT.)

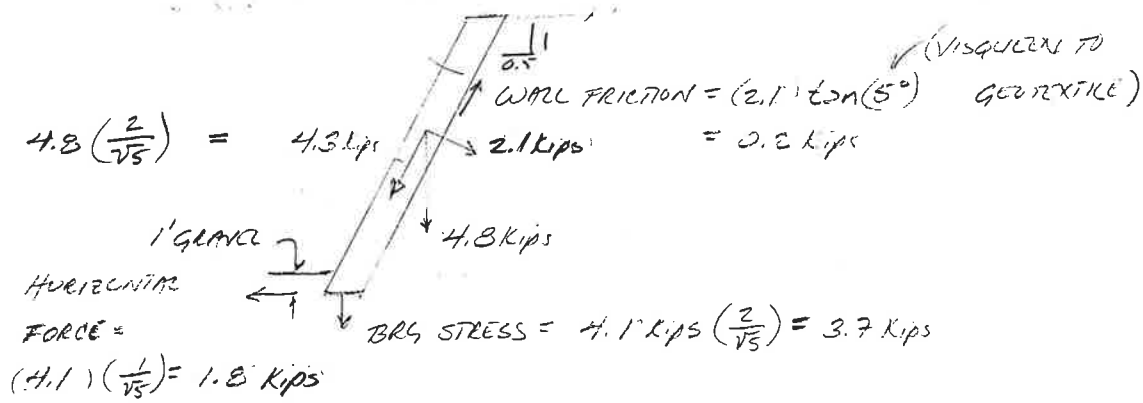
LDS RISER - THE LDS RISER IS LOCATED IN A TRENCH ON THE CELL SIDEWALL. (SEE FIG. 2).

THE RISER IS A 12" DIA., SDR 17 HDPE PIPE
WEIGHT OF THE RISER IS (REF 8A)

$$WT = 85' \times 12.03 lb/ft = 1{,}022 \text{ lbs.} \simeq 1.0 \text{ Kip}$$

THE TRENCH IS BACKFILLED W/ A LIGHTWEIGHT CEMENT
$\gamma = 20$ pcf (REF 9A)

$$\text{CEMENT WT} = 85'[(2' \times 1.5') - 0.79 ft^2](20 lb/ft^3)$$
$$= 3{,}757 \text{ lb} - \text{USE } 3.8 \text{ Kips}$$



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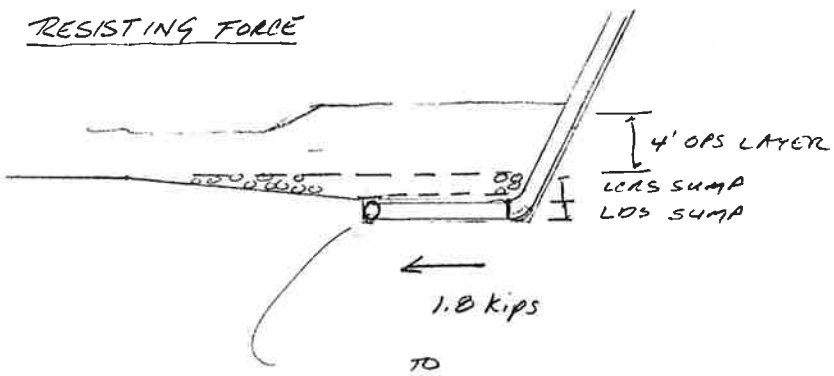
# ENVIRONMENTAL SOLUTIONS, INC.

By RVM Date 2/26/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 21 of 22

Chkd. By EC Date 2/26/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284

RESISTING FORCE




TO BOX CUT, INITIALLY RESIST LATERAL FORCE OVER 6' WIDE SECTION

FORCE IS THEREFORE:

$$1.8 \text{ kips} / 6 \text{ ft} = 0.3 \text{ kips/ft.}$$

(1' THICK)

PASSIVE BLOCK OF EXCAVATED SUBGRADE IS

$$P_p = \sigma_v \tan^2(45 + \phi/2) + 2c \tan(45 + \phi/2)$$

REF 2C FOR SPECIFIC VALUES

$$= (120) \tan^2(45 + 35/2) + 2(2160) \tan(45 + \tfrac{35}{2})$$

$$= 8{,}740 \text{ lb} = 8.74 \text{ kips}$$

$$FS = \frac{8.74}{0.3} = 29 \quad \text{OK.}$$

IT IS ASSUMED THE RISER WILL BE SUPPORTED UNTIL THE GRAVEL & OPS LAYERS ARE ON PLACE TO RESTRAIN THE HORIZONTAL PIPE THEREFORE LATERAL BUCKLING WILL NOT OCCUR.



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# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/30/96 Subject BEATTY LANDFILL - CELL 12 Sheet No. 22 of 22

Chkd. By EC Date 2/13/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284

ANALYSIS (CONT.):

LOS RISER - WALL STRESSES

THE LOS RISER IS ENCASED IN CONCRETE, WHICH WILL BRIDGE MUCH OF THE ULTIMATE STRESSES AROUND THE PIPE. CONSERVATIVELY MODEL PIPE AS THOUGH NORMAL PIPE INSTALLATION.

E' FOR CONC. ≈ 3000 psi

BY DESIGN GUIDELINES PROVIDED BY DRISCOPIPE (REF 8B)
THE MAXIMUM BURIAL DEPTH OF SDR 17 PIPE IS 181 FEET > 100 FEET FOR DESIGN.

ALLOWABLE RING DEFLECTION FOR SDR 17 PIPE IS 4.2% (REF 8C)

ANTICIPATED RING DEFLECTION IS $\frac{\Delta Y}{D}$ - CALCULATE $\Delta Y$ (2)

$$\Delta Y = \frac{D_L K W_c r^3}{EI + 0.061 E' r^3} \qquad \text{(REF 4E)}$$

$D_L = 1.5$ (REF 4E)
$K = 0.083$ "
$W_c = 10\ KSF\left(1ft\right)\left(\frac{1ft}{144in}\right)$
$= 69\ lb/in$
$r = 6.3''$
$E = 150{,}000\ psi$
$I = \pi r^3 t = \pi (12.75)^3 0.75'' = 4883\ in^4$ ROARKS STRESS FORMULAS p. 67
$E' = 3000\ psi$

$$\Delta Y = \frac{(1.5)(0.083)(69)(6.3)^3}{(150{,}000)(4883) + (0.061)(3000)(6.3)^3} = 2.9 \times 10^{-4}$$

$$\frac{\Delta Y}{D} \ll 4.2\%$$

(2) ALTHOUGH FORMULA IS MORE APPROPRIATE FOR EARTH LOADS ON HORIZONTAL PIPES, IT PROVIDES A CONSERVATIVE ESTIMATE FOR THIS EVALUATION.



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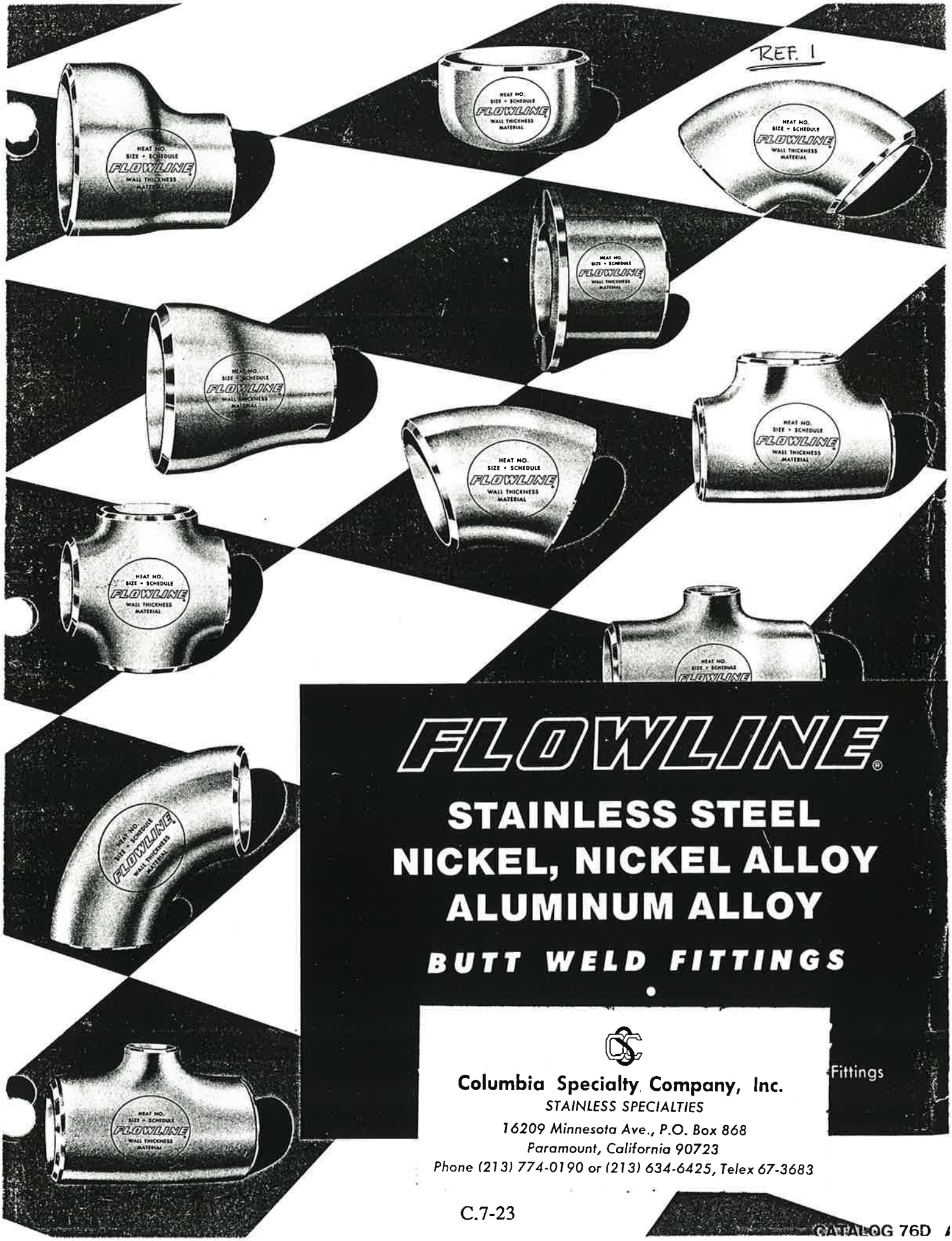
REF. 1

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## NICKEL, NICKEL ALLOY
## ALUMINUM ALLOY

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*STAINLESS SPECIALTIES*
*16209 Minnesota Ave., P.O. Box 868*
*Paramount, California 90723*
*Phone (213) 774-0190 or (213) 634-6425, Telex 67-3683*

Fittings

REF. 1

# DESIGN PROPERTIES OF PIPE

| Pipe Sched. or Weight | Wall Thick. Inches T | Inside Dia. Inches I.D. | Fifth Power of I.D. In.⁵ I.D.⁵ | Surface Area Outside Sq. ft. Per ft. $A_o$ | Surface Area Inside Sq. ft. Per ft. $A_i$ | Cross-Sectional Area of Metal Sq. in. A | Cross-Sectional Flow Area Sq. in. $A_f$ | Flow GPM Q | Weight of Pipe Lb. per ft. W | Weight of Water Lb. per ft. $W_w$ | Radius of Gyration Inches $R_g$ | Moment of Inertia In.⁴ I | Section Modulus In.³ Z | PRESSURE/STRESS RATIOS (Corrosion Allowance—Zero) Y COEFFICIENT 0.0 | 0.4 | 0.5 | 0.7 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **10" NOMINAL PIPE SIZE (O.D. = 10.750")** | | | | | | | | | | | | | | | | | |
| 5S | .134 | 10.482 | 127 | 2.81 | 2.74 | 4.45 | 86.3 | 2692.8 | 15.2 | 37.4 | 3.75 | 63.0 | 11.72 | .0218 | .0220 | .0221 | .0222 |
| 10S | .165 | 10.420 | 123 | 2.81 | 2.73 | 5.49 | 85.3 | 2665.6 | 18.7 | 36.9 | 3.74 | 76.9 | 14.30 | .0269 | .0272 | .0272 | .0274 |
| 20 | .250 | 10.250 | 113 | 2.81 | 2.68 | 8.25 | 82.5 | 2581.3 | 28.0 | 35.7 | 3.71 | 113.7 | 21.16 | .0407 | .0414 | .0415 | .0419 |
| 30 | .307 | 10.136 | 107 | 2.81 | 2.65 | 10.07 | 80.7 | 2521.9 | 34.2 | 34.9 | 3.69 | 137.5 | 25.6 | .0500 | .0510 | .0513 | .0518 |
| 40 ST 40S | .365 | 10.020 | 101 | 2.81 | 2.62 | 11.91 | 78.9 | 2465.6 | 40.5 | 34.1 | 3.67 | 160.8 | 29.9 | .0594 | .0609 | .0612 | .0620 |
| 60 XS 80S | .500 | 9.750 | 88.1 | 2.81 | 2.55 | 16.10 | 74.7 | 2334.4 | 54.7 | 32.3 | 3.63 | 212.0 | 39.4 | .0814 | .0841 | .0848 | .0863 |
| 80 | .594 | 9.562 | 80.0 | 2.81 | 2.50 | 18.92 | 71.8 | 2243.7 | 64.3 | 31.1 | 3.60 | 244.8 | 45.5 | .0965 | .1004 | .1014 | .1035 |
| 100 | .719 | 9.312 | 70.1 | 2.81 | 2.44 | 22.63 | 68.1 | 2128.1 | 76.9 | 29.5 | 3.56 | 286.2 | 53.2 | .1169 | .1226 | .1241 | .1273 |
| 120 | .844 | 9.062 | 61.2 | 2.81 | 2.37 | 26.24 | 64.5 | 2015.6 | 89.2 | 27.9 | 3.52 | 324.3 | 60.3 | .1372 | .1452 | .1473 | .1518 |
| 140 | 1.000 | 8.750 | 51.3 | 2.81 | 2.29 | 30.63 | 60.1 | 1878.1 | 104.1 | 26.0 | 3.47 | 367.9 | 68.4 | .1628 | .1741 | .1772 | .1837 |
| 160 | 1.125 | 8.500 | 44.4 | 2.81 | 2.23 | 34.02 | 56.7 | 1771.8 | 115.7 | 24.6 | 3.43 | 399.4 | 74.3 | .1831 | .1976 | .2016 | .2101 |
| **12" NOMINAL PIPE SIZE (O.D. = 12.750")** | | | | | | | | | | | | | | | | | |
| 5S | .156 | 12.420 | 296 | 3.34 | 3.25 | 6.52 | 121.2 | 3780.0 | 22.2 | 52.5 | 4.45 | 129.2 | 20.3 | .0214 | .0229 | .0229 | .0230 |
| 10S | .180 | 12.390 | 292 | 3.34 | 3.24 | 7.11 | 120.6 | 3768.7 | 24.2 | 52.2 | 4.44 | 140.5 | 22.0 | .0247 | .0250 | .0250 | .0251 |
| 20 | .250 | 12.250 | 276 | 3.34 | 3.21 | 9.82 | 117.9 | 3687.5 | 33.4 | 51.0 | 4.42 | 191.9 | 30.1 | .0343 | .0348 | .0349 | .0352 |
| 30 | .330 | 12.090 | 258 | 3.34 | 3.17 | 12.88 | 114.8 | 3587.5 | 43.8 | 49.7 | 4.39 | 248.5 | 39.0 | .0453 | .0461 | .0463 | .0468 |
| ST 40S | .375 | 12.000 | 249 | 3.34 | 3.14 | 14.58 | 113.1 | 3534.4 | 49.6 | 49.0 | 4.38 | 279 | 43.8 | .0515 | .0526 | .0528 | .0534 |
| 40 | .406 | 11.938 | 242 | 3.34 | 3.13 | 15.74 | 111.9 | 3496.9 | 53.5 | 48.5 | 4.37 | 300 | 47.1 | .0557 | .0570 | .0573 | .0580 |
| XS 80S | .500 | 11.750 | 224 | 3.34 | 3.08 | 19.24 | 108.4 | 3387.5 | 65.4 | 47.0 | 4.33 | 362 | 56.7 | .0686 | .0706 | .0711 | .0721 |
| 60 | .562 | 11.626 | 212 | 3.34 | 3.04 | 21.52 | 106.2 | 3318.8 | 73.2 | 46.0 | 4.31 | 401 | 62.8 | .0771 | .0796 | .0802 | .0815 |
| 80 | .688 | 11.374 | 191 | 3.34 | 2.98 | 26.04 | 101.6 | 3175.0 | 88.5 | 44.0 | 4.27 | 475 | 74.5 | .0943 | .0980 | .0990 | .1010 |
| 100 | .844 | 11.062 | 166 | 3.34 | 2.90 | 31.53 | 96.1 | 3003.1 | 107.2 | 41.6 | 4.22 | 562 | 88.1 | .1157 | .1213 | .1228 | .1259 |
| 120 | 1.000 | 10.750 | 144 | 3.34 | 2.81 | 36.91 | 90.8 | 2837.5 | 125.5 | 39.3 | 4.17 | 642 | 100.7 | .1373 | .1452 | .1474 | .1518 |
| 140 | 1.125 | 10.500 | 128 | 3.34 | 2.75 | 41.09 | 86.6 | 2706.3 | 139.7 | 37.5 | 4.13 | 701 | 109.9 | .1544 | .1646 | .1673 | .1713 |
| 160 | 1.312 | 10.126 | 106 | 3.34 | 2.65 | 47.14 | 80.5 | 2515.6 | 160.3 | 34.9 | 4.07 | 781 | 122.6 | .1801 | .1941 | .1979 | .2061 |
| **14" NOMINAL PIPE SIZE (O.D. = 14.000")** | | | | | | | | | | | | | | | | | |
| 5S | .156(A) | 13.688 | 481 | 3.67 | 3.58 | 6.78 | 147.2 | 4580.1 | 23.1 | 63.7 | 4.90 | 163 | 23.2 | .0195 | .0196 | .0197 | .0198 |
| 10S | .188(A) | 13.624 | 469 | 3.67 | 3.57 | 8.16 | 145.8 | 4537.3 | 27.7 | 63.1 | 4.88 | 195 | 27.8 | .0235 | .0237 | .0238 | .0239 |
| 10 | .250 | 13.500 | 448 | 3.67 | 3.53 | 10.80 | 143.1 | 4468.8 | 36.7 | 62.0 | 4.86 | 255 | 36.5 | .0312 | .0316 | .0317 | .0319 |
| 20 | .312 | 13.375 | 428 | 3.67 | 3.50 | 13.44 | 140.5 | 4390.6 | 45.7 | 60.8 | 4.84 | 315 | 45.0 | .0390 | .0396 | .0398 | .0401 |
| 30 ST 40S | .375(1) | 13.250 | 408 | 3.67 | 3.47 | 16.05 | 137.9 | 4309.4 | 54.6 | 59.7 | 4.82 | 373 | 53.3 | .0469 | .0478 | .0480 | .0485 |
| 40 | .438 | 13.124 | 389 | 3.67 | 3.44 | 18.66 | 135.3 | 4228.1 | 63.4 | 58.6 | 4.80 | 429 | 61.4 | .0548 | .0560 | .0563 | .0569 |

$R_g = \sqrt{\frac{I}{A}} = \frac{\sqrt{O.D.^2 + I.D.^2}}{4}$ = radius of gyration in inches

$I = AR_g^2 = 0.0491\,(O.D.^4 - I.D.^4)$ = moment of inertia, inches fourth

$Z = \frac{2I}{O.D.} = 0.0982\,\frac{O.D.^4 - I.D.^4}{O.D.}$ = section modulus, inches cube

ST designates standard weight steel pipe; dimensions are in conformance with ANSI B36.10.

XS designates extra strong steel pipe; dimensions are in conformance with ANSI B16.10.

XX designates double extra strong steel pipe; dimensions are in conformance with ANSI B16.10.

Numbers 10, 20, 30, 40, 60, 80, 100, 120, 140 and 160 designate schedule numbers; dimensions are in conformance with ANSI B16.10.

Dimensions for Schedules 5S, 10S, 40S, and 80S are in conformance with ANSI B36.19.

(A) Proposed wall thickness for Schedules 5S and 10S.

(1) Thickness agrees with that for Standard Weight Pipe (ANSI B36.10); not included in Schedule 40S.

(2) Thickness agrees with that for Extra Strong Pipe (ANSI B36.10); not included in Schedule 80S.

(continued on next page)

REF 1B

# CHEMICAL AND PHYSICAL PROPERTIES OF STAINLESS, NICKEL, NICKEL ALLOY AND ALUMINUM ALLOY FITTINGS

| Marking Symbol | Tensile Strength Min. P.S.I. | Yield Point Min. P.S.I. | Longitudinal Elongation in 2" Min. % | ASTM | Carbon % | Manganese Max. % | Phosphorus Max. % | Titanium % Max. |
|---|---|---|---|---|---|---|---|---|
| WP 304 | 75,000 | 30,000 | 35 | A-312 | 0.08 max. | 2.00 | 0.040 | .... |
| WP 304H | 75,000 | 30,000 | 35 | A-312 | 0.04-0.10 | 2.00 | 0.040 | .... |
| WP 304L | 70,000 | 25,000 | 35 | A-312 | 0.035 max. | 2.00 | 0.040 | .... |
| WP 309 | 75,000 | 30,000 | 35 | A-312 | 0.15 max. | 2.00 | 0.040 | .... |
| WP 310 | 75,000 | 30,000 | 35 | A-312 | 0.15 max. | 2.00 | 0.040 | .... |
| WP 316 | 75,000 | 30,000 | 35 | A-312 | 0.08 max. | 2.00 | 0.040 | .... |
| WP 316H | 75,000 | 30,000 | 35 | A-312 | 0.04-0.10 | 2.00 | 0.040 | .... |
| WP 316L | 70,000 | 25,000 | 35 | A-312 | 0.035 max. | 2.00 | 0.040 | .... |
| WP 317 | 75,000 | 30,000 | 35 | A-312 | 0.08 max. | 2.00 | 0.040 | .... |
| WP 321 | 75,000 | 30,000 | 35 | A-312 | 0.08 max. | 2.00 | 0.040 | 0.60 |
| WP 321H | 75,000 | 30,000 | 35 | A-312 | 0.04-0.10 | 2.00 | 0.040 | 0.60 |
| WP 347 | 75,000 | 30,000 | 35 | A-312 | 0.08 max. | 2.00 | 0.040 | .... |
| WP 347H | 75,000 | 30,000 | 35 | A-312 | 0.04-0.10 | 2.00 | 0.040 | .... |
| WP 348 | 75,000 | 30,000 | 35 | A-312 | 0.08 max. | 2.00 | 0.040 | .... |
| WP 348H | 75,000 | 30,000 | 35 | A-312 | 0.04-0.10 | 2.00 | 0.040 | .... |
| WPN 200 | 55,000 | 15,000(1) | 40 | B-161 | 0.15 max. | 0.35 | .... | .... |
| WPNL 201 | 50,000 | 12,000(2) | 40 | B-161 | 0.02 max. | 0.35 | .... | .... |
| WPNC 400 | 70,000 | 28,000(3) | 35 | B-165 | 0.30 max. | 2.00 | .... | .... |
| WPNCI 600 | 80,000 | 35,000(4) | 30(5) | B-167 | 0.15 max. | 1.0 | .... | .... |
| WPNIC 800 | 75,000 | 30,000 | 30 | B-407 | 0.10 max. | 1.50 | .... | 0.15-0.60 |
| WP 1060 (H112)(8) | 8,500 | 2,500 | (6) | B-221 | ..... | 0.03 | .... | 0.03 |
| WP 1100 (H112)(8) | 11,000 | 3,000 | (6) | B-221 | ..... | 0.05 | .... | .... |
| WP 3003 (H112)(8) | 14,000 | ..... | (6) | B-221 | ..... | 1.0-1.5 | .... | .... |
| WP 5083 (O)(8) | 39,000 | 16,000 | 14 | B-221 | ..... | 0.40-1.0 | .... | 0.15 |
| WP 5086 (O)(8) | 35,000 | 14,000 | 14 | B-221 | ..... | 0.20-0.7 | .... | 0.15 |
| WP 6061 (T6)(8) | 38,000 | 35,000 | 8 | B-221 | ..... | 0.15 | .... | 0.15 |
| WP 6063 (T4)(8) | 19,000 | 10,000 | 14 | B-221 | ..... | 0.10 | .... | 0.10 |

(1) Yield point 5" O.D. and under; yield point over 5" O.D. is 12,000.
(2) Yield point 5" O.D. and under; yield point over 5" O.D. is 10,000.

The accompanying tables show pipe material specifications in effect when this bulletin was published. Since the specifications may be revised subsequent to issuance of this bulletin, it is advisable to check your material supplier for possible changes in the data.

REF ~~24~~ 2

# SECTION IV.D

# LANDFILL REPORT

US ECOLOGY, INC.

BEATTY, NEVADA

The compaction efforts and subsequent traffic loads generally result in a dense, compact configuration for the waste fill, which greatly reduces the potential for subsequent settlement.

Coupled with the compaction efforts, the overall climate at the Beatty facility greatly reduces the potential for settlement. The hot, dry climate which predominates at the facility, reduces the moisture content of the waste landfilled and the potential for water percolation through the waste which could cause piping or ravelling of the soil fines.

As a result of the indeterminate nature of settlement calculations for waste fills, the use of conservative parameters in a conservative analysis is warranted. The following assumptions about the waste composition and characteristics have been made.

- The waste fill composition consists of 85% soil and soil-like waste, 10% containerized waste, and 5% non-compressible solid waste.

- Soil and soil-like waste is comprised of native sandy soils, contaminated clay-like soils and stabilized sludges. It is assumed that soil-like material contains building and demolition debris consisting of concrete and rock. For purposes of the settlement calculations, the following physical properties have been assumed for the soil and soil-like waste.

**Physical Properties of Soil and Soil-Like Waste**

Average Initial Dry Density 95 pcf

Average Final Dry Density 100 pcf

Specific Gravity of Solids 2.68

REF 2B, C

# GEOTECHNICAL INVESTIGATION FOR CELL 12
# at the
# US Ecology Hazardous Waste Management Facility
# Beatty, Nevada

*Prepared for*

*American Ecology*
*Houston, Texas*

*Prepared by*

*Grant Environmental*
*Denver, Colorado*

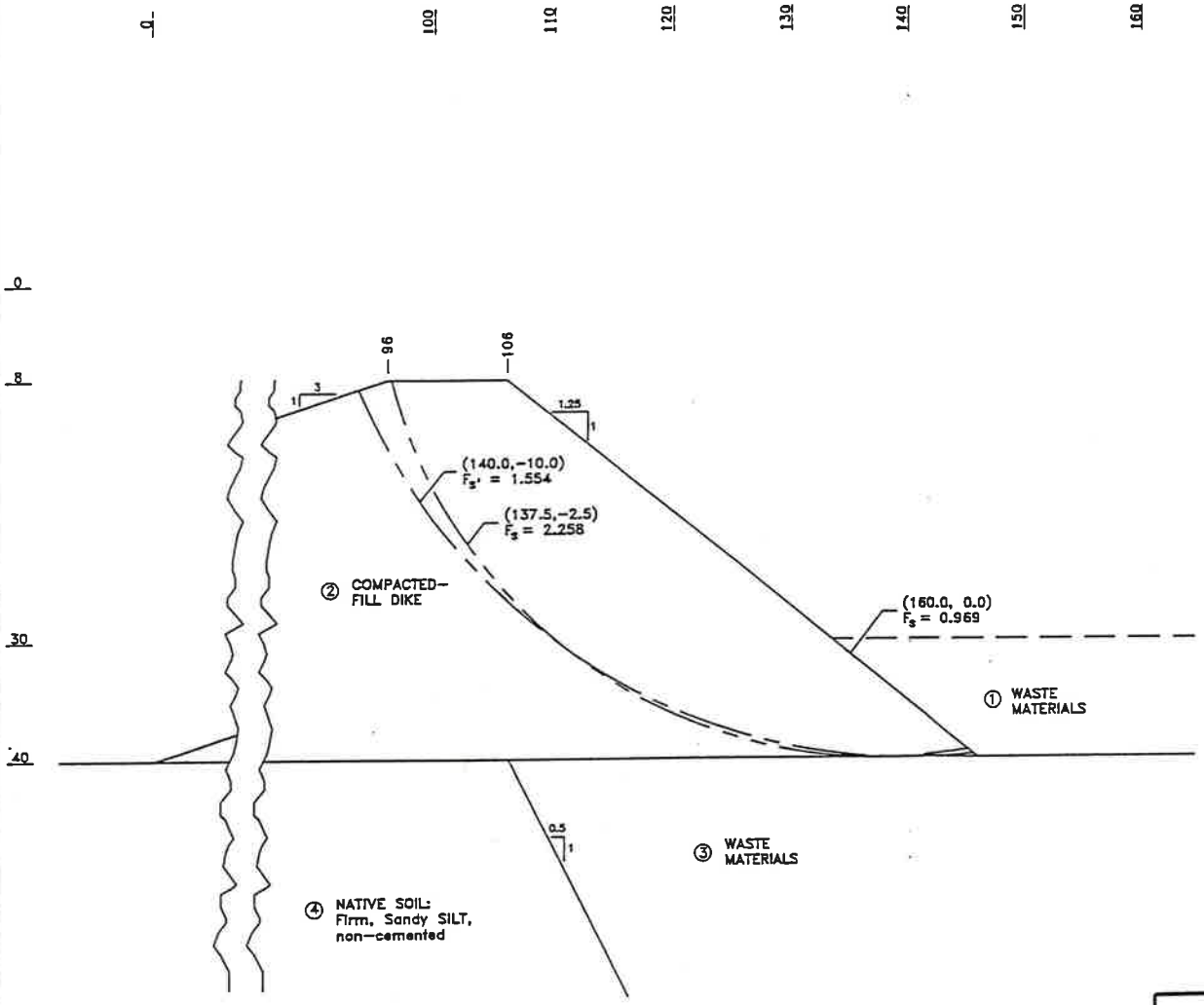
**July 14, 1994**

**Project No. 814796**

REGISTERED PROFESSIONAL ENGINEER · STATE OF NEVADA
JAMES L. GRANT
CIVIL
No. 5904


Noelle R. Cochran
Project Engineer

James L. Grant, P.E.
Senior Project Engineer




NOTE:
COORDINATES USED FOR COMPUTER MODEL.

| Layer | C (psf) | Φ (degrees) | γ (pcf) |
|---|---|---|---|
| 1 | 0 | 30 | 85 |
| 2 | 0 | 38 | 120 |
| 3 | 0 | 30 | 85 |
| 4 | 0 | 35 | 120 |

ASSUME SANDIER FRACTION FROM ON-SITE DAILY COVER $\phi = 35°$

(190.0,-30.0) CENTER OF FAILURE SURFACE

$F_s$ = FACTOR OF SAFETY

$F_s'$ = FACTOR OF SAFETY WITH HORIZONTAL SEISMIC FORCE OF 0.15g.

GRANT ENVIRONMENTAL consulting and engineering DENVER, COLORADO

FIGURE 3.5
CROSS-SECTION OF
COMPACTED FILL DIKE

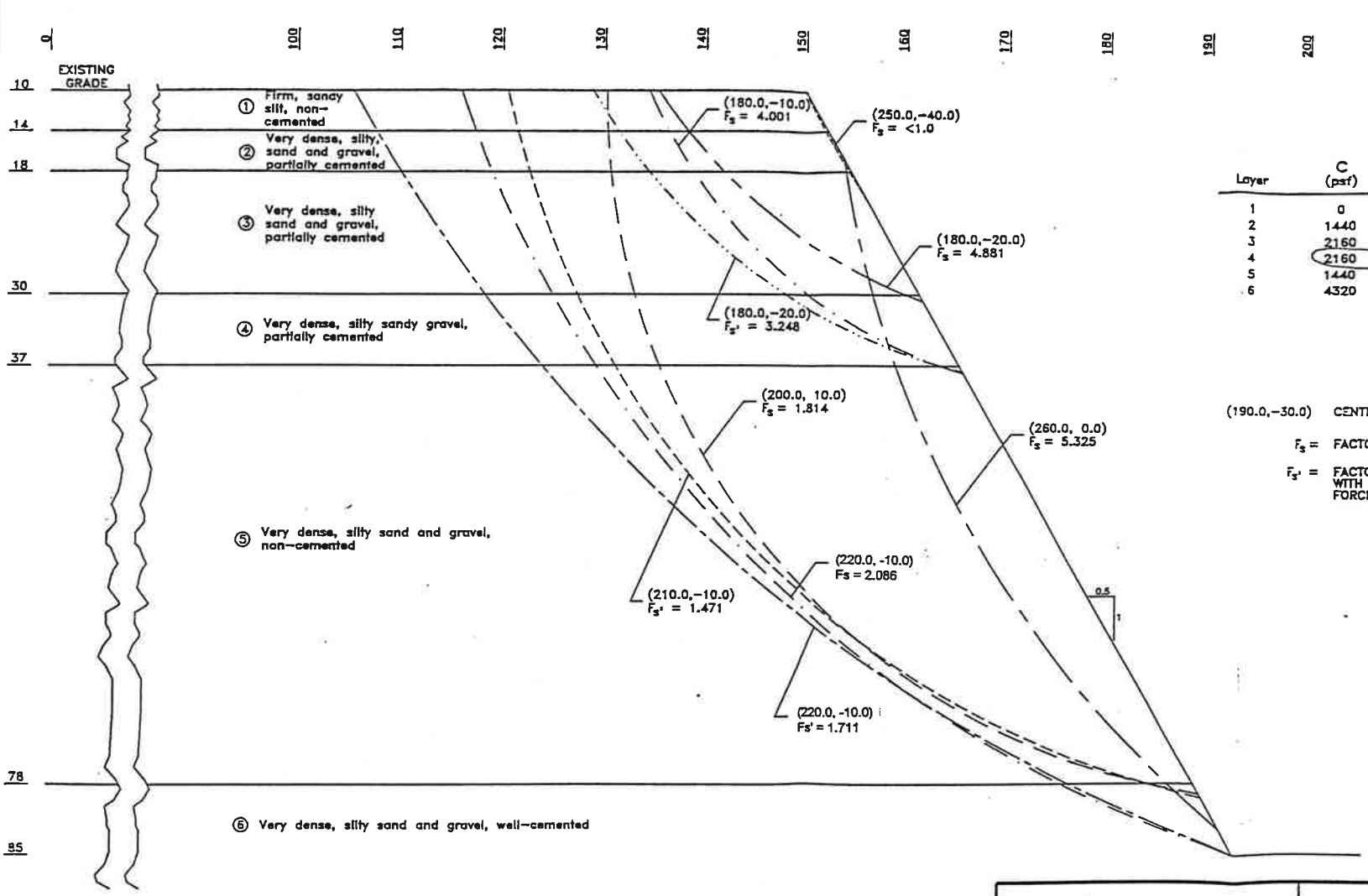
C.7-29

M/T&L DWG: 5/25/94

8142

514796

REF 2.X
B




NOTE:
COORDINATES USED FOR COMPUTER MODEL

| Layer | C (psf) | Φ (degrees) | γ (pcf) |
|---|---|---|---|
| 1 | 0 | 35 | 120 |
| 2 | 1440 | 35 | 120 |
| 3 | 2160 | 35 | 120 |
| 4 | 2160 | 35 | 120 |
| 5 | 1440 | 35 | 120 |
| 6 | 4320 | 35 | 120 |

(190.0,-30.0) CENTER OF FAILURE SURFACE

$F_s$ = FACTOR OF SAFETY

$F_s'$ = FACTOR OF SAFETY WITH HORIZONTAL SEISMIC FORCE OF 0.15g.

GRANT ENVIRONMENTAL consulting and engineering DENVER, COLORADO

FIGURE 3.1

CROSS-SECTION OF NORTH WALL

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C.7-30

REF 2C

REF 3

# ESSENTIALS OF SOIL MECHANICS AND FOUNDATIONS

BASIC GEOTECHNICS

FOURTH EDITION

DAVID F. McCARTHY

REF 3A

**Statical Method—Driven Piles in Sand** The ultimate capacity of a single pile driven in sand is the sum of the point resistance and the skin friction acting along the shaft of the pile:

$$Q_{total} = fA_{surface} + q_{tip}A_{tip} \tag{14-23b}$$

The unit skin friction for a straight-sided pile is a function of the soil pressure acting normal to the pile surface and the coefficient of friction between the soil and pile material (Fig. 14-20). Thus skin friction on a pile shaft is determined in the manner used in engineering mechanics to compute frictional resistance developing between two solid materials with movement impending.

Soil pressure acting normal to a vertical pile surface is assumed to be directly related to an effective vertical soil pressure (overburden pressure) and a lateral pressure coefficient $K$.

$$\bar{\sigma}_h = K\bar{\sigma}_v \tag{8-5}$$

where $\bar{\sigma}_h$ = horizontal soil pressure acting at any depth $Z$ in a soil mass.
$\bar{\sigma}_v$ = effective vertical pressure (overburden) acting at same depth $Z$ within a soil mass.
$K$ = lateral pressure coefficient expressing the ratio of $\bar{\sigma}_h$ to $\bar{\sigma}_v$.

The unit value of skin friction $f$ acting at any depth in a sand mass theoretically becomes

$\bar{\sigma}_v \times A = P_o$

$$f_{sand} = \bar{\sigma}_h \tan\delta = K\bar{\sigma}_v \tan\delta \tag{14-24}$$

where $\tan\delta$ = coefficient of friction between sand and the pile surface.

Representative values for the coefficient of friction between silica sand and the pile materials are shown in Table 14-7. The total skin friction acting along the embedded length of pile is then

$$Q_{friction} = f_{sand}A_{surface} = (K\bar{\sigma}_v \tan\delta)A_{surface} \tag{14-25}$$

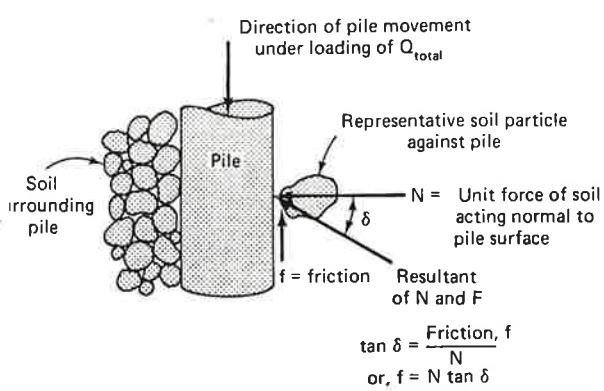


**Figure 14-20** Solid-to-solid friction developing against pile shaft.

REF. 3B

however, because, though increasing, the horizontal stress on all volumetri ments would be equal and opposite in magnitude and direction, effecting a l restraint. For this condition the relationship between horizontal stress $\sigma_h$ an tical stress $\sigma_v$ is indicated by

$$\frac{\sigma_h}{\sigma_v} = \frac{\nu}{1 - \nu} \cong \cong 1 - \sin \phi \qquad (1$$

or

$$\sigma_h = \sigma_v \left(\frac{\nu}{1 - \nu}\right) \cong \sigma_v (1 - \sin \phi) \qquad (1$$

where $\nu$ is Poisson's ratio[1] and $\phi$ is the angle of internal friction for the soil ratio of $\sigma_h$ to $\sigma_v$ in a soil mass is termed the coefficient of lateral earth pre $K$. Where no lateral deformations occur in the soil mass, the value express Equation 16-1a is considered the *at-rest* condition for which the earth pr coefficient is designated $K_0$. Therefore,

$$\sigma_h = K_0 \sigma_v \quad \text{(at-rest condition)} \qquad ($$

Representative values of $K_0$ for generalized soil categories are shown in Table

The relation of subsurface stresses to the soil's limiting strength for a t at-rest condition is shown by the Mohr's circle plot of Figure 16-2b, which ind that a state of equilibrium is maintained. If a smooth unyielding wall (a w infinite rigidity) could be embedded into the soil mass of Figure 16-2a and th to one side then removed without causing change in the previously establishe conditions, the lateral pressures acting against the wall would increase unif with depth, achieving a pressure distribution as shown in Figure 16-2c. Letti represent pressure at any depth against the wall that results from the retaine mass, Equation 16-1c becomes, for the at-rest condition,

$$p_0 = \sigma_h = K_0 \sigma_v = K_0 \gamma_t z \qquad (1$$

and the total lateral force $P_0$ per unit of wall length is the area of the pr diagram, or

$$P_0 = \frac{1}{2} K_0 \gamma_t H^2 \qquad (1$$

whose resultant acts at a depth $\frac{2}{3}H$.

**TABLE 16-1** REPRESENTATIVE VALUES OF $K_0$

| Soil type | $K_0$ |
|---|---|
| Granular, loose | 0.5–0.6 |
| Granular, dense | 0.3–0.5 |
| Clay, soft | 0.9–1.1 (undrained) |
| Clay, hard | 0.8–0.9 (undrained) |

[1]Recall from engineering mechanics that Poisson's ratio relates strain in a lateral direct the strain in a longitudinal direction that occurs as a result of load applied in the longitudinal dir

REF 3C

From Figure 14-6, $I_{r_{critical}} = 13$ for $\phi = 0°$ and curve for $B/L = 0$. Since $I_r > I_{r_{critical}}$, we have $r_c = 1$. For $\phi = 0°$, $N_c = 5.14$; $N_\gamma$ and $N_q$ are negligible:

$$\text{Calculate} \quad q_{ult} = cN_c r_c = (72 \text{ kN/m}^2)(5.14)(1.0) = 370 \text{ kN/m}^2$$

$$q_{design} = (370 \text{ kN/m}^2)(1/3) = 123 \text{ kN/m}^2$$

Allowable footing load = (123 kN/m²)($B$ meters) = 123B kN/m of wall length.

Required footing width, $B = \dfrac{135 \text{ kN/m}}{123 \text{ Kn/m}^2} = 1.1$ m.

---

### Application for Design—The Extended Bearing Capacity Equation

Equations 14-1a, 14-1b and 14-4 apply for the condition of an infinitely long strip foundation with the base horizontal, with a vertical load centrically applied, where the ground surface surrounding the foundation is horizontal; also recall that the shearing strength of the soil above the plane level with the foundation base was neglected. In practice, these conditions are not always present: Foundations have finite dimensions (the commonly used shapes include square, rectangular, and circular as well as the long strip); the soil above the level of the footing may have significant shear strength; the loading on a horizontal foundation may be inclined or eccentrically positioned; the foundation base may be inclined; or the soil surface may be sloped.

The general bearing capacity equation developed for the infinite strip can be extended to consider variations from the basic assumptions by applying modification factors that account for the effect of each variation. The *extended bearing capacity equation* [108] is written

$$q_{net} = cN_c(r_c s_c d_c i_c b_c) + \tfrac{1}{2}B\gamma_1 N_\gamma(r_\gamma s_\gamma d_\gamma i_\gamma b_\gamma) + \gamma_2 D_f(N_q - 1)(r_q s_q d_q i_q b_q) \tag{14-6}$$

where $r_c, r_\gamma, r_q$ = reduction factors to compensate for a possible punching–local shear condition.
$s_c, s_\gamma, s_q$ = shape factors for square, rectangular, and circular foundations.
$d_c, d_\gamma, d_q$ = depth factors.
$i_c, i_\gamma, i_q$ = load inclination factors.
$b_c, b_\gamma, b_q$ = base tilt factors.

Suitable modification factors to use with Equation 14-6 are presented in the following sections. Where a modification factor does not apply for the foundation condition under study, its value for Equation 14-6 is considered to be unity (that is, neglected; factors that do not apply simply drop from the equation).

*In summary, Equation 14-6 should be utilized for calculating the value of $q_{net}$ that is substituted into Equation 14-5 to obtain $q_{design}$.*

### Modification Factors for Square, Rectangular, and Circular Footings

For practical use, the yield zones occurring in the soil beneath an infinitely long footing can be adequately analyzed in a two-dimensional study. Where foundations

REF 3C

weight, and surcharge effects in order to simplify the calculations necessary for foundation design. His equation utilized the concept of dimensionless bearing capacity factors whose values are a function of the shear possessed by the supporting soils. Through ensuing years, the ultimate bearing capacity for shallow and deep foundations has continued to be studied in the quest for refined definition of foundation-soil behavior and a generalized bearing capacity equation which agrees well with failure conditions occurring in model and large-scale foundation tests.[3] Modifications to early concepts have emerged from such studies, but the general form of the Terzaghi bearing capacity equation has been retained because of its practicality.

The ultimate bearing capacity equation, shown as Equation 14-1a and developed from the conditions outlined in Figure 14-3c for long footings, has been found to provide good agreement with ultimate loads observed for model and large-scale foundations. This equation shows the Terzaghi format, but the current dimensionless bearing capacity factors reflect the result of better definition of the soil failure zones:

$$\frac{Q_{ult}}{(B \times L)} \approx \frac{Q''_{ult}}{(B \times 1)} = q_{ult} = cN_c + \frac{1}{2} B\gamma_1 N_\gamma + \gamma_2 D_f N_q \qquad (14\text{-}1a)$$

where $Q_{ult}$ = total loading acting at the base of the strip foundation whose width is $B$ and length is $L$ (kips, kN, etc.).

$Q''_{ult}$ = load acting at base of foundation for unit length in L-direction (k/ft, kN/m, etc.).

$q_{ult}$ = ultimate gross bearing capacity or soil bearing pressure (ksf, kN/m$^2$, etc.).

$c$ = cohesion of the soil below foundation level (ksf, kN/m$^2$, etc.).

$D_f$ = depth of footing below lowest adjacent soil surface (ft, m, etc.).

$\gamma_1$ = effective unit weight of soil below foundation level (pcf, kcf, kN/m$^3$, etc.).

$\gamma_2$ = effective unit weight of soil above foundation level (pcf, kcf, kN/m$^3$, etc.).

$N_c, N_\gamma, N_q$ = soil-bearing capacity factors, dimensionless terms, whose values relate to the angle of internal friction, $\phi$, whereby

$$N_q = \tan^2 (45 + \phi/2)(e)^{\pi \tan\phi}$$

$$N_c = (N_q - 1) \cot\phi \qquad (\text{for } \phi > 0°)$$

$$= 5.14 \qquad (\text{for } \phi = 0°)^4$$

$$N_\gamma = 2(N_q + 1) \tan\phi$$

The relationship between the bearing capacity factors $N_q$, $N_c$, and $N_\gamma$ and the angle of internal friction is presented in Figure 14-4.

[3]Meyerhof [179], Hansen [108], Vesic [298], DeBeer [67].

[4]A rigorous mathematical solution for the value of $N_c$ when $\phi$ equals zero is presented by Jumikis [136], pgs. 626–27.

REF 3C

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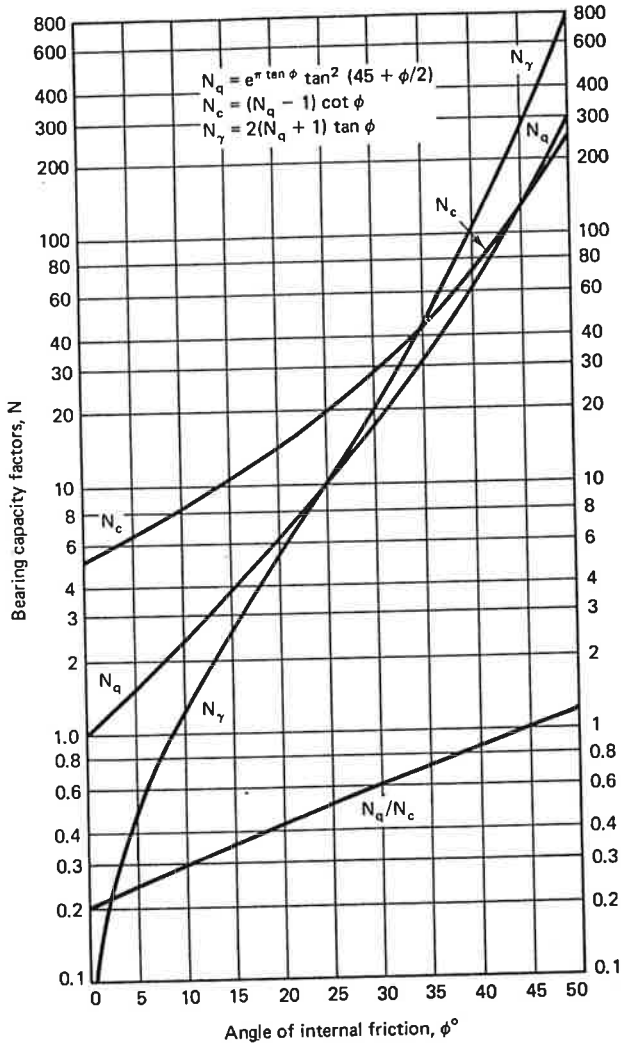



| $\phi$ | $N_c$ | $N_q$ | $N_\gamma$ |
|---|---|---|---|
| 0 | 5.14 | 1.0 | 0.0 |
| 5 | 6.5 | 1.6 | 0.5 |
| 10 | 8.3 | 2.5 | 1.2 |
| 15 | 14.0 | 3.9 | 2.6 |
| 20 | 14.8 | 6.4 | 5.4 |
| 25 | 20.7 | 10.7 | 10.8 |
| 30 | 30.1 | 18.4 | 22.4 |
| 32 | 35.5 | 23.2 | 30.2 |
| 34 | 42.2 | 29.4 | 41.1 |
| 36 | 50.6 | 37.7 | 56.3 |
| 38 | 61.4 | 48.9 | 78.0 |
| 40 | 75.3 | 64.2 | 109.4 |
| 42 | 93.7 | 85.4 | 155.6 |
| 44 | 118.4 | 115.3 | 224.6 |
| 46 | 152.1 | 158.5 | 330.4 |
| 48 | 199.3 | 222.3 | 496.0 |
| 50 | 266.9 | 319.1 | 762.9 |

**Figure 14-4** Bearing capacity factors for the bearing capacity equation.

The first and third terms in this equation represent the Prandtl–Reissner solution for the condition of a weightless soil. The second term represents the separate solution to a determination of the ultimate bearing capacity for a cohesionless soil with weight but no overburden effects (a necessary consideration because the Prandtl–Reissner terms do not include a shearing resistance from the effect of soil weight and internal friction, $\phi$, in the failure zones). This method of superposition (i.e., a simple combination of three terms) to produce Equation 14-1a is not a theoretically correct procedure, but errors are on the safe side and small for most practical applications.

REF 3C

of a finite length are involved, such as with square, rectangular, and circular shapes, the yield zones are three-dimensional and Equations 14-1a, 14-1b, and 14-4 do not directly apply. Mathematical solutions have been managed for the case of a circular footing, and use of a shape factor permits the format of the general bearing capacity equation to be used. Mathematical solutions have not yet been obtained for square and rectangular shapes because of the complexity of the problem, but shape factors are available, as developed from empirical data, to include with the general bearing capacity equation. The shape factors appropriate for square, rectangular, and circular foundations are as follows:

$$s_c = 1 + (B/L)(N_q/N_c) \tag{14-7a}$$

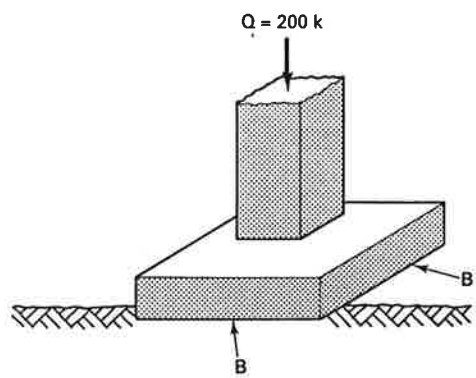
$$s_\gamma = 1 - 0.4(B/L) \tag{14-7b}$$

$$s_q = 1 + (B/L)\tan\phi \tag{14-7c}$$

For the square and circular foundation shapes, the $B/L$ ratio equals unity. Note that for a strip foundation, the $B/L$ value approaches zero and the shape factors become 1.

---

**ILLUSTRATION 14-4.** A square foundation for a column is to carry a loading of 200 kips. Determine the footing dimensions if they bear on the surface of a sand having properties indicated on the sketch. Use a factor safety of 3 for the extended general bearing capacity equation.




Soil properties: sand
$\gamma$ = 130 pcf
$\phi_d$ = 36° (c = 0)
$N_\gamma$ = 50
$I_r$ = 450, $I_{r(critical)}$ = 150

For a square foundation on the surface of sand, with no inclination or tilt,

$$q_{net} = \tfrac{1}{2}B\gamma_1 N_\gamma r_\gamma s_\gamma$$

where $s_\gamma = 1 - 0.4(B/L) = 0.6$. Since $I_r > I_{r_{critical}}$, $r_\gamma = 1$. Obtain

REF 3C

changes in bearing imposed onto the soil beneath the loaded side of the foundation). The effect of eccentric placement of foundation loading can be conveniently and conservatively considered by substitution of a reduced foundation width ($B'$) or length ($L'$) in the bearing capacity equation, using

$$B' = B - 2e_B \tag{14-9a}$$

or

$$L' = L - 2e_L \tag{14-9b}$$

where $e_B$ and $e_L$ are the eccentricity distances with reference to the center of the foundation area (see Fig. 14-10).

The inclined loading can be handled by the inclusion of inclination factors ($i_c$, $i_\gamma$, $i_q$) with the extended bearing capacity equation, whereby

$$i_q = \left[1 - \frac{Q_h}{Q + BLc \cot \phi}\right]^m \tag{14-10a}$$

$$i_r = \left[1 - \frac{Q_h}{Q + BLc \cot \phi}\right]^{m+1} \tag{14-10b}$$

$$i_c = i_q - \left(\frac{1 - i_q}{N_c \tan \phi}\right) \quad (\text{when } \phi > 0°) \tag{14-10c}$$

$$= 1 - \frac{mQ_h}{cN_cBL} \quad (\text{when } \phi = 0°) \tag{14-10d}$$

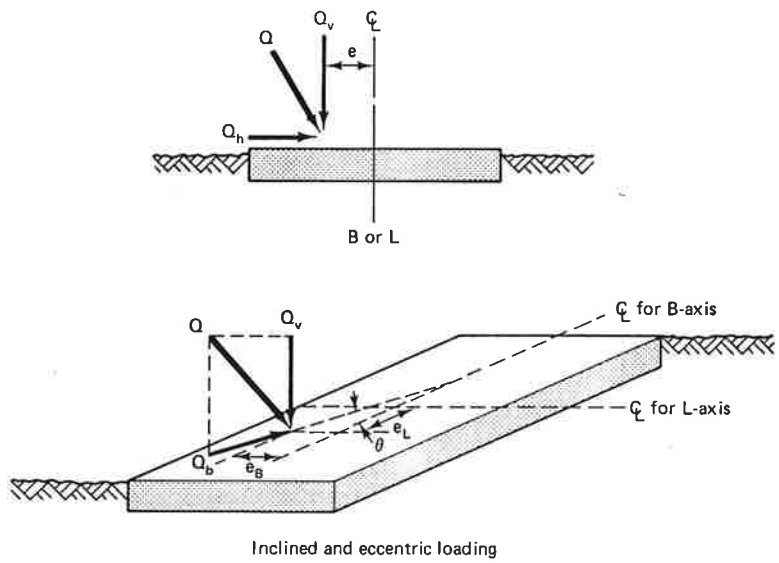



**Figure 14-10** Definition of conditions for eccentric and inclined loading.

REF 3C

Recommended values of $m$ are

$$m_L = (2 + L/B)/(1 + L/B) \quad \text{if the angle of inclination is in the plane of the } L\text{-axis} \tag{14-10e}$$

$$m_B = (2 + B/L)/(1 + B/L) \quad \text{if the angle of inclination is in the plane of the } B\text{-axis} \tag{14-10f}$$

Should the inclined load act in a plane that makes an angle $\theta$ with the $L$-axis of the foundation, the value for $m$ becomes

$$m_\theta = m_L \cos^2\theta + m_B \sin^2\theta \tag{14-10g}$$

When an inclined loading is also eccentric, the modified values $B'$ and $L'$ should be used in Equations 14-10a, b, c, and d. The actual values of $B$ and $L$ are still used to determine the $m$ factors, however.

The horizontal component of an inclined load acts to slide the foundation laterally. If the shearing or frictional resistance developed under the base is not adequate, sliding occurs. The maximum horizontal force that can be resisted along the base of the footing is

$$Q_{h(\max)} = Q_v \tan\delta + A'c_a \tag{14-11}$$

where $A'$ is the effective bearing area of the footing ($B'L'$), $\delta$ is the angle of friction between the foundation material and soil, and $c_a$ is the adhesion between the foundation and soil (for clay, a value related to the undrained shear strength).

**Inclined Foundation Base** Where the base of a foundation is inclined or tilted from the horizontal and an applied load acts normal to the base, the pattern of the soil yield zone beneath the foundation is different from the pattern that develops beneath the level footing carrying a vertical load. For this condition, base tilt factors, $b_c$, $b_\gamma$, and $b_q$, developed originally for the strip footing foundation but considered also appropriate for other foundation shapes, can be applied to the general bearing capacity equation:

$$b_\gamma = \left(1 - \frac{\alpha}{57.3}\tan\phi\right)^2 \tag{14-12a}$$

$$b_q = b_\gamma \tag{14-12b}$$

$$b_c = b_q - \frac{(1 - b_q)}{N_c \tan\phi} \quad (\text{for } \phi > 0°) \tag{14-12c}$$

$$= 1 - .0067\,\alpha \quad (\text{for } \phi = 0°) \tag{14-12d}$$

In these expressions, $\alpha$ represents the angle of the base inclination, in degrees, with respect to the horizontal.

REF 4

# BURIED PIPE DESIGN

A.P. MOSER

REF 4A



plied that stress, strain, deflection, and buckling are independent, but only convenient parameters on which to focus ones attention. For a particular product, certain performance limits are not considered because others will always occur first. The following is a list of performance limits which are often considered in design and could be thought of as possible responses to soil pressure:

Wall crushing (stress)
Wall buckling
Reversal of curvature (deflection)
Overdeflection
Strain limit
Longitudinal stresses
Shear loadings
Fatigue
Delamination

**Wall crushing.** Wall crushing is the terminology used to describe the condition of localized yielding for a ductile material or cracking failure for brittle materials. This performance limit is reached when the in-wall stress reaches the yield stress or the ultimate stress of the pipe material. The ring-compression stress is the primary contributor to this performance limit. (See Fig. 3.14.)

$$\text{Ring compression} = \frac{P_v D}{(2A)} \tag{3.11}$$

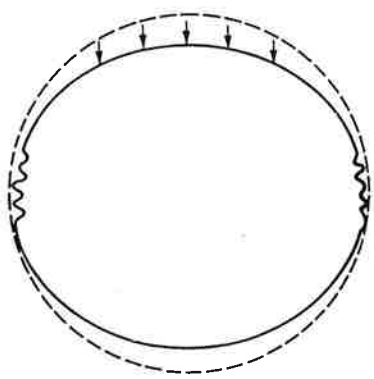


**Figure 3.14** Wall crushing at the 3 and 9 o'clock positions.

REF. 4B



$$P_{cr} = \frac{2}{R}\sqrt{\frac{KEI}{1 - \nu^2}} \tag{3.16}$$

If the "subgrade modulus" $K$ is replaced by the soil stiffness $E'$ we have

$$P_{cr} = 2\sqrt{\frac{E'}{1 - \nu^2}\left(\frac{EI}{R^3}\right)} \tag{3.17}$$

In both Eqs. (3.16) and (3.17) initial out-of-roundness is neglected but the reduction in $P_{cr}$ because of this is assumed to be no greater than 30 percent. As a result, it is recommended that a safety factor of 2 be used with the above formula in the design of a flexible conduit to resist buckling. The Scandinavians have rewritten the above formula for critical buckling pressure as follows:

$$P_{cr} = 1.15\sqrt{P_b E'} \qquad P_b = \frac{2E}{1 - \nu^2}\left(\frac{t^3}{D}\right) \tag{3.18}$$

Actual tests show that while the above equations work fairly well for steel pipe, the equations are conservative for either plastic pipe or fiberglass reinforced plastic pipe. However, one of the above equations should be used for design, keeping in mind that predicted buckling pressure will be on the conservative side for most plastic pipe.

Extreme caution should be used when considering large diameter pipes. The above equations assume the external pressure (or internal vacuum) to be essentially constant around the pipe. This condition is not met when a very large pipe is placed in shallow burial below the water table. In this case, the hydrostatic pressure can vary substantially from the top to the bottom of the pipe.

**Over-deflection.** Deflection is a design parameter for flexible pipes and so-called "semirigid" or "semiflexible" pipes. It is rarely, if ever, considered in the design of rigid pipe installations.

Flexible pipe products will have a deflection design limit (Fig. 3.16). This design limit is not a performance limit, but is often based on a performance limit with a safety factor. For example, PVC pipes will not start a reversal of curvature until about 30 percent deflection. (See Fig. 3.17.) Thus a design deflection of 7.5 percent is based on a safety factor of 4.

Not all design deflections are based on reversal of curvature. For cement-lined steel and ductile iron pipe, the design deflections are based on deflection limits (performance limits) which produce substantial cracking in the cement lining. Other products have deflection limits to limit bending stresses or strains. The design engineer must

REF 4C



recall that the load
a flexible pipe. For
at least 1.5 times
il loads on pipe). If
eflection lag factor

formula is the soil
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lag factor. Inconsis-
ted values of $E'$.
of $E'$ was conducted
of Reclamation.[5]
mpile a table of
ities (see Table 3.4).
the Iowa formula to
heoretical deflection
By assuming the $E'$
1, Howard was able
within ± 2 percent
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= 1.5 in his calcula-
ported, a lag factor
oretically correct. In
field and laboratory
iversity if the prism
ection lag factor. Al-
s study were taken
pipe with diameters
nformation to guide

**TABLE 3.4 Average Values of Modulus of Soil Reaction, $E'$ (For Initial Flexible Pipe Deflection)**

| Soil type-pipe bedding material (Unified Classification System*) | $E'$ for degree of compaction of bedding, lb/in$^2$ | | | |
|---|---|---|---|---|
| | Dumped | Slight, < 85% proctor, < 40% relative density | Moderate, 85%–95% proctor, 40%–70% relative density | High, >95% proctor, > 70% relative density |
| Fine-grained soils (LL > 50)† Soils with medium to high plasticity CH, MH, CH-MH | No data available; consult a competent soils engineer; Otherwise use $E' = 0$ | | | |
| Fine-grained soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with less than 25% coarse-grained particles | 50 | 200 | 400 | 1000 |
| Fine-grained soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with more than 25% coarse-grained particles Coarse-grained soils with fines GM, GC, SM, SC contains more than 12% fines | 100 | 400 | 1000 | 2000 |
| Coarse-grained soils with little or no fines GW, GP, SW, SP‡ contains less than 12% fines | 200 | 1000 | 2000 | 3000 |
| Crushed rock | 1000 | 3000 | 3000 | 3000 |
| Accuracy in terms of percentage deflection§ | ± 2 | ± 2 | ± 1 | ± 0.5 |

*ASTM Designation D2487, USBR Designation E-3

†LL = liquid Limit

‡Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC)

§For ± 1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.

NOTE: Values applicable only for fills less than 50 ft (15 m). Table does not include any safety factor. For use in predicting initial deflections only, appropriate deflection lag factor must be applied for long-term deflections. If bedding falls on the borderline between two compaction categories, select lower $E'$ value or average the two values. Percentage proctor based on laboratory maximum dry density from test standards using about 12,500 ft-lb/ft$^3$ (598,000 J/m$^1$) (ASTM D698, AASHO T-99, USBR Designation E-11). 1 lb/in$^2$ = 6.9 kN/m$^2$.

SOURCE: Amster K. Howard, "Soil Reaction for Buried Flexible Pipe," U.S. Bureau of Reclamation, Denver, Colo. Reprinted with Permission from American Society of Civil Engineers *J. Geotech. Eng. Div.*, January 1977, pp. 33–43.

REF 41



(3.7)

top of pipe, lb/in$^2$
l stiffness, $E_s$ to pipe-
cludes all the proper-
)

on, $I = t^3/12$, then

(3.8)

ıt load in question in

ch terms as $D_L$ and $K$

(3.9)

mple relationship be-
ratio $\Delta y/(D\epsilon)$ and
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100

io.

used for predicting ring deflection. It is based on current theoretical as well as empirical data generated in Europe and America.

In most flexible pipe installations, the pipes are relatively flexible compared to recommended sidefill. Thus, the pipe follows the soil down and the deflection ratio approaches unity. The stiffness ratio, $R_s$, is usually greater than 300 which is to the right of the plot of Fig. 3.11. Even if $R_s$ is usually greater than 300, it is conservative to assume $(\Delta Y/D)/\epsilon$. So the ring deflection becomes

$$\frac{\Delta Y}{D} = \epsilon \tag{3.10}$$

This demonstrates that flexible pipe is deflected down about as much as the sidefill settles. The vertical soil strain in the fill depends upon the soil compressibility and the nominal load (Fig. 3.12). Curves shown in Fig. 3.13 relate soil strain to the soil pressure.

The use of soil strain to predict pipe deflection then becomes a simple exercise. The ratio of pipe deflection to soil strain can be determined from Fig. 3.11. This value will usually be unity for most flexible pipe installations. The load on the pipe is calculated using the prism (embankment) load theory; and the soil strain can be determined from Fig. 3.13.

For the soil to be used as embedment, a series of simple laboratory tests can be run to produce data similar to that which are shown in Fig. 3.13. However, experience has shown that data given in Fig. 3.13 are representative of most soils and can be used for design. Thus it is evident that soil density is the most important parameter in limiting pipe deflection.

### Empirical method

Each of the methods discussed so far for determining load and deflection has a theoretical basis, and except for the prism load theory, all require experimental investigation to determine the unknown con-

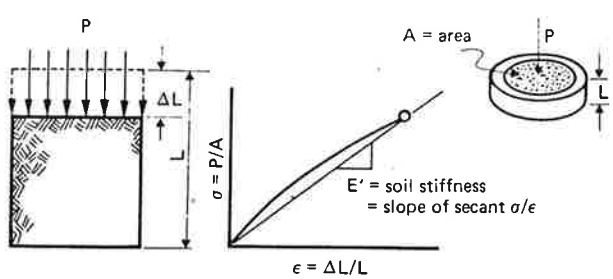


**Figure 3.12** Concept for predicting settlement of soil by means of stress strain compression data from field or laboratory.

REF 4E



Equation (3.4) can be used to predict deflections of buried pipe if the three empirical constants $K$, $D_L$, and $e$ are known. The bedding constant, $K$, accommodates the response of the buried flexible pipe to the opposite and equal reaction to the load force derived from the bedding under the pipe. The bedding constant varies with the width and angle of the bedding achieved in the installation. The bedding angle is shown in Fig. 3.6. Table 3.3 contains a list of bedding factors, $K$, dependent upon the bedding angle. These were determined theoretically by Spangler and published in 1941. As a general rule, a value of $K = 0.1$ is assumed.

In 1958, Reynold K. Watkins, a graduate student of Spangler, was investigating the modulus of passive resistance through model studies and examined the Iowa formula dimensionally.[31] The analysis determined that $e$ could not possibly be a true property of the soil in that its dimensions are not those of a true modulus. As a result of Watkins' effort, another soil parameter was defined. This was the modulus of soil reaction, $E' = er$. Consequently, a new formula called the modified Iowa formula was proposed.

$W_c = \gamma H$

$$\Delta X = \frac{D_L K W_c r^3}{\frac{EI}{r^3} + 0.061E'\cancel{r}} \tag{3.5}$$

Two other observations from Watkins' work are of particular note. (1) There is little point in evaluating $E'$ by a model test and then using this modulus to predict ring deflection as the model gives ring deflection directly. (2) Ring deflection may not be the only performance limit.

Another parameter in the Iowa formula, needed to calculate deflections, is the deflection lag factor $D_L$. Spangler recognized that in soil-pipe systems, as with all engineering systems involving soil, the soil consolidation at the sides of the pipe continues with time after installation of the pipe. His experience had shown that deflections could increase by as much as 30 percent over a period of 40 years. For this reason, he recommended the incorporation of a deflection lag factor of

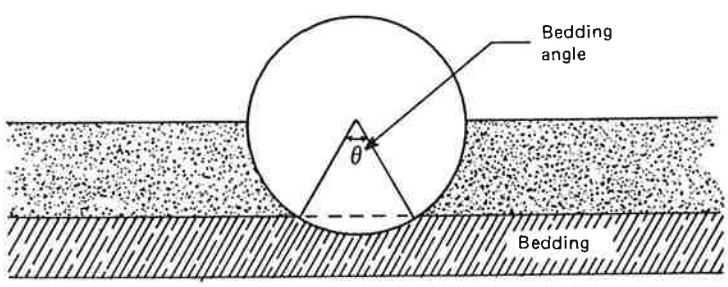


Figure 3.6 Bedding angle.

REF 4F



TABLE 3.3 Values of Bedding Constant, $K$

| Bedding angle, degrees | $K$ |
|---|---|
| 0 | 0.110 |
| 30 | 0.108 |
| 45 | 0.105 |
| 60 | 0.102 |
| 90 | 0.096 |
| 120 | 0.090 |
| 180 | 0.083 |

1.5 as a conservative design procedure. However, recall that the load proposed by Spangler was the Marston load for a flexible pipe. For most sewer pipe installations the prism load is at least 1.5 times greater than the Marston load (see Chap. 2 for soil loads on pipe). If the prism load is used for design, a design deflection lag factor $D_L = 1.0$ should be used.

The remaining parameter in the modified Iowa formula is the soil modulus $E'$. Many research efforts have attempted to measure $E'$ without success. The most useful method has involved the measurement of deflections of a buried pipe for which installation conditions are known, followed by a back calculation through the Iowa formula to determine the effective value of $E'$. This requires assumed values for the load, the bedding factor, and the deflection lag factor. Inconsistent assumptions have led to a variation in reported values of $E'$.

One attempt to acquire information on values of $E'$ was conducted by Amster K. Howard of the United States Bureau of Reclamation.[5] Howard used data from laboratory and field tests to compile a table of average $E'$ values for various soil types and densities (see Table 3.4). He assigned values to $E'$, $K$, and $W_c$ and then used the Iowa formula to calculate a theoretical value of deflection. This theoretical deflection was then compared with actual measurements. By assuming the $E'$ values of Table 3.4 and a bedding constant $K = 0.1$, Howard was able to correlate the theoretical and empirical results to within ± 2 percent deflection when he used the prism soil load. This means that if theoretical deflections using Table 3.4 were approximately 5 percent, measured deflections would range between 3 and 7 percent. Mr. Howard is reported to have used a deflection lag factor $D_L = 1.5$ in his calculations. However if the prism load was used as reported, a lag factor $D_L = 1.0$ would have to have been used to be theoretically correct. In any case, the data in Table 3.4 is consistent with field and laboratory data taken over a 20-year period at Utah State University if the prism load is used along with a value of 1.0 for the deflection lag factor. Although the vast majority of data from Howard's study were taken from tests on steel and reinforced plastic mortar pipe with diameters greater than 24 in, it does provide some useful information to guide

REF 5

*Manual of*

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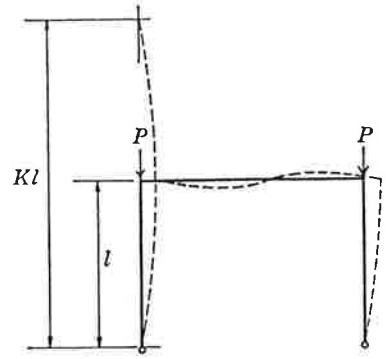
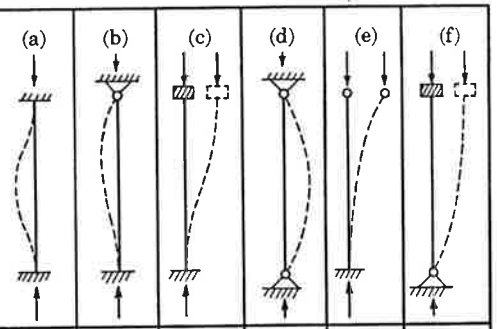



**Fig. C1.8.1**

these conditions are approximated in actual design. In general, these suggested values are slightly higher than their theoretical equivalents, since joint fixity is seldom fully realized.

If the column base in case (f) of Table C1.8.1 were truly pinned, $K$ would actually exceed 2.0 for a frame such as that pictured in Fig. C1.8.1, because the flexibility of the horizontal member would prevent realization of full fixity at the top of the column. On the other hand, it has been shown[14] that the restraining influence of foundations, even where these footings are designed only for vertical load, can be very substantial in the case of flat-ended column base details with ordinary anchorage. For this condition, a design $K$-value of 1.5 would generally be conservative in case (f).

TABLE C1.8.1

| | (a) | (b) | (c) | (d) | (e) | (f) |
|---|---|---|---|---|---|---|
| Buckled shape of column is shown by dashed line |  | | | | | |
| Theoretical $K$ value | 0.5 | 0.7 | 1.0 | 1.0 | 2.0 | 2.0 |
| Recommended design value when ideal conditions are approximated | 0.65 | 0.80 | 1.2 | 1.0 | 2.10 | 2.0 |
| End condition code | | Rotation fixed and translation fixed<br>Rotation free and translation fixed<br>Rotation fixed and translation free<br>Rotation free and translation free | | | | |

1.5.1.2.2 At beam end connections where the top flange is coped, and in similar situations where failure might occur by shear along a plane through the fasteners, or by a combination of shear along a plane through the fasteners plus tension along a perpendicular plane, on the area effective in resisting tearing failure:

$$F_v = 0.30F_u$$

The effective area is the minimum net failure surface, bounded by the bolt holes.*

### 1.5.1.3 Compression

1.5.1.3.1 On the gross section of axially loaded compression members whose cross sections meet the provisions of Sect. 1.9, when $Kl/r$, the largest effective slenderness ratio of any unbraced segment as defined in Sect. 1.8, is less than $C_c$:

$$F_a = \frac{\left[1 - \dfrac{(Kl/r)^2}{2C_c^{\,2}}\right]F_y}{\dfrac{5}{3} + \dfrac{3(Kl/r)}{8C_c} - \dfrac{(Kl/r)^3}{8C_c^{\,3}}} \qquad (1.5\text{-}1)$$

where

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

1.5.1.3.2 On the gross section of axially loaded compression members, when $Kl/r$ exceeds $C_c$:

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} \qquad (1.5\text{-}2)$$

1.5.1.3.3 On the gross section of axially loaded bracing and secondary members, when $l/r$ exceeds 120:**

$$F_{as} = \frac{F_a \text{ [by Formula (1.5-1) or (1.5-2)]}}{1.6 - \dfrac{l}{200r}} \qquad (1.5\text{-}3)$$

1.5.1.3.4 On the gross area of plate girder stiffeners:

$$F_a = 0.60F_y$$

1.5.1.3.5 On the web of rolled shapes at the toe of the fillet (crippling, see Sect. 1.10.10):

$$F_a = 0.75F_y$$

---

* See Commentary Fig. C1.5.1.2.

** For this case, $K$ is taken as unity.

REF 6

# FUNDAMENTALS OF STRUCTURAL STEEL DESIGN

William T. Segui

C.7-50

REF C4

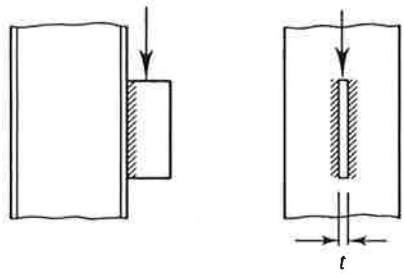



*Figure 6.27*

discussed in Section 1.17 of the AISC Commentary, but none apply to fillet welds made by the shielded metal arc process.

Allowable stresses for welds are given in AISC 1.5.3, Table 1.5.3. The allowable shearing stress in a fillet weld is 0.3 times the tensile strength of the weld metal. For the usual two electrodes, the allowable stresses are therefore as follows:

E70XX: $F_v = 0.3(70) = 21$ ksi

E80XX: $F_v = 0.3(80) = 24$ ksi

An additional requirement is the stipulation that the shearing stress on the base metal shall not exceed $0.40F_y$, where $F_y$ is the yield stress of the base metal (see also AISC 1.5.1.2). This can be explained by examining the welded bracket connection in Fig. 6.27. Assume that the load is close enough to the welded end so that any eccentricity is negligible. If the two welds are the same size, the capacity of the pair of welds per inch of length is

$$2(0.707)(\text{size})F_v$$

and the capacity of the bracket plate in shear per inch of length is

$$t(0.4F_y)$$

Obviously, more load cannot be resisted by the weld than the base metal (the bracket in this case) is capable of carrying. This investigation must always be made when the base metal is subjected to shear. In most cases, both of the connected parts will be subjected to this shear.

### EXAMPLE 6.9

A flat bar used as a tension member is connected to a gusset plate as shown in Fig. 6.28. The welds are 3/16 in. fillet welds made with E70XX electrodes. Assume that the connected parts have adequate strength. Determine the load capacity $P$.

### Solution

Since the welds are placed symmetrically about the axis of the member, this connection qualifies as a simple connection, and each part of the weld will resist an equal share of the load. Only the total length of weld matters, and as long as the resultant applied load passes

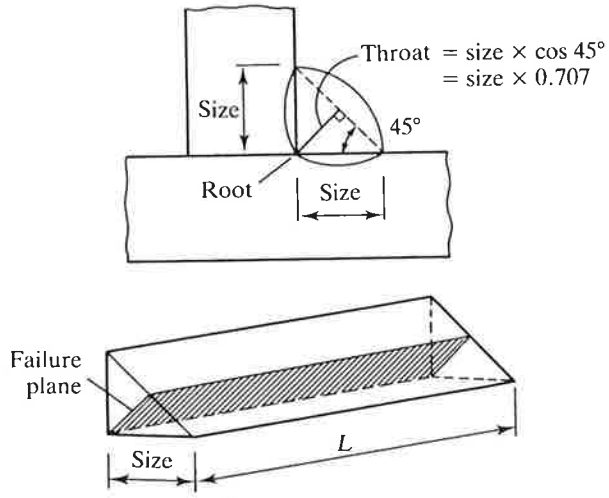



*Figure 6.26*

load of $P$, the critical shearing stress is

$$f_v = \frac{P}{0.707 \times \text{size} \times L}$$

If the allowable shearing stress $F_v$ is used in this equation, the load capacity of the weld can be written as

$$P = 0.707 \times \text{size} \times L \times F_v \tag{6.8}$$

Equation (6.8) is the basic equation for the analysis or design of fillet welds. For design, the required length for a given size or the required size for an available length can be found, subject to specification limits on both size and length.

The strength of a fillet weld depends upon the weld metal used; that is, it is a function of the type of electrode. The strength of the electrode is defined as its ultimate tensile strength, with strengths of 60, 70, 80, 90, 100, and 110 kips per square inch available for the shielded metal arc welding process. The standard notation for specifying an electrode is the letter $E$ followed by two or three digits indicating the strength in kips per square inch and two digits specifying the type of coating. Since strength is the property of primary concern to the design engineer, the last two digits are usually represented by XX, and a typical designation would be E70XX or just E70, indicating an electrode with an ultimate tensile strength of 70 ksi. Electrodes should be selected to match the base metal. For the commonly used grades of steel, only the following two electrodes need be considered:

E70XX electrodes: for steels with a yield stress between 36 ksi and 60 ksi

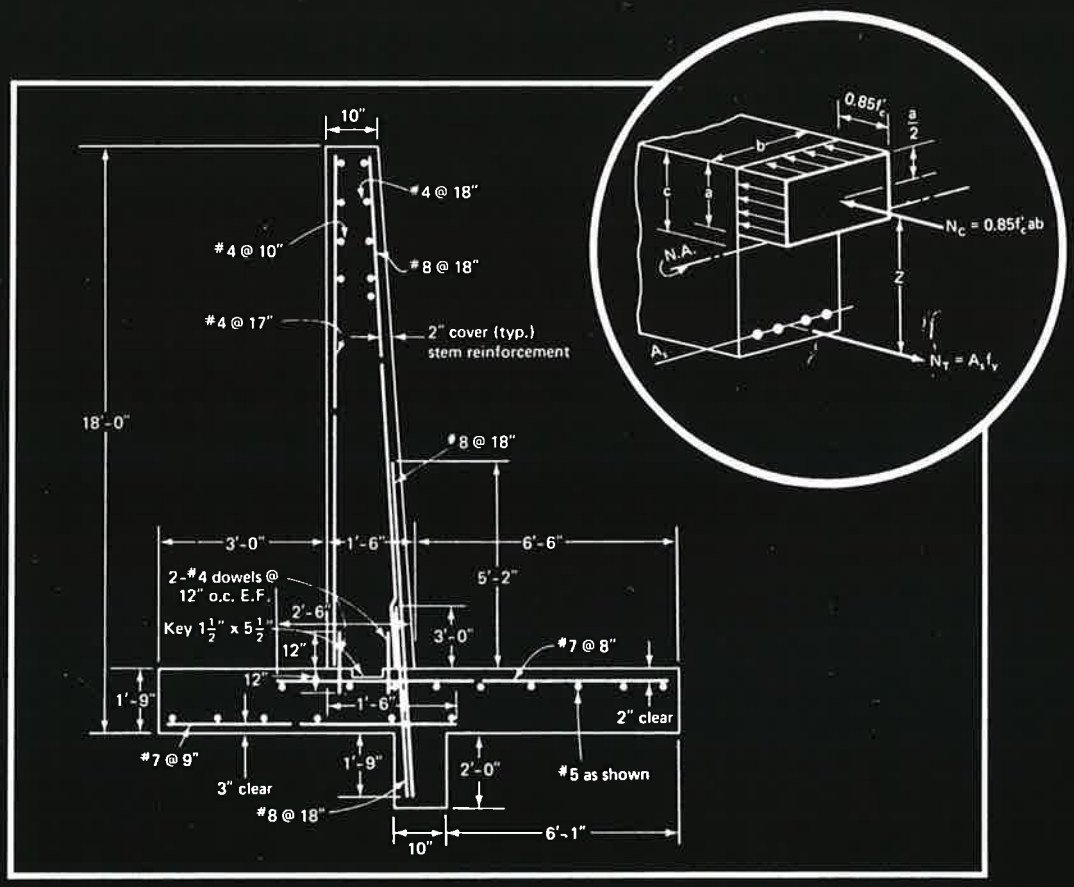
E80XX electrodes: for steels with a yield stress of 65 ksi

The notation for designating electrodes, as well as all provisions of the AISC Specification dealing with welds, has been taken from the *Structural Welding Code* of the American Welding Society [6.6]. Any criteria not covered in the AISC Specification can be found in the AWS Code. Four minor inconsistencies between the two documents are

LEONARD SPIEGEL
GEORGE F. LIMBRUNNER

REF 7

# REINFORCED CONCRETE DESIGN




## THIRD EDITION

REF 7A



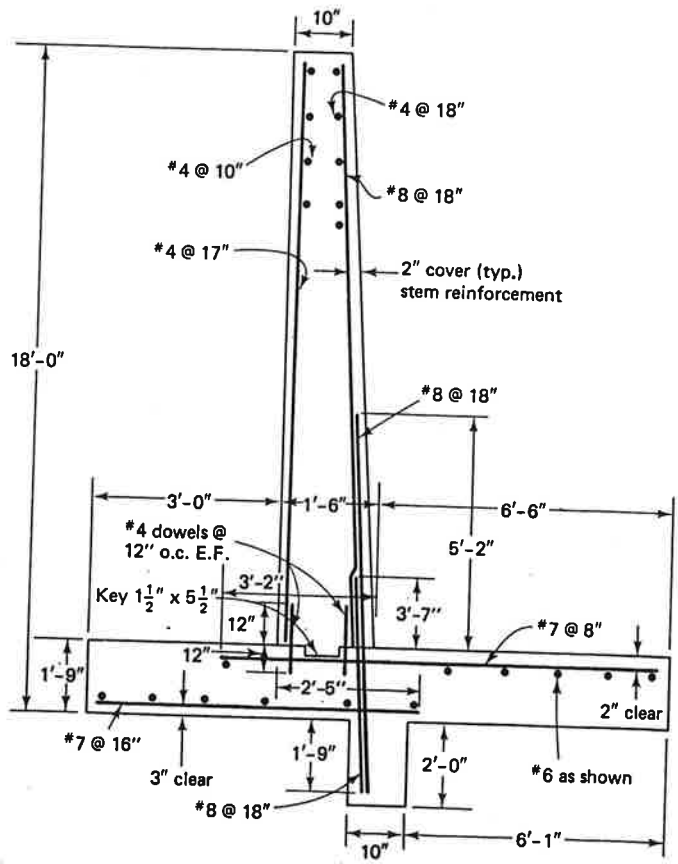



**Figure 8–20** Typical wall section.

The factored shear is

$$V_u = (1.7)(916)(2.0) + 1.7(\tfrac{1}{2})(666)(2) = 4{,}247 \text{ lb}$$

$$V_u < \phi V_n \qquad \text{(O.K.)}$$

## 8–4 DESIGN CONSIDERATIONS FOR BEARING WALLS

Bearing walls [Fig. 8–1(g)] were briefly described at the beginning of this chapter as those walls that carry vertical load in addition to their own weight. Recommendations for the empirical design of such walls are presented in Chapter 14 of the ACI Code (318–89) and apply primarily to relatively short walls spanning vertically, and subject to vertical loads only, such as those resulting from the reactions of floor or roof systems

REF 7A

hap 8

supported on walls. Walls, other than short walls carrying "reasonably concentric" loads, should be designed as compression members for axial load and flexure in accordance with ACI 318–89 Chapter 10. "Reasonably concentric" implies that the resultant factored load falls within the middle third of the cross section.

The design axial load strength or capacity of such a wall will be

$$\phi P_{nw} = 0.55\phi f'_c A_g \left[1 - \left(\frac{k\ell_c}{32h}\right)^2\right] \qquad \text{ACI Eq. (14-1)}$$

where $\phi = 0.70$
$h$ = thickness of wall (in.)
$\ell_c$ = vertical distance between supports (in.)
$A_g$ = gross area of section (in.$^2$)
$k$ = effective length factor

The effective length factor $k$ shall be:

1. For walls braced top and bottom against lateral translation and
   a. Restrained against rotation at one or both ends (top and/or bottom) 0.8
   b. Unrestrained against rotation at both ends 1.0
2. For walls not braced against lateral translation 2.0

Where the wall is subject to concentrated loads, the effective length of wall for each concentration must not exceed the center-to-center distance between loads nor exceed the width of bearing plus four times the wall thickness.

The following requirements applicable to bearing walls are prescribed, among others, by the ACI Code, Chapter 14.

1. Reinforced concrete bearing walls must have a thickness of at least 1/25 of the unsupported height or width, whichever is shorter, and not less than 4 in.
2. Thickness of nonbearing walls shall not be less than 4 in. nor less than 1/30 times the least distance between members that provide lateral support.
3. The area of horizontal reinforcement must be at least 0.0025 times the area of the wall (0.0025$bh$ per foot) and the area of vertical reinforcement not less than 0.0015 times the area of the wall (0.0015$bh$ per foot), where $b$ = 12 in. and $h$ is the wall thickness. These values may be reduced to 0.0020 and 0.0012, respectively, if the reinforcement is not larger than 5/8 in. in diameter and consists of either welded wire fabric or deformed bars with $f_y$ of 60,000 psi or greater.
4. Exterior basement walls and foundation walls must not be less than 7½ in. thick.
5. Reinforced concrete walls must be anchored to floors and roofs or to columns, pilasters, buttresses, and intersecting walls.
6. Walls more than 10 in. thick, except for basement walls, must have reinforcement in each direction for each face. The exterior surface shall have a minimum of one-

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REF 7B

**TABLE A–1** REINFORCING STEEL

| Type of steel and ASTM specification number | Sizes available | Grade | Minimum tensile strength (psi) | $f_y$ (psi) | $\epsilon_y$ |
|---|---|---|---|---|---|
| Billet steel, A615 | Nos. 3–6 | 40 | 70,000 | 40,000 | 0.00138 |
| | Nos. 3–11<br>Nos. 14 and 18 | 60 | 90,000 | 60,000 | 0.00207 |
| | Nos. 11, 14, and 18 | 75 | 100,000 | 75,000 | 0.00259 |
| Rail steel, A616 | Nos. 3–11 | 50<br>60 | 80,000<br>90,000 | 50,000<br>60,000 | 0.00172<br>0.00207 |
| Axle steel, A617 | Nos. 3–11 | 40<br>60 | 70,000<br>90,000 | 40,000<br>60,000 | 0.00138<br>0.00207 |
| Low-alloy steel, A706 | Nos. 3–11<br>Nos. 14 and 18 | 60 | 80,000<br>(Min.: $1.25f_y$) | 60,000<br>(Max.: 78,000) | 0.00207 |

| Bar number | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 14 | 18 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Unit weight per foot (lb) | 0.376 | 0.668 | 1.043 | 1.502 | 2.044 | 2.670 | 3.400 | 4.303 | 5.313 | 7.65 | 13.60 |
| Diameter[a] (in.) | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 | 1.000 | 1.128 | 1.270 | 1.410 | 1.693 | 2.257 |
| Area (in.$^2$) | 0.11 | 0.20 | 0.31 | 0.44 | 0.60 | 0.79 | 1.00 | 1.27 | 1.56 | 2.25 | 4.00 |

[a]The nominal dimensions of a deformed bar (diameter and area) are equivalent to those of a plain round bar having the same weight per foot as the deformed bar.

REF. 8

# DRISCOPIPE®

PHILLIPS 66

C.7-57

DRISCOPIPE

REF. 8A

# Dimensions & Pressure Ratings
## 1000 Series Pipe

| Size | SDR | PSI @73.4 | Weight lbs. per ft | Dimensions - inches Nominal O D | Approx. I D | Min. Wall | Packaging |
|---|---|---|---|---|---|---|---|
| 1" | 11 | 160 | .19 | 1.315 | 1.075 | .120 | 500 ft.coil |
| 1-1/4 | 11 | 160 | .30 | 1.660 | 1.358 | .151 | 500 ft.coil |
| 1-1/2 | 11 | 160 | .40 | 1.900 | 1.554 | .173 | 500 ft.coil |
| 2" | 7 | 267 | .92 | 2.375 | 1.697 | .339 | 88 jts.per bundle |
| | 9 | 200 | .74 | | 1.847 | .264 | |
| | 11 | 160 | .62 | | 1.943 | .216 | 14 bundles per truck |
| | 13.5 | 128 | .52 | | 2.023 | .176 | |
| | 15.5 | 110 | .46 | | 2.069 | .153 | |
| | 17 | 100 | .42 | | 2.095 | .140 | |
| 3" | 7 | 267 | 2.00 | 3.500 | 2.500 | .500 | 46 jts.per bundle |
| | 9 | 200 | 1.62 | | 2.722 | .389 | |
| | 11 | 160 | 1.35 | | 2.864 | .318 | 14 bundles per truck |
| | 13.5 | 128 | 1.12 | | 2.982 | .259 | |
| | 15.5 | 110 | .99 | | 3.048 | .226 | |
| | 17 | 100 | .91 | | 3.088 | .206 | |
| | 19 | 89 | .82 | | 3.132 | .184 | |
| | 21 | 80 | .74 | | 3.166 | .167 | |
| | 26 | 64 | .61 | | 3.230 | .135 | |
| | 32.5 | 51 | .49 | | 3.284 | .108 | |
| 4" | 7 | 267 | 3.31 | 4.500 | 3.214 | .643 | 29 jts.per bundle |
| | 9 | 200 | 2.67 | | 3.500 | .500 | |
| | 11 | 160 | 2.23 | | 3.682 | .409 | 14 bundles per truck |
| | 13.5 | 128 | 1.85 | | 3.834 | .333 | |
| | 15.5 | 110 | 1.63 | | 3.920 | .290 | |
| | 17 | 100 | 1.50 | | 3.970 | .265 | |
| | 19 | 89 | 1.35 | | 4.026 | .237 | |
| | 21 | 80 | 1.23 | | 4.072 | .214 | |
| | 26 | 64 | 1.00 | | 4.154 | .173 | |
| | 32.5 | 51 | .81 | | 4.224 | .138 | |
| 5-3/8" | 21 | 80 | 1.75 | 5.375 | 4.863 | .256 | 15 jts.per bundle |
| | 26 | 64 | 1.43 | | 4.961 | .207 | |
| | 32.5 | 51 | 1.15 | | 5.045 | .165 | 14 bundles per truck |
| 5" | 7 | 267 | 5.05 | 5.563 | 3.973 | .795 | |
| | 9 | 200 | 4.08 | | 4.327 | .618 | |
| | 11 | 160 | 3.42 | | 4.551 | .506 | |
| | 13.5 | 128 | 2.84 | | 4.739 | .412 | |
| | 15.5 | 110 | 2.50 | | 4.845 | .359 | |
| | 17 | 100 | 2.29 | | 4.909 | .327 | |
| | 19 | 89 | 2.07 | | 4.977 | .293 | |
| | 21 | 80 | 1.88 | | 5.033 | .265 | |
| | 26 | 64 | 1.53 | | 5.135 | .214 | |
| | 32.5 | 51 | 1.23 | | 5.221 | .171 | |
| 6" | 7 | 267 | 7.16 | 6.625 | 4.733 | .946 | 13 jts.per bundle |
| | 9 | 200 | 5.78 | | 5.153 | .736 | |
| | 11 | 160 | 4.84 | | 5.421 | .602 | 14 bundles per truck |
| | 13.5 | 128 | 4.03 | | 5.643 | .491 | |
| | 15.5 | 110 | 3.54 | | 5.771 | .427 | |
| | 17 | 100 | 3.25 | | 5.845 | .390 | |
| | 19 | 89 | 2.93 | | 5.927 | .349 | |
| | 21 | 80 | 2.66 | | 5.995 | .315 | |
| | 26 | 64 | 2.17 | | 6.115 | .255 | |
| | 32.5 | 51 | 1.75 | | 6.217 | .204 | |
| 7" | 7 | 267 | 8.29 | 7.125 | 5.089 | 1.018 | 11 jts.per bundle |
| | 9 | 200 | 6.69 | | 5.541 | .792 | |
| | 11 | 160 | 5.61 | | 5.829 | .648 | 12 bundles per truck |
| | 13.5 | 128 | 4.66 | | 6.069 | .528 | |
| | 15.5 | 110 | 4.10 | | 6.205 | .460 | |
| | 17 | 100 | 3.76 | | 6.287 | .419 | |
| | 19 | 89 | 3.39 | | 6.375 | .375 | |
| | 21 | 80 | 3.08 | | 6.445 | .340 | |
| | 26 | 64 | 2.51 | | 6.577 | .274 | |
| | 32.5 | 51 | 2.02 | | 6.685 | .220 | |

Page 1

| Size | SDR | PSI @73.4 | Weight lbs. | Dimensions - inches Nominal O D | Approx. I D | Min. Wall | Packaging |
|---|---|---|---|---|---|---|---|
| 8" | 7 | 267 | 12.14 | 8.625 | 6.161 | 1.232 | 9 jts.per bundle |
| | 9 | 200 | 9.80 | | 6.709 | .958 | |
| | 11 | 160 | 8.21 | | 7.057 | .784 | |
| | 13.5 | 128 | 6.82 | | 7.347 | .639 | 10 bundles per truck |
| | 15.5 | 110 | 6.00 | | 7.513 | .556 | |
| | 17 | 100 | 5.50 | | 7.611 | .507 | |
| | 19 | 89 | 4.96 | | 7.717 | .454 | 3600 ft. per truck |
| | 21 | 80 | 4.52 | | 7.803 | .411 | |
| | 26 | 64 | 3.68 | | 7.961 | .332 | |
| | 32.5 | 51 | 2.97 | | 8.095 | .265 | |
| 10" | 7 | 267 | 18.86 | 10.750 | 7.678 | 1.536 | per truck |
| | 9 | 200 | 15.23 | | 8.362 | 1.194 | |
| | 11 | 160 | 12.75 | | 8.796 | .977 | 80 jts.per loose load |
| | 13.5 | 128 | 10.59 | | 9.158 | .796 | |
| | 15.5 | 110 | 9.33 | | 9.362 | .694 | |
| | 17 | 100 | 8.55 | | 9.486 | .632 | 56 jts.per strip load |
| | 19 | 89 | 7.71 | | 9.618 | .566 | |
| | 21 | 80 | 7.01 | | 9.726 | .512 | |
| | 26 | 64 | 5.71 | | 9.924 | .413 | |
| | 32.5 | 51 | 4.62 | | 10.088 | .331 | |
| 12" | 7 | 267 | 26.53 | 12.750 | 9.108 | 1.821 | per truck |
| | 9 | 200 | 21.44 | | 9.916 | 1.417 | |
| | 11 | 160 | 17.94 | | 10.432 | 1.159 | 52 jts.per loose load |
| | 13.5 | 128 | 14.89 | | 10.862 | .944 | |
| | 15.5 | 110 | 13.12 | | 11.104 | .823 | |
| | 17 | 100 | 12.03 | | 11.250 | .750 | 42 jts.per strip load |
| | 19 | 89 | 10.84 | | 11.408 | .671 | |
| | 21 | 80 | 9.86 | | 11.536 | .607 | |
| | 26 | 64 | 8.04 | | 11.770 | .490 | |
| | 32.5 | 51 | 6.48 | | 11.966 | .392 | |
| 13" | 7 | 267 | 29.24 | 13.386 | 9.562 | 1.912 | per truck |
| | 9 | 200 | 23.62 | | 10.412 | 1.487 | |
| | 11 | 160 | 19.78 | | 10.952 | 1.217 | 48 jts.per loose load |
| | 13.5 | 128 | 16.43 | | 11.402 | .992 | |
| | 15.5 | 110 | 14.46 | | 11.658 | .864 | |
| | 17 | 100 | 13.26 | | 11.812 | .787 | 42 jts.per strip load |
| | 19 | 89 | 11.96 | | 11.976 | .705 | |
| | 21 | 80 | 10.86 | | 12.112 | .637 | |
| | 26 | 64 | 8.87 | | 12.356 | .515 | |
| | 32.5 | 51 | 7.15 | | 12.562 | .412 | |
| 14" | 7 | 267 | 31.99 | 14.000 | 10.000 | 2.000 | per truck |
| | 9 | 200 | 25.84 | | 10.888 | 1.556 | |
| | 11 | 160 | 21.64 | | 11.454 | 1.273 | 48 jts.per loose load |
| | 13.5 | 128 | 17.97 | | 11.926 | 1.037 | |
| | 15.5 | 110 | 15.81 | | 12.194 | .903 | |
| | 17 | 100 | 14.52 | | 12.352 | .824 | 30 jts.per strip load |
| | 19 | 89 | 13.07 | | 12.526 | .737 | |
| | 21 | 80 | 11.90 | | 12.666 | .667 | |
| | 26 | 64 | 9.69 | | 12.924 | .538 | |
| | 32.5 | 51 | 7.83 | | 13.138 | .431 | |
| 16" | 9 | 200 | 33.75 | 16.000 | 12.444 | 1.778 | per truck |
| | 11 | 160 | 28.27 | | 13.090 | 1.455 | |
| | 13.5 | 128 | 23.46 | | 13.630 | 1.185 | 35 jts.per loose load |
| | 15.5 | 110 | 20.65 | | 13.936 | 1.032 | |
| | 17 | 100 | 18.95 | | 14.118 | .941 | |
| | 19 | 89 | 17.07 | | 14.316 | .842 | 30 jts.per strip load |
| | 21 | 80 | 15.53 | | 14.476 | .762 | |
| | 26 | 64 | 12.66 | | 14.770 | .615 | |
| | 32.5 | 51 | 10.21 | | 15.016 | .492 | |

REF 8B

*Simplified Burial Design:* A conservative estimate of the ability of Driscopipe pipelines to perform in a buried environment is found in Chart 24. It is based on a minimum 2:1 safety factor and 50 year design service life. A detailed burial design starts on page 37. The detailed design should be used for critical or marginal applications or whenever a more precise solution is desired.

*Detailed Burial Design:*
*Design by Wall Crushing:* Wall crushing would theoretically occur when the stress in a pipe wall, due to the external vertical pressure, exceeded the long-term compressive strength of the pipe material. To ensure that the Driscopipe wall is strong enough to endure the external pressure the following check should be made:

$$S_A = \frac{(SDR - 1)}{2} P_T$$

## Values of E′

## Based on Soil Type (ASTM D2321) and Degree of Compaction

| Soil Type of Initial Backfill Embedment Material | Description | E′ (psi) for Degree of Compaction (Proctor Density, %) | | | |
|---|---|---|---|---|---|
| | | Loose | Slight (70-85%) | Moderate (85-95%) | High (95%) |
| I | Manufactured angular, granular materials (crushed stone or rock, broken coral, cinders, etc.) | 1,000 | 3,000 | 3,000 | 3,000 |
| II | Coarse grained soils with little or no fines | N.R. | 1,000 | 2,000 | 3,000 |
| III | Coarse grained soils with fines | N.R. | N.R. | 1,000 | 2,000 |
| IV | Fine-grained soils | N.R. | N.R. | N.R. | N.R. |
| V | Organic soils (peat, muck, clay, etc.) | N.R. | N.R. | N.R. | N.R. |

N.R. = Not Recommended for use by ASTM D2321 for pipe wall support

**Chart 24**

| SDR | Maximum Burial Depth, ft. in dry soil of 100 lbs/cu. ft. | | | Maximum External Pressure psi | | | Maximum Deflection, % after installation | | |
|---|---|---|---|---|---|---|---|---|---|
| | Soil Modulus, psi* | | | Soil Modulus, psi* | | | Soil Modulus, psi* | | |
| | 1000 | 2000 | 3000 | 1000 | 2000 | 3000 | 1000 | 2000 | 3000 |
| 32.5 | 25 | 32 | 37 | 17 | 22 | 26 | 1.7 | 0.9 | 0.6 |
| 26 | 33 | 45 | 52 | 23 | 31 | 36 | 2.3 | 1.2 | 0.8 |
| 21 | 46 | 61 | 71 | 32 | 42 | 49 | 3.2 | 1.6 | 1.1 |
| 19 | 52 | 69 | 81 | 36 | 48 | 56 | 3.6 | 1.8 | 1.2 |
| 17 | 61 | 121 | 181 | 42 | 84 | 126 | 4.2 | 2.1 | 1.4 |
| 15.5 | 56 | 112 | 168 | 39 | 78 | 117 | 3.9 | 2.0 | 1.3 |
| 13.5 | 49 | 98 | 147 | 34 | 68 | 102 | 3.4 | 1.7 | 1.1 |
| 11 | 39 | 78 | 117 | 27 | 54 | 81 | 2.7 | 1.4 | 0.9 |
| 9.3 | 33 | 68 | 101 | 23 | 47 | 70 | 2.3 | 1.2 | 0.8 |
| 8.3 | 30 | 61 | 89 | 21 | 42 | 62 | 2.1 | 1.1 | 0.7 |
| 7.3 | 26 | 52 | 79 | 18 | 36 | 55 | 1.8 | 0.9 | 0.6 |

*assumes no external loads

4. Based upon the pipe SDR and the value of the polyethylene modulus of elasticity, E, calculate the pipe's hydrostatic, critical-collapse differential pressure, $P_C$:

$$P_C = \frac{2.32\,(E)}{(SDR)^3}$$

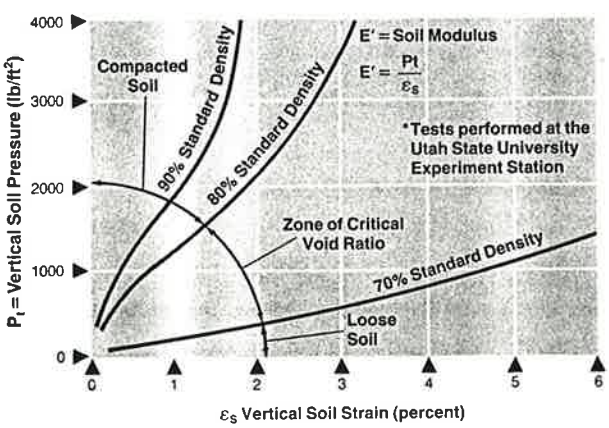
5. Calculate the soil modulus, E′, by plotting the total external soil pressure, $P_t$, against a specified soil density to derive the soil strain as shown in the example problem on Chart 26.
6. Calculate the critical buckling pressure at the top of the pipe by the formula:

$$P_{cb} = 0.8\sqrt{E' \times P_c}$$

7. Calculate the Safety Factor: S.F. $= P_{CB} \div P_t$
   In burial applications, a safety factor of 1.0 may be considered a minimum because of the margin of safety provided by the arching action of the soil. However, Driscopipe endorses using a more conservative value approaching or exceeding a 2.0 safety factor.
8. The above procedures could be reversed to derive the minimum pipe SDR required for a given soil pressure and an estimated soil density. However, this procedure should permit the engineer to optimize the system design quickly by examining several combinations.

**Chart 26**

## Plot of Vertical Stress-Strain Data for Typical Trench Backfill (Except Clay) from Actual Tests*




**EXAMPLE**
**Find: E′ @ 2000 PSF and 80% Density**
**Formula: $E' = P_t/\varepsilon_s$**
**Calculations: E′ = 2000 PSF / .018 = 111111 PSF = 771 psi**

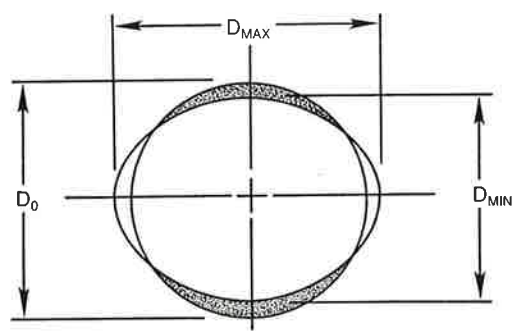
**Note: The curves shown on this chart are sample curves for a granular soil. If other types of soil are used for backfill, such as clay or clay loam, curves should be developed from laboratory test data for the material used. Soil pressures greater than 4000 psf may be examined by extrapolating the slope of the curve or by generating curves by testing at those higher soil pressures. Probable error of curves is about half the distance between adjacent lines.**

*Design by Ring Deflection:* Ring deflection is defined as the ratio of the vertical change in diameter to the original diameter. It is often expressed as a percentage. Ring deflection for buried Driscopipe is conservatively the same as (no more than) the vertical compression of the soil envelope around the pipe. Design by ring deflection matches the ability of Driscopipe to accommodate, without structural distress, the vertical compression of the soil enveloping the buried pipeline. *Design by ring deflection comprises a calculation of vertical soil strain to ensure it will be less than the allowable ring deflection of the pipe.* See Chart 27. The tabulation shows that with lower values of SDR, the allowable deflection is less. For installations which require this thicker wall to resist the external soil pressure, actual ring deflection can easily be limited to the tabular values by proper compaction of the backfill around the pipe. The recommended allowable deflection for the various SDRs are:

**Chart 27**

| SDR | Allowable Ring Deflection |
|---|---|
| 32.5 | 8.1% |
| 26.0 | 6.5% |
| 21.0 | 5.2% |
| 19.0 | 4.7% |
| 17.0 | 4.2% |
| 15.5 | 3.9% |
| 13.5 | 3.4% |
| 11.0 | 2.7% |

The allowable ring deflection of polyethylene pipe is a function of the allowable tangential strain in the outer surface of the pipe wall. A conservative limit of 1-1½% tangential strain in the outer surface of the pipe wall due to vertical deflection of the pipe "ring" by soil compression can be understood by comparing two pipes of the same diameter but different wall thickness.




$$\% \text{ Ring Deflection} = \left(1 - \frac{D_{MIN}}{D_0}\right) \times 100\%$$

**NOTE: 5% deflection decreases flow-area by ¼%. 10% deflection decreases flow-area by 1%.**

REF #9

Elastizell CORPORATION OF AMERICA
P.O. BOX 1462, ANN ARBOR, MICHIGAN 48106
AC 313 / 761-6900

Re: ELASTIZELL EF (Engineered Fill)

I'm pleased to report that ELASTIZELL EF has continued to grow in acceptance with proven results by both public agencies and the private sector. This is due to continuing product research and development, the many and varied successful applications installed by our certified applicators, and the confidence which geotechnical engineers, consultants, contractors, owners and designers have in this unique product.

ELASTIZELL EF is a specialized material formulated specifically by the Elastizell Corporation of America for these geotechnical applications. It is often used in difficult situations where either the standard fill materials are too heavy or alternate solutions are too costly and time consuming as demonstrated by value engineering.

The various applications of ELASTIZELL EF are as broad as the designer's imagination. Let's briefly review some of the most common applications of this material which may weigh as low as 15 pcf, more commonly at 25 pcf and yet is considerably stronger than most soil and fill materials–which are more than four times heavier.

1. **Weight Reducing Fills** – The unique high strength/low density relationship of ELASTIZELL EF means that it is an ideal material to drastically reduce dead loads on a structure while maintaining or increasing accompanying elevations. This is especially important with older structures when new uses are contemplated. This may save the difficult and costly strengthening of existing buildings, bridges, or specialized structures such as culverts.

2. **Bridge Approach Fills** – This is probably the most common application in which ELASTIZELL EF is placed over poor soil at a bridge approach or grade separation. The ELASTIZELL EF thickness at the abutment may be from less than 5 feet to often more than 20 feet. Different densities may be required in various locations of the approach fill to satisfy specific requirements.

3. **Roadway Fills** – ELASTIZELL EF may be applied as a roadway base fill in rehabilitating an existing roadway or for the construction of a new road over poor soil. In maintaining an existing road, the additional weight of roadway leveling patches will often further consolidate the soil leading to additional differential subsidence. For these as well as new roadway sites over poor soils, ELASTIZELL EF provides a solid, yet lightweight roadway base, which reduces further settlements and will span any differential subgrade movement.

4. **Void Fills** – Because of its light weight, flowability, ease of placement, and cementitious properties (no need for compaction), ELASTIZELL EF is ideal for void fill applications. In pipeline construction, these applications may include abandonment fills, annular fills for sliplined pipes, fills between both a pipe and a tunnel cut as well as a pipe and a cribbed excavation. Other void fill applications may be sinkholes, subsealing settlement voids under slabs, and voids created by fill material washouts.

5. **Foundation Fills** – Slab or footing pad fills of ELASTIZELL EF spread loads over larger areas as well as reduce the weight of these supporting structures. These are important considerations over poor soils and can often replace more costly alternatives such as piling, caissons, etc.

6. **Pressure Reducing Fills** – ELASTIZELL EF can be used to reduce lateral loads (bending moments) in retaining walls and other structures. Once ELASTIZELL EF sets, it does not apply lateral pressure against these structures. Depending on its shape and location, the ELASTIZELL EF may be designed to reduce or eliminate active soil pressures on these structures. This may result in substantial savings in the design and construction of new structures and the repair of distressed structures.

In conclusion, ELASTIZELL EF may be the most cost efficient solution on a value engineering basis for your geotechnical problem. ELASTIZELL EF is installed by the immediate pumping of material directly from the automated batching and mixing cycle to the point of placement. This maintains the consistent in-place density of the ELASTIZELL EF to assure a quality Engineered Fill which does not require compaction and is several times lighter than other fill materials. Our development of many varied applications and the practical experience gained from these installations assure you of The Elastizell Advantage.

Please call me so that we may discuss your application, its technical design, and cost information. Thank you for your consideration.

Leo A. Legatski
President

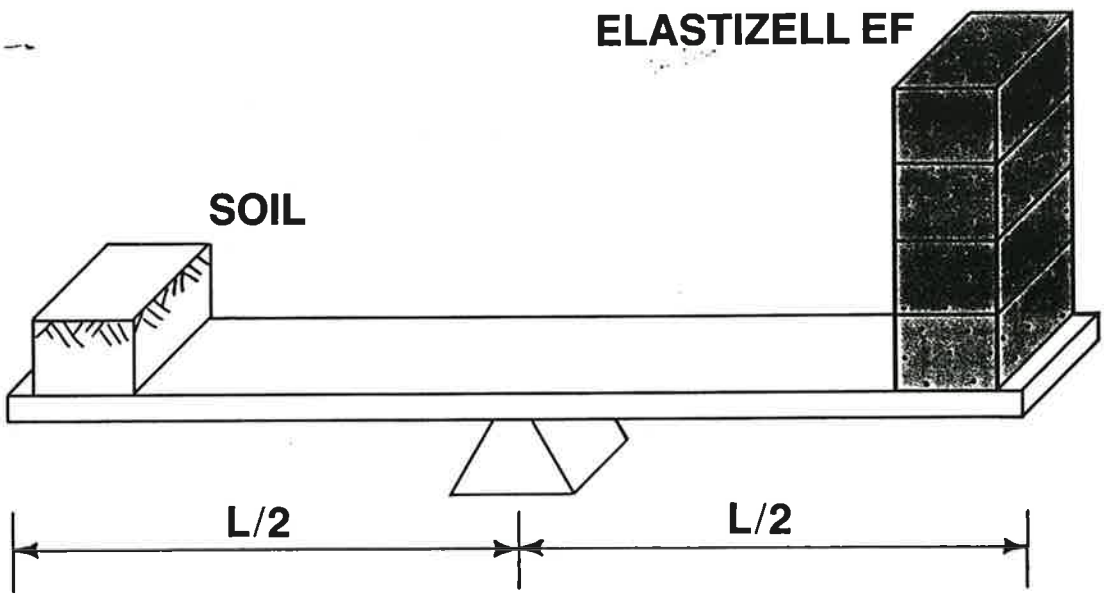
RET 8A
9



# ELASTIZELL EF (Engineered Fill)

## RESEARCH REPORT

## Load Balancing & Load Reduction




## Comparison of Fill Material Densities

CAUTION: For products that may appear similar, demand material test data, in-place performance documentation, and a certified applicator with experienced personnel using approved equipment.

| | |
|---|---|
| **ELASTIZELL EF** | |
| Class I | 18-24 pcf |
| Class II | 24-30 pcf |
| Class III | 30-36 pcf |
| Class IV | 36-42 pcf |
| Water | 62.4 pcf |
| Lightweight Aggregates | 60-90 pcf |
| Soils | 120 pcf |
| Aggregates | 125 pcf |
| Lean Concrete | 145 pcf |

**Elastizell Corporation of America**

# C.8 LCRS FLOW CAPACITY AND PUMP SIZING

# Environmental Solutions, Inc.

By: RVH Date:2/22/96 Beatty Landfill-Cell 12 Page No. 1
Chk'd By: EC Date: 3/8/96 American Ecology Corporation Project No. 95-284

## LCRS Flow Capacity and Pump Sizing

**Purpose:**

1. Determine the potential flow rate into the leachate collection and removal sumps.

2. Size the flow elements (geonets, pipes, and gravel) and pumps to handle the projected flows.

3 Determine most effective method and pump requirements for run-off flows

**Method:**

1. Identify flow sources. The single largest source of liquid entering the sumps will be from infiltration when the cell has been completed but only the 2 foot thick select waste layer has been placed.

2. The design will assume the 25 year-24 hour storm event occurs just after placement of the select waste layer.

3. An infiltration factor will be selected.

4. The quantity of run-off will be calculated. Methods for handling run-off (which shall be considered hazardous waste) will be identified.

5. The quantity of infiltration, and the resultant flow to the sumps will be calculated. The sump flow elements will be sized to handle that flow. Pumps capable of evacuating that flow from the sumps will be identified.

**Summary**

1. 25 Year-24 Hour storm yields 2 inches of precipitation

2. Infiltration flow is controlled by unsaturated flow.

3. The maximum LCRS flow is 2 gallons per minute.

4. The geonet has insufficient capacity for the maximum infiltration flow and must be supplemented by 3 inch diameter slotted pipes (see layout in Figure 3).

5. In run-off systems must be able to store up to171,000 gallons of water, or be capable of pumping up to 100 gpm, if only a nominal impoundment is desired.

6. The maximum requirement for out of cell capacity is 300,000 gallons, assuming no measures to reduce contact between precipitation and waste are utilized.

**Analysis:**

Hydrology

1. The critical period for sizing the leachate collection system (i.e., when the greatest flow is likely to occur) is directly after placement of the 2 foot thick layer of select

## LCRS Flow Capacity and Pump Sizing

waste over the bottom. The quantity of infiltration reaching the LCRS through the waste will be the greatest at this point, although all water touching the waste will be considered leachate. During the construction period precipitation will be impounded within the cell, removed and discharged to the existing channels.

2. Figure 1 shows the configuration of Cell 12. Each phase of the cell is indicated along with slope, bottom, and total catchment areas. Cell 12C includes the top deck area where Cell 12 adjoins Cell 11.

3. The 25 year-24 hour storm yields 2 inches of precipitation (Reference 1A). The working deck is assumed to be well compacted by heavy equipment running over its surface, yielding a relatively high run-off curve number. Since the characteristics of the waste soil are unknown at this time assume soil group B (Reference 1A) and an SCS RCN of 85 (Ref 2A). Further assume run-off from the slopes (with the sacrificial liner) is intercepted and directed to run-off collection tanks/ponds. Table 1, below, shows the distribution for precipitation falling on the floor only:

**Table 1**

| Cell | Area (ac) | Precipitation (ac-ft) | Run-off (ac-ft) | Infiltration (ac-ft) |
|---|---|---|---|---|
| 12A | 2.36 | 0.39 | 0.16 | 0.24 |
| 12B | 1.81 | 0.30 | 0.12 | 0.18 |
| 12C | 1.46 | 0.24 | 0.10 | 0.15 |

LCRS Infiltration:

1. No allowance is made for evaporation, and all infiltration will be routed through the LCRS system. This provides a conservative total volume for the system design.

2. Sandy soils, assumed to comprise the 2 foot protective layer, have a wide range of porosities and moisture retention values. Typical values are taken from the HELP model literature (Reference 3A) as being conservative representations of this soil. The values are:

| | |
|---|---|
| Soil Type | SC |
| Hydraulic Conductivity | $1.2 \times 10^{-4}$ cm/s |
| Porosity | 0.398 |
| Field Capacity | 0.244 |
| Wilting Point | 0.136 |

3. For the largest infiltration case, Cell 12A, there is 0.24 ac-ft, or nearly 80,000 gallons, of infiltration. It is necessary to determine whether saturated or unsaturated flow conditions control drainage into the LCRS. It is assumed that the soil layer contains, at most, sufficient moisture to be at field capacity (i.e., no gravity drainage is initially occuring within the soil). A 2 foot thick layer of soil of unit area therefore has an available pore space of approximately:

$$2ft^3 \times 7.48\frac{gal}{ft^3} \times (0.398 - 0.244) = {}^{2.3gal}\!/_{ft^2}$$

## LCRS Flow Capacity and Pump Sizing

4. The rate at which the storm saturates the soil layer is given by:

$$0.24\text{ac}-\text{ft}\times\frac{1}{2.36\text{ac}}\times 7.48\frac{\text{gal}}{\text{ft}^3}\times\frac{1\text{day}}{1440\text{min}}\cong\frac{1\text{gal}}{1850\text{min}}$$

It would require nearly three days of steady rain at this rate to saturate the 2 foot soil layer assuming no simultaneous drainage occured. Conversely, only 34 percent of the available pore space is filled. Therefore unsaturated flow controls drainage from the 2 foot soil layer. Laminar flow through a porous media is estimated by:

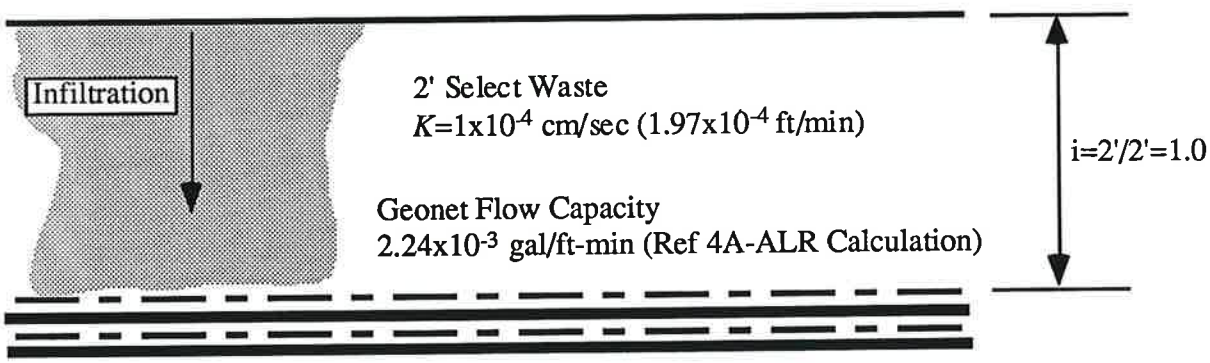
$$Q = Kia \qquad (1)$$

Where:

| | |
|---|---|
| $Q$ | Flow ($ft^3$/min) |
| $K$ | Hydraulic Conductivity (ft/min) |
| i | gradient |
| a | flow area ($ft^2$) |

for the unsaturated case unsaturated hydraulic conductivity, $K_u$, replaces $K$.

5. The unit flow parameters are shown in Figure 2 below:




6 Unsaturated hydraulic conductivity is estimated by Campbell's equation (Reference 3B), definitions for the terms are found in the reference:

$$K_u = K_s\left(\frac{\Theta-\Theta_r}{\phi-\Theta_r}\right)^{3+(\frac{2}{\lambda})}$$

7. The residual water content, $\Theta_r$, is the amount of water remaining in the soil pores under infinite capillary suction. Residual water content is calculated by Rawls (Reference 3C) as follows:

$$\Theta_r = 0.014+.25WP; WP \geq 0.04$$

## LCRS Flow Capacity and Pump Sizing

Where WP is wilting point, which for the designated soil is 0.136>0.04. Therefore the equation is applicable. The residual water content is then:

$$\Theta_r = 0.048$$

8. As discussed in the reference, residual water content and the pore-size distribution index, $\lambda$, are constants in the Brooks-Corey equation relating volumetric content to matrix potential (capillary pressure and adsorptive forces, Reference 3D):

$$\frac{\Theta - \Theta_r}{\phi - \Theta_r} = \left(\frac{\psi_b}{\psi}\right)^{\lambda}$$

9. This equation is solved assuming the volumetric water content is the field capacity, at 0.33 bars of capillary suction, $\psi$, and then the wilting point, at 15 bars capillary suction. Solving simutaneously yields both the pore size distribution index and bubbling pressure. Solving for the values assumed for the site soil yields the following:

**Constants:**

| | |
|---|---|
| $\Theta r$ | 0.048 |
| $\phi$ | 0.398 |

| **Variables** | $\Theta$ | $\psi$ | $\Theta-\Theta r/\phi-\Theta r$ | $(\psi b/\psi)\lambda$ |
|---|---|---|---|---|
| Field Capacity | 0.244 | 0.33 | 0.56 | 0.56 |
| Wilting Point | 0.136 | 15 | 0.25 | 0.25 |

**Solved Values**

| | |
|---|---|
| $\psi b$ | 0.021 |
| $\lambda$ | 0.210 |

10. Since only 34 percent of the available pore space was filled by infiltration the volumetric water content is:

$$\Theta = FC + 0.34(\phi - FC)$$
$$\Theta = 0.244 + 0.34(0.398 - 0.244)$$
$$\Theta = 0.296$$

11. Based on this calculation the unsaturated hydraulic conductivity is calculated from the following using Campbell's equation:

$$K_u = 1\times10^{-4}\frac{\text{cm}}{\text{sec}}\left(\frac{0.296-0.048}{0.398-.048}\right)^{\left(3+\left(\frac{2}{0.210}\right)\right)}$$

$$K_u = 1.361\times10^{-6}\frac{\text{cm}}{\text{sec}} = 2.68\times10^{-6}\frac{\text{ft}}{\text{min}}$$

## LCRS Flow Capacity and Pump Sizing

12. Conservatively using a unit hydraulic gradient and assuming no evaporation, the rate at which the infiltration drains from the 2 foot layer into a unit area of the LCRS is:

$$q = \left(2.68\times10^{-6}\frac{\text{ft}}{\text{min}}\right)(1.0) = 2.68\times10^{-6}\frac{\text{ft}^3}{\text{ft}^2-\text{min}}$$

$$q = 2.68\times10^{-6}\frac{\text{ft}^3}{\text{ft}^2-\text{min}}\left(7.48\frac{\text{gal}}{\text{ft}^3}\right) = 2.00\times10^{-5}\frac{\text{gal}}{\text{ft}^2-\text{min}}$$

Drainage over the entire 2.36 acre cell is then:

$$Q = \left(2.00\times10^{-5}\frac{\text{gal}}{\text{ft}^2-\text{min}}\right)(2.36\text{ac})\left(\frac{43{,}560\text{ft}^2}{\text{ac}}\right)$$

$$Q = \frac{2.06\text{gal}}{\text{min}}$$

13. To accomodate this flow into the LCRS the geonet must have a minimum perimeter of $2.06/2.24\text{x}10^{-3}$=919 feet. Since providing this much flow capacity with geocomposite alone is infeasible a pipe system will be provided to improve drainage characteristics into the LCRS sumps.

14. A single 3 inch diameter corrugated polyethylene pipe (manning n=0.015) at a slope of 1 percent has a flow capacity of approximately 34 gallons per minute (Reference 5A). This is more than sufficient to accomodate the maximum LCRS flow into the sump. As shown in Figure 3, the pipes will be located in along the grade breaks of the bottom with lateral lines located every 100 feet along the grade break length. The basis for the layout is described below.

15. From the manufacturer's literature 3" diameter pipe comes with slots providing 1.44 $\text{in}^2$ (Reference 5B) of flow area per foot. The orifice equation (Reference 6A) is used to estimate flow into the pipe per foot of length. The orifice equation is as follows:

$$Q = CA\sqrt{2gH}$$

The coefficient C=0.6, g=32.2ft/$\text{s}^2$, and H is assumed to be 1 inch. The calculation is therefore:

$$Q = (0.6)(1.44\text{in}^2)\left(\frac{1\text{ft}^2}{144\text{in}^2}\right)\sqrt{2\left(\frac{32.2\text{ft}}{\text{sec}^2}\right)(1\text{in})\left(\frac{1\text{ft}}{12\text{in}}\right)}$$

$$Q = \frac{0.014\text{ft}^3}{\text{sec}}\left(\frac{7.48\text{gal}}{\text{ft}^3}\right)\left(\frac{60\text{sec}}{\text{min}}\right) = 6.2\text{gpm}$$

## LCRS Flow Capacity and Pump Sizing

Since this is substantially more than the maximum estimated flow rate, the spacing of laterals is controlled by the flow capacity of the geocomposite. As stated above the geocomposite can accomodate $2.24 \times 10^{-3}$ gpm per foot of width. Since water percolates into the geocomposite at the rate of 2.00x10-5 gpm per square foot the lateral spacing is calculated as follows:

$$L = \left(2.24 \times 10^{-3} \frac{\text{gal}}{\text{min} - \text{ft}}\right)\left(\frac{\text{min} - \text{ft}^2}{2.00 \times 10^{-5}\,\text{gal}}\right)$$

$$L = 111\text{ft}$$

Use 100 foot spacing for lateral pipes.

16. The LCRS sump is configured as shown in Figure 4. Beginning at the 10 percent slope the sump is filled with clean 1" diameter (nom.) drain rock. To assure unimpeded drainage into the riser the 3" diameter lines will be continued until adjacent to the riser.

17 The LCRS pump must be capable of lifting the leachate vertically approximately 100 feet, the maximum depth of fill above the sump. Head losses in the discharge line are expected to be minimal, allow 5 feet of loss. Minor losses are anticipated to be no more than 20 percent of the subtotaled loss, or approximately 21 feet. Total estimated dynamic head (TDH) is therefore:

$$TDH = 100ft + 5ft + 21ft = 126ft$$

18. Using a factor of safety of 4 for the pump capacity yields 8 gallons per minute. A Protec recipricating pump, Model No. RP-2, with variable speed electric control is recommended (See Reference 7).

Surface Run-Off:

1. During the critical period when waste filling has just started the greatest quantity of run-off is generated by Cell 12A. Approximately 0.16 acre-feet (53,000 gallons) of precipitation runs off the cell bottom. Since the slopes are covered by a sacrificial liner, no infiltration occurs at those surfaces and run-off is equal to the direct precipitation. Since Cell 12A has the largest slope area it is the conservative case. If a gutter system is not used another 0.19 acre-feet (61,000 gallons) runs off the slopes. Handling of run-off is an operational consideration. However, two methods are likely 1) directing the run-off to a temporary lined pond; 2) transferring the liquid to a holding tank.

2. If run-off is directed to a temporary holding pond it must have a capacity of:

(53,000 gal+61,000 gal)(1.5)=171,000 gal

## LCRS Flow Capacity and Pump Sizing

The value 1.5 is a factor of safety to assure adequate containment capacity. The volume of a regular type pond (frustrum of a pyramid) is found by the following formula (Reference 8A):

$$V = \frac{1}{3}h\left(A_{top} + A_{bot} + \sqrt{A_{top}A_{bot}}\right)$$

Assuming a depth of 5 feet, 80 foot sides, and 2:1 slopes yields a volume of

$$V = \frac{1}{3}(5)\left((80)^2 + (80-20)^2 + \sqrt{(80)^2(80-20)^2}\right)$$

$$V = 24{,}667\text{ft}^3\left(\frac{7.48\text{gal}}{\text{ft}^3}\right) = 185{,}000\text{gal} > 171{,}000\text{gal}$$

During the initial fill stages the bottom of the cell will slope towards the sump. The most likly location for a storage pond would be in the adjacent cell area. A pump system, as described below, would be necessary until the sufficient waste has been deposited to create the desired grade.

3. In the event alternative 2 is chosen a suitable pump must be specified. The peak flow into the sump, or other run-off collection area, can be estimated by the SCS unit hydrograph method (Reference 2B) where:

$$Q_p = \frac{484A}{T_R}$$

Total catchment area, A, is shown in Figure 1 to be 3.49 acres ($5.45\text{x}10^{-3}$ mi$^2$). A factor of 484 is used, which is likely conservative given the generally flat nature of the catchment slope.$T_R$ is the rising time and is calculated by the following relationship (Reference 2C):

$$T_R = \frac{D}{2} + t_p$$

The duration of the storm event, D, is 24 hours. The lag time, $t_p$, is found by the following equation (Reference 2D):

$$t_p = \frac{l^{0.8}(S+1)^{0.7}}{1900y^{0.5}}$$

The catchment length, l, is found from Figure 1 to be approximately 450 feet (Cell 12A, east to west). The average catchment slope is 1 percent. The factor S is calculated from the SCS curve number, previously identified as 85. The relationship is
(Reference 2E):

**LCRS Flow Capacity and Pump Sizing**

$$S = \left(\frac{1000}{CN}\right) - 10 = \left(\frac{1000}{85}\right) - 10 = 1.76$$

This implies the lag time is:

$$t_p = \frac{(450)^{0.8}(1.76+1)^{0.7}}{1900(1)^{0.5}} = 0.14\text{hrs}$$

Given that result, the rising time is calculated to be:

$$T_R = \frac{24}{2} + 0.14 = 12.14\text{hrs}$$

and the peak flow is then:

$$Q_p = \frac{484(5.45\times10^{-3}\text{mi}^2)}{12.14} = 0.22\text{cfs}\left(\frac{449\text{gpm}}{\text{cfs}}\right) = 98\text{gpm}$$

4. To successfully transfer the accumulated water without undo ponding, and thus greater infiltration into the LCRS, a small impoundment ( e.g., 10,000-20,000 gallons) and a high capacity pump (e.g. 100 gpm) are required. The pump must be capable of generating sufficient head to lift the water out of the cell and into the desired holding facility. It is assumed the landfill operations staff will size the impoundment and select the necessary pump if this alternative is selected.

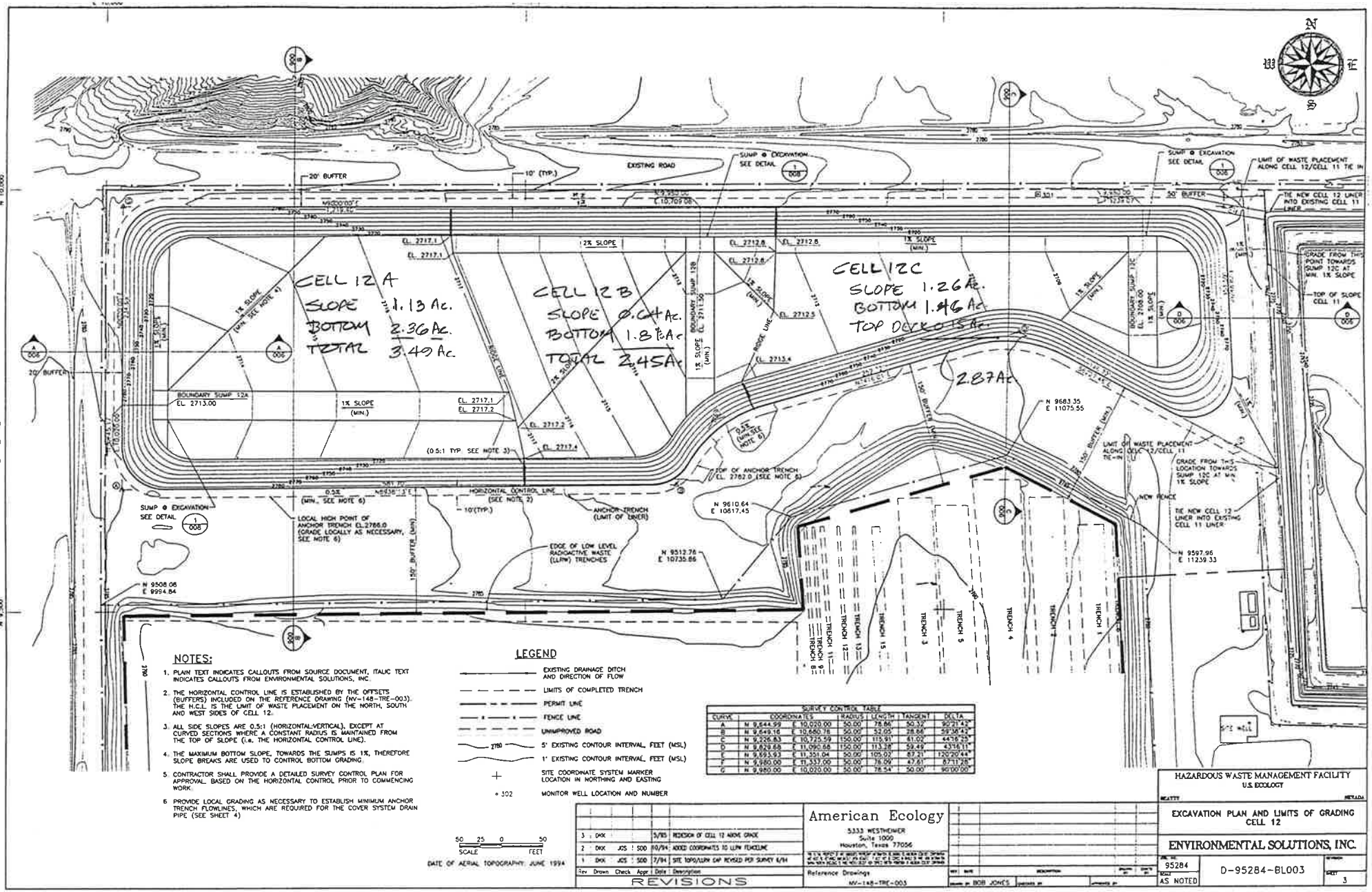
5. Construction of the starter berms, buttress fills and cover system will begin once waste filling is within 3 feet of the top of the excavation slope. The maximum waste area of (8.9 acres) within the horizontal control line (Figure 1) is potentially exposed to precipitation. As a conservative estimate it is assumed no exposure reduction measures are implemented, and the maximum run-off volume is contact water (i.e., has contacted waste).

6. The maximum volume of contact run-off is 198,282 gallons. The three foot freeboard around the facility is sufficient to contain this liquid on the surface of the fill. Should it be necessary to transfer the water out of the cell approximately 300,000 gallons of capacity is required (with a 50 percent factor of safety).

~35'
1 : 0.5 = 2 MIDHEIGHT PERIMETER (MHP)

35' x MHP = SLOPE AREA

SHEET 9 OF 11




## NOTES:

1. PLAIN TEXT INDICATES CALLOUTS FROM SOURCE DOCUMENT, ITALIC TEXT INDICATES CALLOUTS FROM ENVIRONMENTAL SOLUTIONS, INC.
2. THE HORIZONTAL CONTROL LINE IS ESTABLISHED BY THE OFFSETS (BUFFERS) INCLUDED ON THE REFERENCE DRAWING (NV-148-TRE-003). THE H.C.L. IS THE LIMIT OF WASTE PLACEMENT ON THE NORTH, SOUTH AND WEST SIDES OF CELL 12.
3. ALL SIDE SLOPES ARE 0.5:1 (HORIZONTAL:VERTICAL), EXCEPT AT CURVED SECTIONS WHERE A CONSTANT RADIUS IS MAINTAINED FROM THE TOP OF SLOPE (i.e. THE HORIZONTAL CONTROL LINE).
4. THE MAXIMUM BOTTOM SLOPE, TOWARDS THE SUMPS IS 1%, THEREFORE SLOPE BREAKS ARE USED TO CONTROL BOTTOM GRADING.
5. CONTRACTOR SHALL PROVIDE A DETAILED SURVEY CONTROL PLAN FOR APPROVAL, BASED ON THE HORIZONTAL CONTROL PRIOR TO COMMENCING WORK.
6. PROVIDE LOCAL GRADING AS NECESSARY TO ESTABLISH MINIMUM ANCHOR TRENCH FLOWLINES, WHICH ARE REQUIRED FOR THE COVER SYSTEM DRAIN PIPE (SEE SHEET 4)

## LEGEND

- EXISTING DRAINAGE DITCH AND DIRECTION OF FLOW
- LIMITS OF COMPLETED TRENCH
- PERMIT LINE
- FENCE LINE
- UNIMPROVED ROAD
- 2780 5' EXISTING CONTOUR INTERVAL, FEET (MSL)
- 1' EXISTING CONTOUR INTERVAL, FEET (MSL)
- + SITE COORDINATE SYSTEM MARKER LOCATION IN NORTHING AND EASTING
- • 302 MONITOR WELL LOCATION AND NUMBER

SURVEY CONTROL TABLE

| CURVE | COORDINATES | | RADIUS | LENGTH | TANGENT | DELTA |
|---|---|---|---|---|---|---|
| A | N 9,644.99 | E 10,020.00 | 50.00' | 78.54 | 50.00' | 90°01'42" |
| B | N 9,649.16 | E 10,680.78 | 50.00' | 52.05 | 28.66 | 59°38'42" |
| C | N 9,226.83 | E 10,725.59 | 150.00' | 115.91 | 61.02' | 44°16'25" |
| D | N 9,829.68 | E 11,090.66 | 150.00' | 113.28 | 59.49' | 43°16'11" |
| E | N 9,593.93 | E 11,351.04 | 50.00' | 105.02' | 87.21' | 120°20'44" |
| F | N 9,980.00 | E 11,337.00 | 50.00' | 16.09' | 47.61' | 87°17'28" |
| G | N 9,980.00 | E 10,020.00 | 50.00' | 78.54 | 50.00' | 90°00'00" |

50 25 0 50
SCALE FEET

DATE OF AERIAL TOPOGRAPHY: JUNE 1994

| Rev | Drawn | Check | Appr | Date | Description |
|---|---|---|---|---|---|
| 3 | DKK | | | 5/95 | REVISION OF CELL 12 ABOVE GRADE |
| 2 | DKK | JCS | SDD | 10/94 | ADDED COORDINATES TO LLRW TRENCHES |
| 1 | DKK | JCS | SDD | 7/94 | SITE TOPO/LLRW CAP REVISED PER SURVEY 6/94 |

REVISIONS

American Ecology
5333 WESTHEIMER
Suite 1000
Houston, Texas 77056

Reference Drawings
NV-148-TRE-003

DRAWN BY BOB JONES

HAZARDOUS WASTE MANAGEMENT FACILITY
U.S. ECOLOGY
BEATTY NEVADA

EXCAVATION PLAN AND LIMITS OF GRADING CELL 12

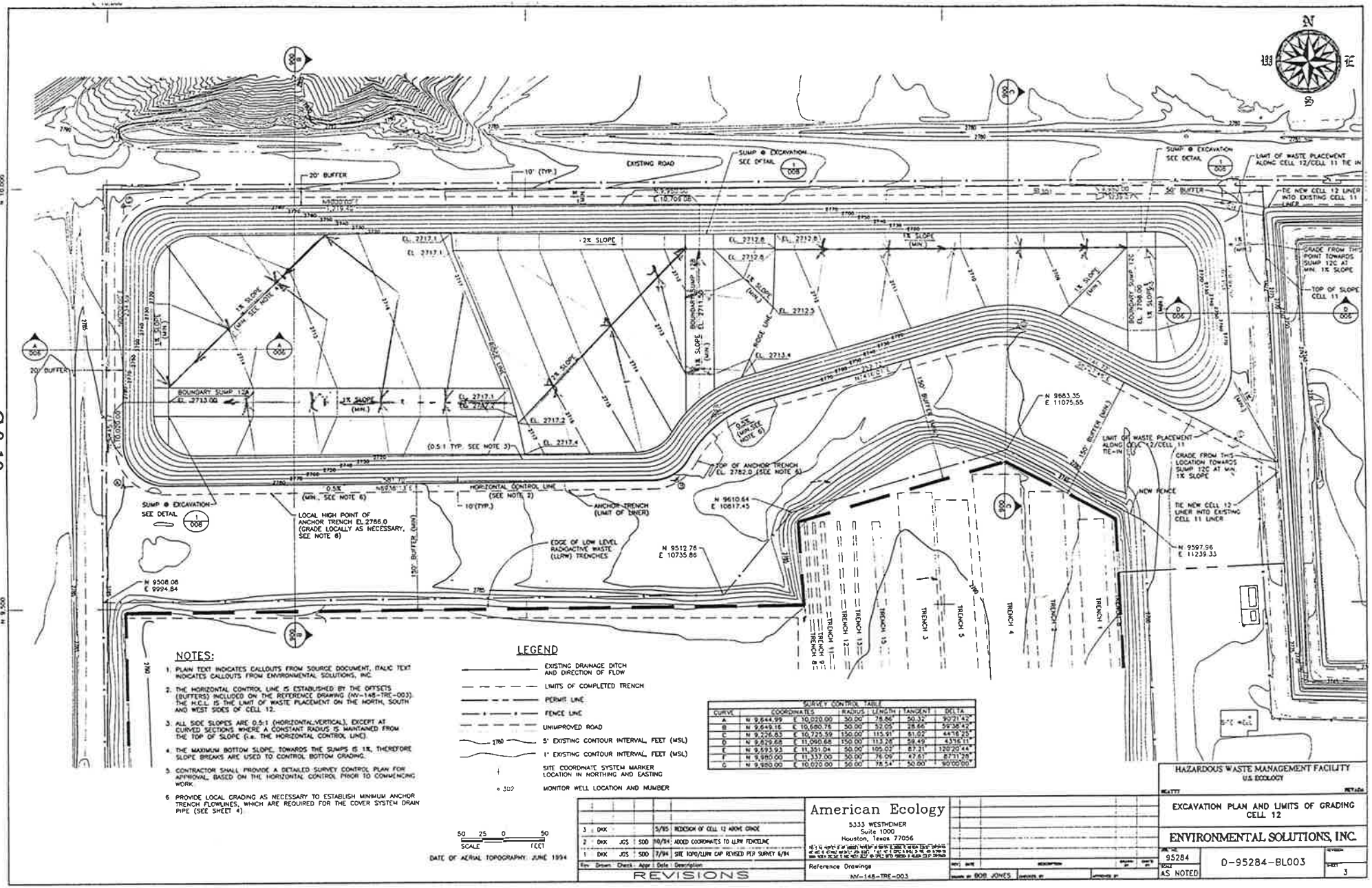
ENVIRONMENTAL SOLUTIONS, INC.

JOB NO. 95284
SCALE AS NOTED
D-95284-BL003
SHEET 3

C.8-9

Figure 1

SHEET 10 OF 11

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## LEGEND

- EXISTING DRAINAGE DITCH AND DIRECTION OF FLOW
- LIMITS OF COMPLETED TRENCH
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- 1780 — 5' EXISTING CONTOUR INTERVAL, FEET (MSL)
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| D | N 9,829.68 | E 11,060.68 | 150.00' | 113.28' | 59.49' | 43°16'11" |
| E | N 9,593.93 | E 11,351.04 | 50.00' | 105.03' | 67.31' | 120°29'44" |
| F | N 9,980.00 | E 11,337.00 | 50.00' | 76.09' | 47.61' | 87°11'28" |
| G | N 9,980.00 | E 10,020.00 | 50.00' | 78.54' | 50.00' | 90°00'00" |

50 25 0 50
SCALE FEET

DATE OF AERIAL TOPOGRAPHY: JUNE 1994

| Rev | Drawn | Check | App | Date | Description |
|---|---|---|---|---|---|
| 3 | DKK | | | 5/95 | REDESIGN OF CELL 12 ANCHOR TRENCH |
| 2 | DKK | JCS | SDD | 10/94 | ADDED COORDINATES TO LLRW FENCELINE |
| 1 | DKK | JCS | SDD | 7/94 | SITE TOPO/LLRW CAP REVISED PER SURVEY 6/94 |

REVISIONS

American Ecology
5333 WESTHEIMER
Suite 1000
Houston, Texas 77056

Reference Drawings
NV-148-TRE-003

DRAWN BY BOB JONES

HAZARDOUS WASTE MANAGEMENT FACILITY
U.S. ECOLOGY

EXCAVATION PLAN AND LIMITS OF GRADING CELL 12

ENVIRONMENTAL SOLUTIONS, INC.

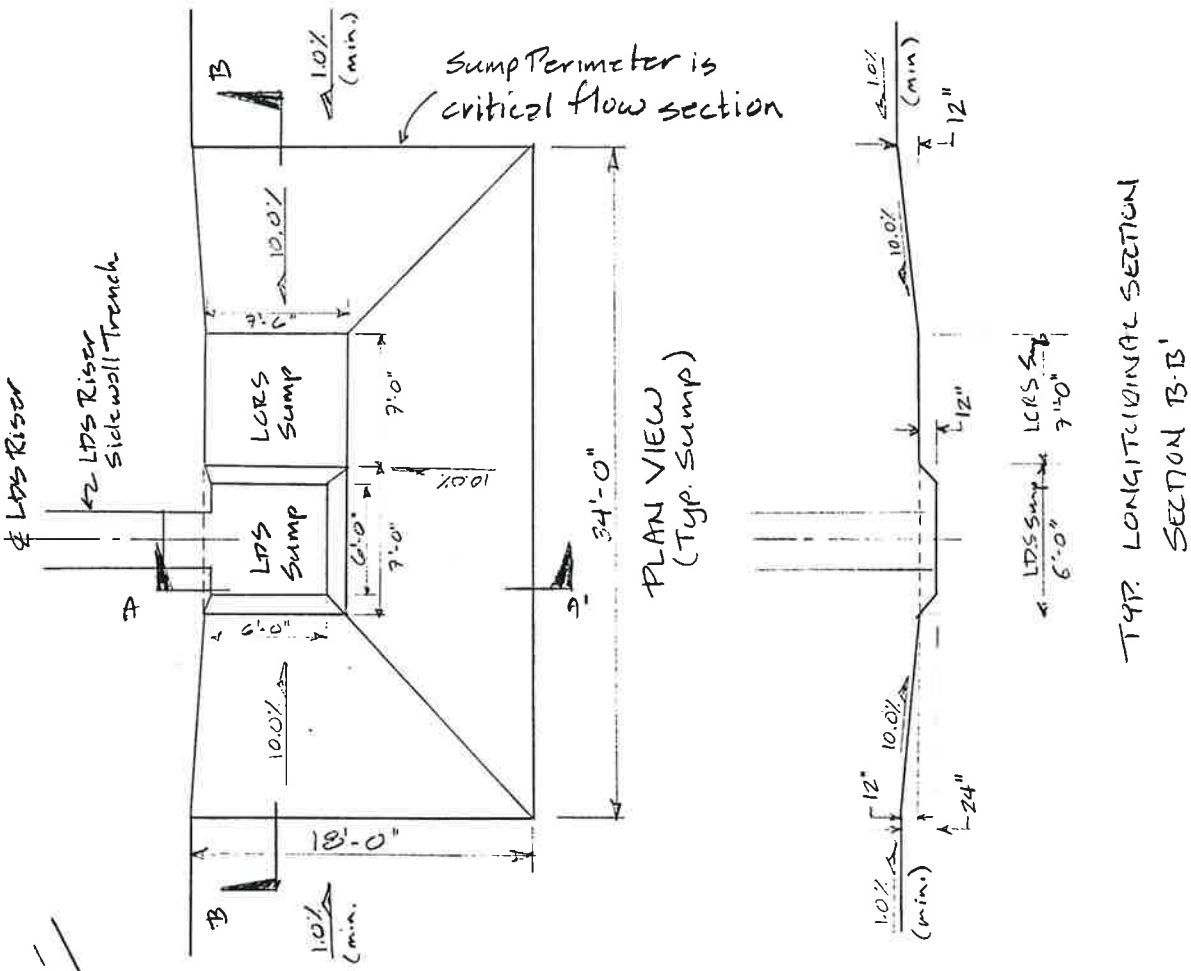
JOB NO. 95284
SCALE AS NOTED
D-95284-BL003
SHEET 3

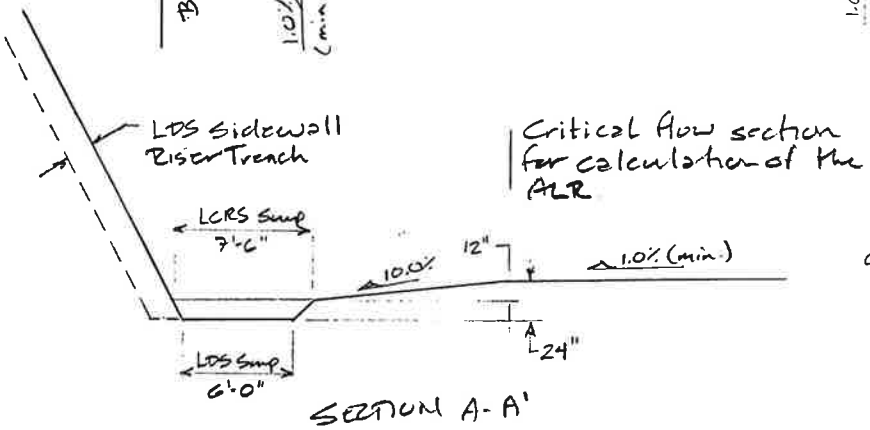
FIGURE 3

# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/20/96 Subject BEATTY LANDFILL-CELL 12 Sheet No. 11 of 11

Chkd. By LMC Date 1/23/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284




SECTION A-A'

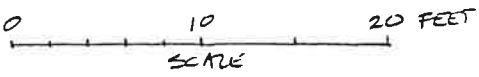



FIGURE 4

TYPICAL SUMP GEOMETRY

Notes:

1) Liner elements & riser pipes not shown for clarity.



Recyclable and made from recycled paper.

REF 1

# SECTION IV.D

# LANDFILL REPORT

US ECOLOGY, INC.

BEATTY, NEVADA

REF 1

# APPENDIX D

# DRAINAGE DIVERSION DITCHES

# DESIGN CALCULATIONS

BEATTY RUN-ON CONTROL SYSTEM DESIGN

Drainage Area Characteristics:

1. Drainage area = 1.7 sq. mi.

2. Max. flow length ($\ell$) = 21,650 ft.

3. Avg. land slope ($\gamma$) = $^{160}/(3.87)\ (5280) = 0.0078 = 0.78\%$

4. Sandy soil, barren land. SCS classifies soils as predominantly hydrological group B with some group A. Therefore, use soil group B and SCS Pasture/Range class with poor hydrologic condition.

5. Use AMC II for worst case.

6. SCS runoff curve No. (RCN) = 79 to account for steps 4 and 5

Compute Time of Concentration (Tc):

$$\text{LAG (L)} = \frac{(\ell)^{.8}\ (S+1)^{.7}}{1900\ (\gamma)^{.5}}$$

$$S = \frac{1000}{RCN} - 10 = \frac{1000}{79} - 10 = 2.66$$

$$L = \frac{(21{,}650)^{.8}\ (3.66)^{.7}}{1900\ (0.78)^{.5}} = 4.35 \text{ hrs.}$$

$$Tc = {}^{L}/0.6 = \frac{4.35}{0.6} = 7.25 \text{ hrs.}$$

Compute Excess Rainfall (Q):

25-yr, 24-hr storm = 2.0 in.

(From NOAA, Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume VII - Nevada)

Q = 0.52 in. for RCN = 79

Compute Peak Flow for 25-yr, 24-hr Rainfall:

Ref: SCS ENGR-20 (Rev. 2)

PK = (D.A.) (Q) (CSM)

CSM = 83 for Tc = 6 hrs, $T_t$ = 0 hrs.
CSM = 72 for Tc = 7.25 hrs by interpolation
CSM = 66 for Tc = 8 hrs, $T_t$ = 0 hrs.

Peak flow = (1.7) (0.52) (72) = 63.65 CFS

# Hydrology and Floodplain Analysis

Philip B. Bedient • Wayne C. Huber

REF 2

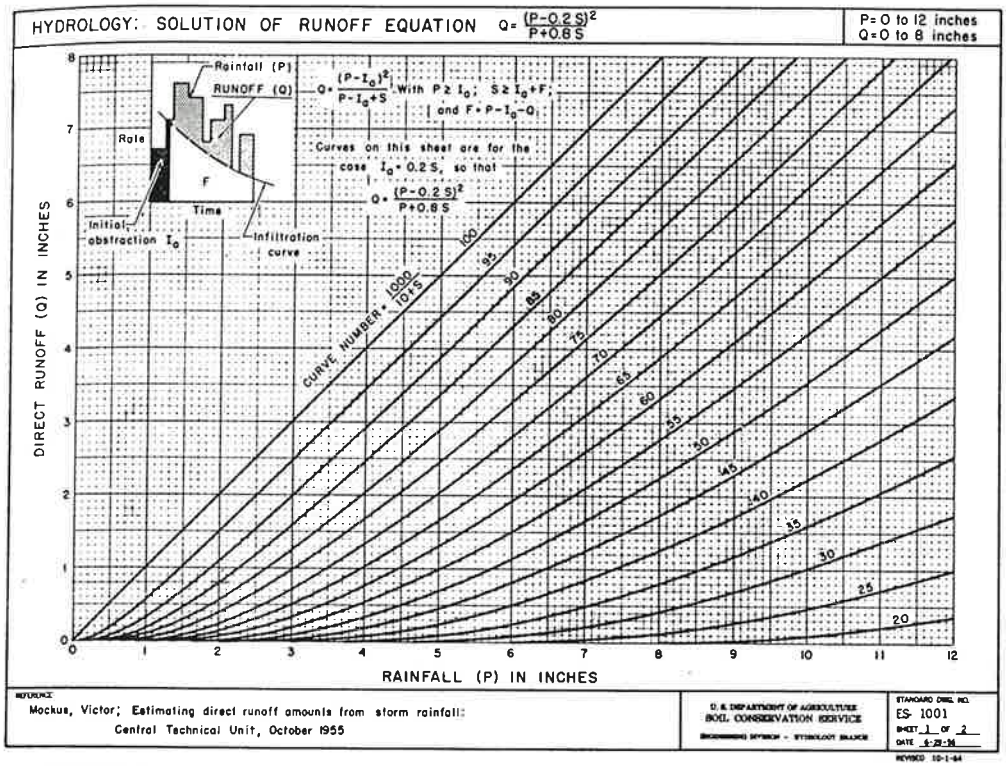



FIGURE 2.14

**Graphical solution of rainfall-runoff equation.**

EXAMPLE 2.8

## SCS UNIT HYDROGRAPH

For the watershed of Example 2.7, develop a unit hydrograph using the SCS method. The watershed consists of meadows in good condition with soil group D. The average slope in the watershed is 100 ft/mi. Assume the same duration of rainfall as found in Example 2.7. Sketch the resulting hydrograph.

SOLUTION

Equation (2.18) gives the following relationship for $t_p$:

$$t_p = \frac{\ell^{0.8}\,(S+1)^{0.7}}{1900y^{1/2}}.$$

From Table 2.1, the SCS curve number is found to be 78. Therefore,

Since this is a small watershed,

$$T_b \approx 4t_p = 4(8.6) \text{ hr}$$

$$\boxed{T_b = 34.4 \text{ hr.}}$$

And the duration of rainfall

$$D = t_p/5.5$$
$$= 8.6/5.5 \text{ hr}$$

$$\boxed{D = 1.6 \text{ hr.}}$$

## SCS Method

The method developed by the Soil Conservation Service (SCS, 1957) is based on a **dimensionless hydrograph,** developed from a large number of unit hydrographs ranging in size and geographic location. The hydrograph is represented as a simple triangle (Fig. 2.13), with rainfall duration $D$ (hr), time of rise $T_R$ (hr), time of fall $B$(hr), and peak flow $Q_p$ (cfs). The volume of direct runoff is

$$\text{Vol} = \frac{Q_p T_R}{2} + \frac{Q_p B}{2}, \qquad \text{or}$$

$$Q_p = \frac{2 \text{ Vol}}{T_R + B} \tag{2.14}$$

From a review of a large number of hydrographs, it was found that

$$B = 1.67\ T_R. \tag{2.15}$$

Therefore, Eq. (2.14) becomes, for 1.0 in. of rainfall excess,

$$Q_p = \frac{0.75 \text{ Vol}}{T_R}$$
$$= \frac{(0.75)(640)A(1.008)}{T_R}$$
$$= \frac{484A}{T_R}, \tag{2.16}$$

REF 2.3

where

$A$ = area of basin (sq mi),

$T_R$ = time of rise (hr).

Capece et al. (1984) found that a factor as low as 10–50 holds for flat, high-water-table watersheds rather than the value 484 presented here. Thus, care must be used when applying this method.

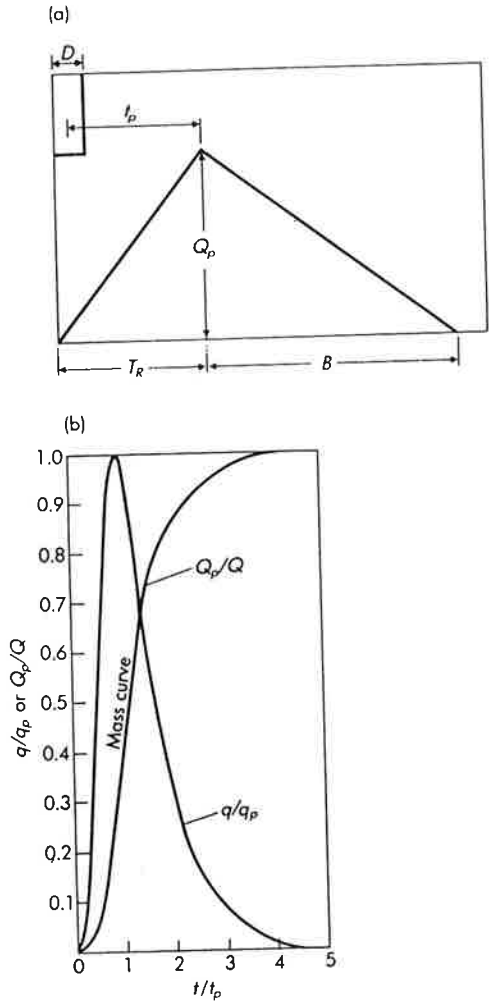



FIGURE 2.13
**(a) SCS triangular unit hydrograph. (b) SCS dimensionless unit hydrograph (SCS, 1964).**

From Fig. 2.13 it can be shown that

$$T_R = D/2 + t_p, \tag{2.17}$$

REF 2C

where

$D$ = rainfall duration (hr),

$t_p$ = lag time from centroid of rainfall to $Q_p$ (hr).

Lag time $t_p$ is estimated from any one of several empirical equations used by the SCS, such as

$$t_p = \frac{\ell^{0.8}(S+1)^{0.7}}{1900y^{0.5}}, \quad \text{REF 20} \tag{2.18}$$

where

$t_p$ = lag time (hr),
$\ell$ = length to divide (ft),
$y$ = average watershed slope (%),
$S = 1000/\text{CN} - 10$,
CN = curve number for various soil/land use (see Table 2.1).

The SCS dimensionless unit hydrograph can be used to develop a curved hydrograph, using the same $t_p$ and $Q_p$ as the triangular hydrograph in Fig. 2.13.

Soil Conservation Service (1964) runoff estimates assume a relationship between accumulated total storm rainfall $P$, runoff $Q$, and infiltration plus initial abstraction $(F + I_a)$. It is assumed that

$$F/S = Q/P_e, \tag{2.19}$$

where $F$ is infiltration occurring after runoff begins, $S$ is **potential abstraction,** $Q$ is direct runoff in in., and $P_e$ is effective storm runoff $(P - I_a)$. With $F = (P_e - Q)$ and $P_e = (P - I_a) = (P - 0.2S)$ based on data from small watersheds,

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}. \tag{2.20}$$

The SCS method uses the **runoff curve number** CN, which is related to storage by

$$\text{CN} = 1000/(S + 10), \quad \text{REF 26} \tag{2.21}$$

where potential abstraction $S$ (in.) becomes

$$S = (1000/\text{CN}) - 10. \tag{2.22}$$

Figure 2.14 presents the SCS solution in graphical form for a range of CNs and rainfall amounts. Runoff curve numbers for selected land uses are presented in Table 2.1. The hydrologic soil group varies from A for sandy, well-drained soils to D for clayey, poorly drained soils. The SCS report *Urban Hydrology for Small Watersheds* (1986) provides a simple graphical and tabular procedure for determining peak flows for urban areas. Example 2.8 illustrates the SCS UH method.

United States
Environmental Protection
Agency

Office of Research and
Development
Washington DC 20460

EPA/600/R-94/168b
September 1994

3
REF

EPA

# The Hydrologic Evaluation of Landfill Performance (HELP) Model

## Engineering Documentation for Version 3

REF 8A
3

TABLE 1. DEFAULT LOW DENSITY SOIL CHARACTERISTICS

| Soil Texture Class | | | Total Porosity vol/vol | Field Capacity vol/vol | Wilting Point vol/vol | Saturated Hydraulic Conductivity cm/sec |
|---|---|---|---|---|---|---|
| HELP | USDA | USCS | | | | |
| 1 | CoS | SP | 0.417 | 0.045 | 0.018 | $1.0 \times 10^{-2}$ |
| 2 | S | SW | 0.437 | 0.062 | 0.024 | $5.8 \times 10^{-3}$ |
| 3 | FS | SW | 0.457 | 0.083 | 0.033 | $3.1 \times 10^{-3}$ |
| 4 | LS | SM | 0.437 | 0.105 | 0.047 | $1.7 \times 10^{-3}$ |
| 5 | LFS | SM | 0.457 | 0.131 | 0.058 | $1.0 \times 10^{-3}$ |
| 6 | SL | SM | 0.453 | 0.190 | 0.085 | $7.2 \times 10^{-4}$ |
| 7 | FSL | SM | 0.473 | 0.222 | 0.104 | $5.2 \times 10^{-4}$ |
| 8 | L | ML | 0.463 | 0.232 | 0.116 | $3.7 \times 10^{-4}$ |
| 9 | SiL | ML | 0.501 | 0.284 | 0.135 | $1.9 \times 10^{-4}$ |
| 10 | SCL | SC | 0.398 | 0.244 | 0.136 | $1.2 \times 10^{-4}$ |
| 11 | CL | CL | 0.464 | 0.310 | 0.187 | $6.4 \times 10^{-5}$ |
| 12 | SiCL | CL | 0.471 | 0.342 | 0.210 | $4.2 \times 10^{-5}$ |
| 13 | SC | SC | 0.430 | 0.321 | 0.221 | $3.3 \times 10^{-5}$ |
| 14 | SiC | CH | 0.479 | 0.371 | 0.251 | $2.5 \times 10^{-5}$ |
| 15 | C | CH | 0.475 | 0.378 | 0.251 | $2.5 \times 10^{-5}$ |
| 21 | G | GP | 0.397 | 0.032 | 0.013 | $3.0 \times 10^{-1}$ |

$a$ = constant representing the effects of various fluid constants and gravity, 21 $cm^3/sec$

$\phi$ = total porosity, vol/vol

$\theta_r$ = residual volumetric water content, vol/vol

$\psi_b$ = bubbling pressure, cm

$\lambda$ = pore-size distribution index, dimensionless

A more detailed explanation of Equation 11 can be found in Appendix A of the HELP program Version 3 User's Guide and the cited references.

### *Saturated Hydraulic Conductivity*

Saturated hydraulic conductivity is used to describe flow through porous media where the void spaces are filled with a wetting fluid (e.g., water). The saturated hydraulic conductivity of each layer is specified in the input. Equations for estimating the hydraulic conductivity for soils and other materials are presented in Appendix A of the HELP Program Version 3 User's Guide.

### *Unsaturated Hydraulic Conductivity*

Unsaturated hydraulic conductivity is used to describe flow through a layer when the void spaces are filled with both wetting and non-wetting fluid (e.g., water and air). The HELP program computes the unsaturated hydraulic conductivity of each soil and waste layer using the following equation, reported by Campbell (1974):

REF 2B 3

$$K_u = K_s \left[ \frac{\theta - \theta_r}{\phi - \theta_r} \right]^{3 + \left(\frac{2}{\lambda}\right)} \tag{5}$$

where

| | | |
|---|---|---|
| $K_u$ | = | unsaturated hydraulic conductivity, cm/sec |
| $K_s$ | = | saturated hydraulic conductivity, cm/sec |
| $\theta$ | = | actual volumetric water content, vol/vol |
| $\theta_r$ | = | residual volumetric water content, vol/vol |
| $\phi$ | = | total porosity, vol/vol |
| $\lambda$ | = | pore-size distribution index, dimensionless |

Residual volumetric water content is the amount of water remaining in a layer under infinite capillary suction. The HELP program uses the following regression equation, developed using mean soil texture values from Rawls et al. (1982), to calculate the residual volumetric water content:

REF 2C 3

$$\theta_r = \begin{cases} 0.014 + 0.25\,WP & \text{for } WP \geq 0.04 \\ 0.6\,WP & \text{for } WP < 0.04 \end{cases} \tag{6}$$

where

| | | |
|---|---|---|
| $WP$ | = | volumetric wilting point, vol/vol |

The residual volumetric water content and pore-size distribution index are constants in the Brooks-Corey equation relating volumetric water content to matrix potential (capillary pressure and adsorptive forces) (Brooks and Corey, 1964):

REF 2D

$$\frac{\theta - \theta_r}{\phi - \theta_r} = \left(\frac{\psi_b}{\psi}\right)^{\lambda} \tag{7}$$

where

$\psi$ = capillary pressure, bars

$\psi_b$ = bubbling pressure, bars

Bubbling pressure is a function of the maximum pore size forming a continuous network of flow channels within the medium (Brooks and Corey, 1964). Brakensiek et al. (1981) reported that Equation 7 provided a reasonably accurate representation of water retention and matrix potential relationships for tensions greater than 50 cm or 0.05 bars (unsaturated conditions).

The HELP program solves Equation 7 for two different capillary pressures simultaneously to determine the bubbling pressure and pore-size distribution index of volumetric moisture content for use in Equation 7. The total porosity is known from the input data. The capillary pressure-volumetric moisture content relationship is known at two points from the input of field capacity and wilting point. Therefore, the field capacity is inserted in Equation 7 as the volumetric moisture content and 0.33 bar is inserted as the capillary pressure to yield one equation. Similarly, the wilting point and 15 bar are inserted in Equation 7 to yield a second equation. Having two equations and two unknowns (bubbling pressure and pore-size distribution index), the two equations are solved simultaneously to yield the unknowns. This process is repeated for each layer to obtain the parameters for computing moisture retention and unsaturated drainage.

### 3.3.3 Saturated Hydraulic Conductivity for Vegetated Materials

The HELP program adjusts the saturated hydraulic conductivities of soils and waste layers in the top half of the evaporative zone whenever those soil characteristics were selected from the default list of soil textures. This adjustment, developed for the model from changes in runoff characteristics and minimum infiltration rates as function of vegetation, is made to account for channeling due to root penetration. These adjustments for vegetation are not made for user-specified soil characteristics; they are made only for default soil textures, which assumed that the soil layer is unvegetated and free of continuous root channels that provide preferential drainage paths. The HELP program calculates the vegetated saturated hydraulic conductivity as follows:

# Environmental Solutions, Inc.

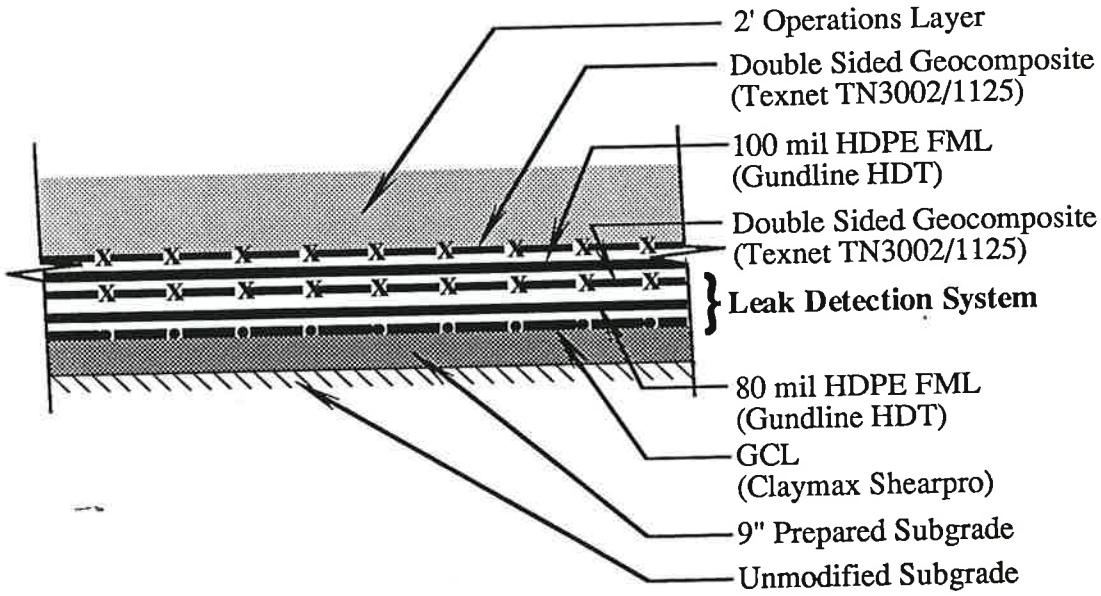
By: RVH
Date: 1/19/96

Subject: Beatty Landfill-Cell 12
American Ecology Corporation

Checked By: EC
Checked On: 3/8/96

## LDS Flow Capacity (Action Leakage Rate) and Pump Sizing

**Figure 1**




2. The sump locations are shown on Figure 2 (Attached). There are three sumps proposed for Cell 12. The typical geometry is shown in Figure 3 (Attached).

Flow Capacities

1. The following flow elements will be used in the LDS

   - Double sided geocomposite (Texnet TN3002/1125)
   - Geonet (Polynet PN3000)-used in the sump areas if additional flow capacity is required.
   - Gravel (clean, poorly graded, nominal 3/4" diameter)-used in the sump proper only.

2. The double sided geocomposite is used in the slope liner LDS as well as in the bottom liner as shown in Figure 1. A continuous strip of geocomposite will be used for each system. Therefore flow within the LDS will be controlled by the minimum bottom slope.

3. Flow within the geosynthetics is calculated using Darcy's Equation (which assumes laminar flow within the net) as follows (Reference 2A):

## LDS Flow Capacity (Action Leakage Rate) and Pump Sizing

$$q = \Theta_{eff} i \qquad (1)$$

Where:

$q$ = flow per unit width

$\Theta_{eff}$ = effective transmissivity

$i$ = hydraulic gradient

Flow within the gravel also uses Darcy's formula, however, transmissivity is replaced with hydraulic conductivity and the thickness of the flow area.

4. Effective transmissivity for the geosynthetics is calculated by applying several safety factors to the published transmissivity value. The following formula (Reference 2B) is used for that calculation, definitions are provided in the attached reference:

$$\Theta_{eff} = \frac{\Theta}{(FS_{CR} \times FS_{IN} \times FS_{CC} \times FS_{BC})} \qquad (2)$$

5. The following table shows the unit flow capacity for the geosynthetic elements and the gravel based on the applicable transmissivity, or hydraulic conductivities, hydraulic gradients, and safety factors:

**Table 1**

| Flow Element | Q | $FS_{CR}$ | $FSI_N$ | $FS_{CC}$ | $FS_{BC}$ | $Q_{eff}$[1] | $i$ | q | q |
|---|---|---|---|---|---|---|---|---|---|
| Units: | m2/s | NA | NA | NA | NA | m2/s | NA | m3/m-s | gal/ft-day |
| TN3002/1125 | 2.20E-04 | 1.4 | 1.5 | 1.5 | 1.5 | 4.66E-05 | 0.01 | 4.66E-07 | 3.24 |
| PN3000 | 2.00E-03 | 1.4 | 1.5 | 1.5 | 1.5 | 4.23E-04 | 0.01 | 4.23E-06 | 29.45 |
| TN3002/1125 | 2.20E-04 | 1.4 | 1.5 | 1.5 | 1.5 | 4.66E-05 | 0.1 | 4.66E-06 | 32.39 |
| PN3000 | 2.00E-03 | 1.4 | 1.5 | 1.5 | 1.5 | 4.23E-04 | 0.1 | 4.23E-05 | 294.47 |

Transmissivity values are provided by the manufacturer (Reference 3A and 3B). Safety factors are taken from the literature and are attached (Reference 2C). Flow capacities are shown at hydraulic gradients of 1 percent and 10 percent for the nominal cell bottom slope and the minimum sump slope, respectively. Since the gravel is used only within the minimum sump boundaries around the riser pipes, flow capacities are calculated for the geosynthethics only.

Controlling Section

1. As shown in Figure 3 there are two potentially controlling sections: 1) the 7'x7' LDS perimeter; and 2) the perimeter at the grade break between the 1 and 10 percent slopes. In the controlling sections multiple layers of geocomposite or geonet alone may be used to provide sufficient flow capacity.

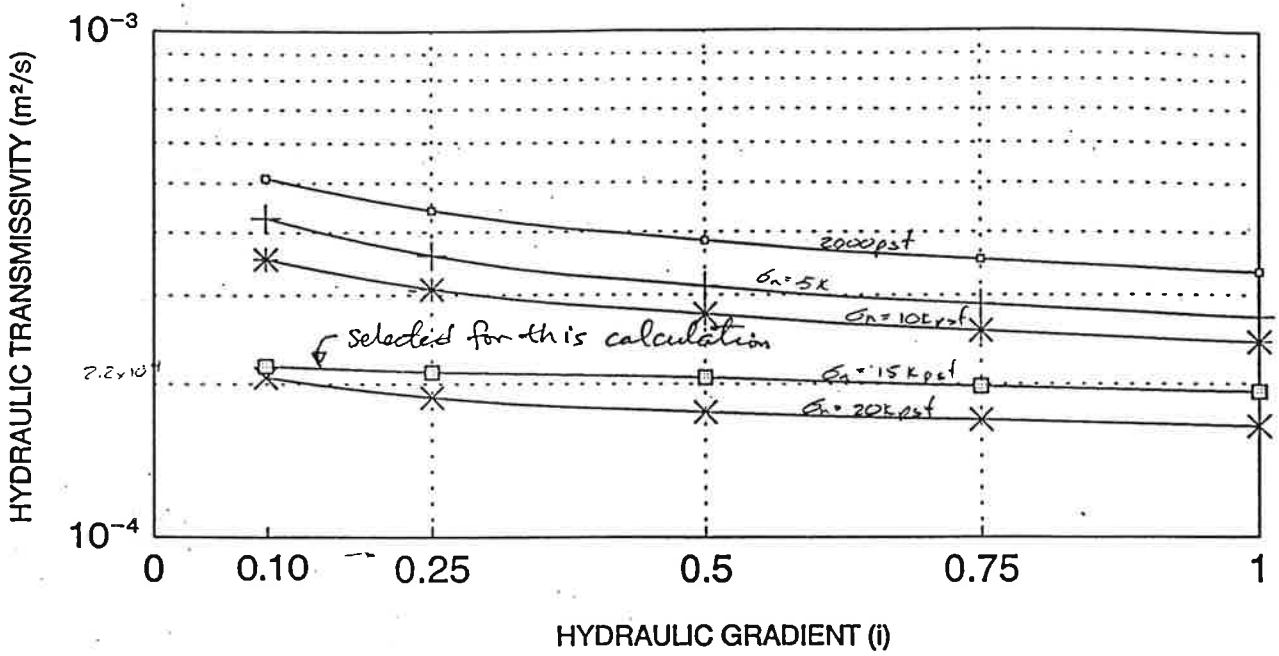
---

[1] The effective transmissivity of the geocomposite (using the factors of safety listed in Table 1) still exceeds the minimum transmissivity requirement ($3 \times 10^{-5}$ $m^2/s$) of 40 CFR § 264.301(c)(3)(ii).

Attachment B

# TEX-NET TN3002/1125

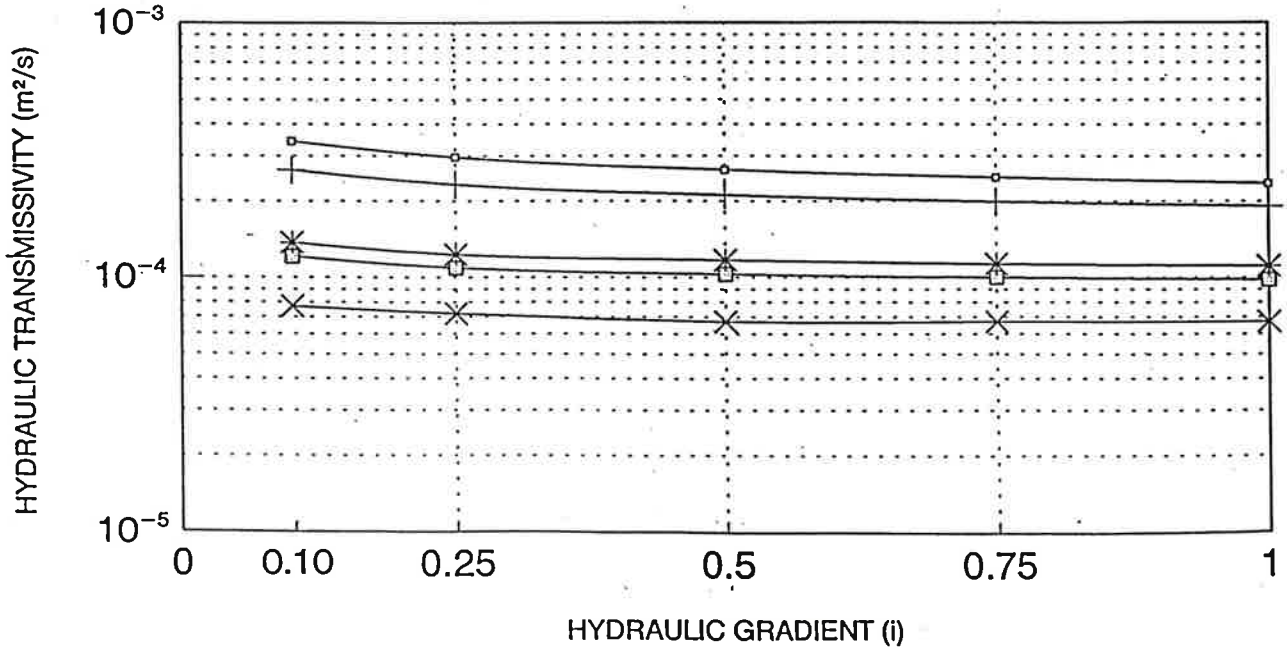
plate/FRICTION SEAL/TN3002/1125/FRICTION SEAL/plate




| | |
|---|---|
| ▫ | 2000 psf |
| + | 5000 psf |
| ✳ | 10000 psf |
| ⊞ | 15000 psf |
| × | 20000 psf |

# TEX-NET TN3002CN/1125

plate/FRICTION SEAL/TN3002CN/1125/FRICTION SEAL/plate



REF # 5

# ADS SPECIFIER MANUAL

## ADVANCED DRAINAGE SYSTEMS, INC.

ADVANCED DRAINAGE SYSTEMS, II

TABLE 7

CIRCULAR PIPE FLOW CAPACITY
Full Flow (cubic feet per second)

Mannings "n"= 0.015

| Dia. (in.) | *Conv. Factor | % Slope (feet per 100 feet) 0.02 | 0.05 | 0.10 | 0.20 | 0.35 | 0.50 | 0.75 | 1.00 | 1.25 | 1.50 | 1.75 | 2.0 | 2.5 | 5.0 | 10.0 | 20.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | (c.f.s.) | | | | | | | | |
| 3 | 0.766 | 0.011 | 0.017 | 0.024 | 0.034 | 0.045 | 0.054 | 0.066 | 0.077 | 0.086 | 0.09 | 0.10 | 0.11 | 0.12 | 0.17 | 0.24 | 0.34 |
| 4 | 1.649 | 0.023 | 0.037 | 0.052 | 0.074 | 0.098 | 0.117 | 0.143 | 0.165 | 0.184 | 0.20 | 0.22 | 0.23 | 0.26 | 0.37 | 0.52 | 0.74 |
| 5 | 2.991 | 0.042 | 0.067 | 0.095 | 0.134 | 0.177 | 0.211 | 0.259 | 0.299 | 0.334 | 0.37 | 0.40 | 0.42 | 0.47 | 0.67 | 0.95 | 1.34 |
| 6 | 4.863 | 0.069 | 0.109 | 0.154 | 0.217 | 0.288 | 0.344 | 0.421 | 0.486 | 0.544 | 0.60 | 0.64 | 0.69 | 0.77 | 1.09 | 1.54 | 2.17 |
| 8 | 10.473 | 0.148 | 0.234 | 0.331 | 0.468 | 0.620 | 0.741 | 0.907 | 1.047 | 1.171 | 1.28 | 1.39 | 1.48 | 1.66 | 2.34 | 3.31 | 4.68 |
| 10 | 18.99 | 0.27 | 0.42 | 0.60 | 0.85 | 1.12 | 1.34 | 1.64 | 1.90 | 2.12 | 2.33 | 2.51 | 2.69 | 3.00 | 4.25 | 6.00 | 8.49 |
| 12 | 30.88 | 0.44 | 0.69 | 0.98 | 1.38 | 1.83 | 2.18 | 2.67 | 3.09 | 3.45 | 3.78 | 4.08 | 4.37 | 4.88 | 6.90 | 9.76 | 13.81 |
| 15 | 55.98 | 0.79 | 1.25 | 1.77 | 2.50 | 3.31 | 3.96 | 4.85 | 5.60 | 6.26 | 6.86 | 7.41 | 7.92 | 8.85 | 12.52 | 17.70 | 25.04 |
| 18 | 91.04 | 1.29 | 2.04 | 2.88 | 4.07 | 5.39 | 6.44 | 7.88 | 9.10 | 10.18 | 11.15 | 12.04 | 12.87 | 14.39 | 20.36 | 28.79 | 40.71 |
| 21 | 137.32 | 1.94 | 3.07 | 4.34 | 6.14 | 8.12 | 9.71 | 11.89 | 13.73 | 15.35 | 16.82 | 18.17 | 19.42 | 21.71 | 30.71 | 43.43 | 61.41 |
| 24 | 196.06 | 2.77 | 4.38 | 6.20 | 8.77 | 11.60 | 13.86 | 16.98 | 19.61 | 21.92 | 24.01 | 25.94 | 27.73 | 31.00 | 43.84 | 62.00 | 87.68 |
| 27 | 268.41 | 3.80 | 6.00 | 8.49 | 12.00 | 15.88 | 18.98 | 23.24 | 26.84 | 30.01 | 32.87 | 35.51 | 37.96 | 42.44 | 60.0 | 84.9 | 120.0 |
| 30 | 355.48 | 5.03 | 7.95 | 11.24 | 15.90 | 21.03 | 25.14 | 30.79 | 35.55 | 39.74 | 43.54 | 47.03 | 50.27 | 56.21 | 79.5 | 112.4 | 159.0 |
| 36 | 578.05 | 8.17 | 12.93 | 18.28 | 25.85 | 34.20 | 40.87 | 50.06 | 57.81 | 64.63 | 70.80 | 76.47 | 81.75 | 91.40 | 129.3 | 182.8 | 258.5 |
| 42 | 872.0 | 12.33 | 19.50 | 27.57 | 38.99 | 51.6 | 61.7 | 75.5 | 87.2 | 97.5 | 106.8 | 115.3 | 123.3 | 137.9 | 195.0 | 275.7 | 389.9 |
| 48 | 1244.9 | 17.61 | 27.84 | 39.37 | 55.67 | 73.6 | 88.0 | 107.8 | 124.5 | 139.2 | 152.5 | 164.7 | 176.1 | 196.8 | 278.4 | 393.7 | 556.7 |

* Conveyance Factor = (1.486 x R2/3 x A) / n



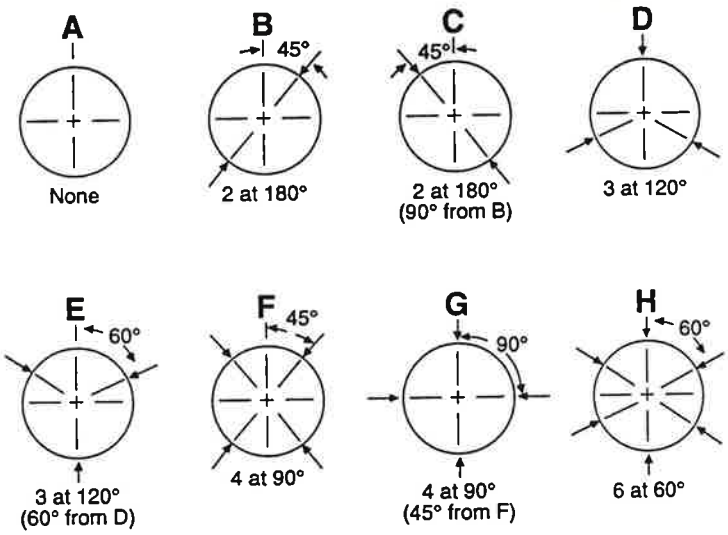
REF 44
5

REF 5B

# ADVANCED DRAINAGE SYSTEMS, INC.
## "FACT SHEET"
## STANDARD PIPE PERFORATIONS
## 3-24" I.D. SINGLE WALL PIPE

| Nominal I.D. | Perforation Type | Slot Length or Diameter (Max.) | Slot Width (Max.) | Nominal Water Inlet Area (Sq. In./Ft.) | Perforation Configuration |
|---|---|---|---|---|---|
| 3 | Slot | 1.25 | 0.125 | 1.44 | AF |
| 4 | Slot | 1.25 | 0.125 | 2.01 | AH |
| 5 | Slot | 1.57 | 0.125 | 2.01 | AH |
| 6 | Slot | 1.88 | 0.125 | 2.01 | AH |
| 8 | Slot | 2.50 | 0.125 | 1.57 | AD |
| 10 | Slot | 2.50 | 0.125 | 1.26 | AD |
| 12 | Slot | 2.50 | 0.125 | 3.38 | H |
| 15 | Slot | 2.50 | 0.125 | 1.58 | H |
| 18 | Circular | 0.375 | — | 1.58 | H |
| 24 | Circular | 0.375 | — | 1.58 | 11" Centers |

PERFORATION CONFIGURATIONS



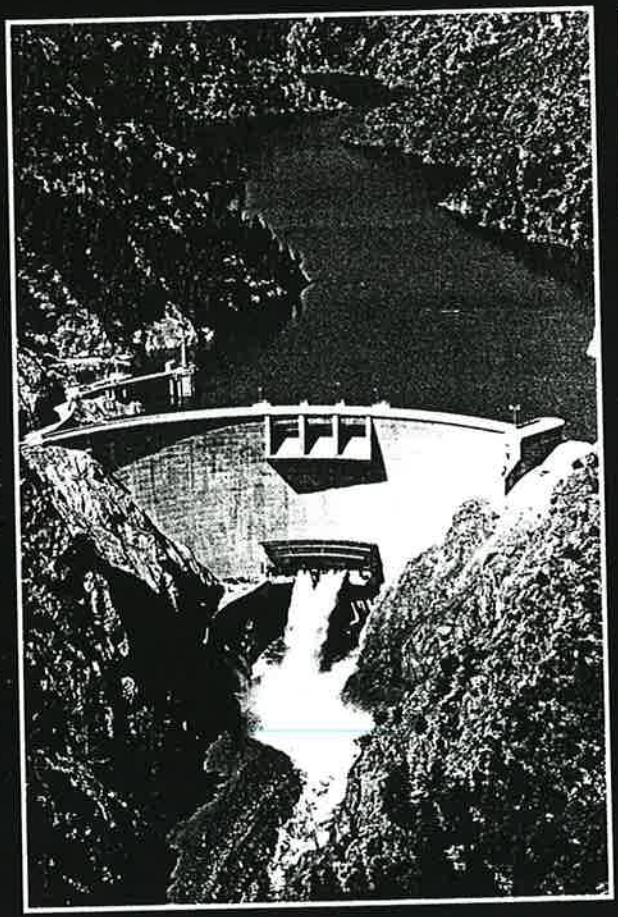

NOTE 1
ADS pipe is perforated for water entry with slots or circular perforations. The perforations are uniformly spaced along the length and circumference of the pipe.

NOTE 2
Unless otherwise specified, ADS pipe is manufactured to comply with the perforation requirements specified in the following industry standards: ASTM F405, ASTM F667, AASHTO M252, AASHTO M294, and SCS Code 606.

Issued: March 15, 1991

REF # 4

# DAVIS' HANDBOOK OF APPLIED HYDRAULICS



FOURTH EDITION

Vincent J. Zipparro
Hans Hasen

REF 8A

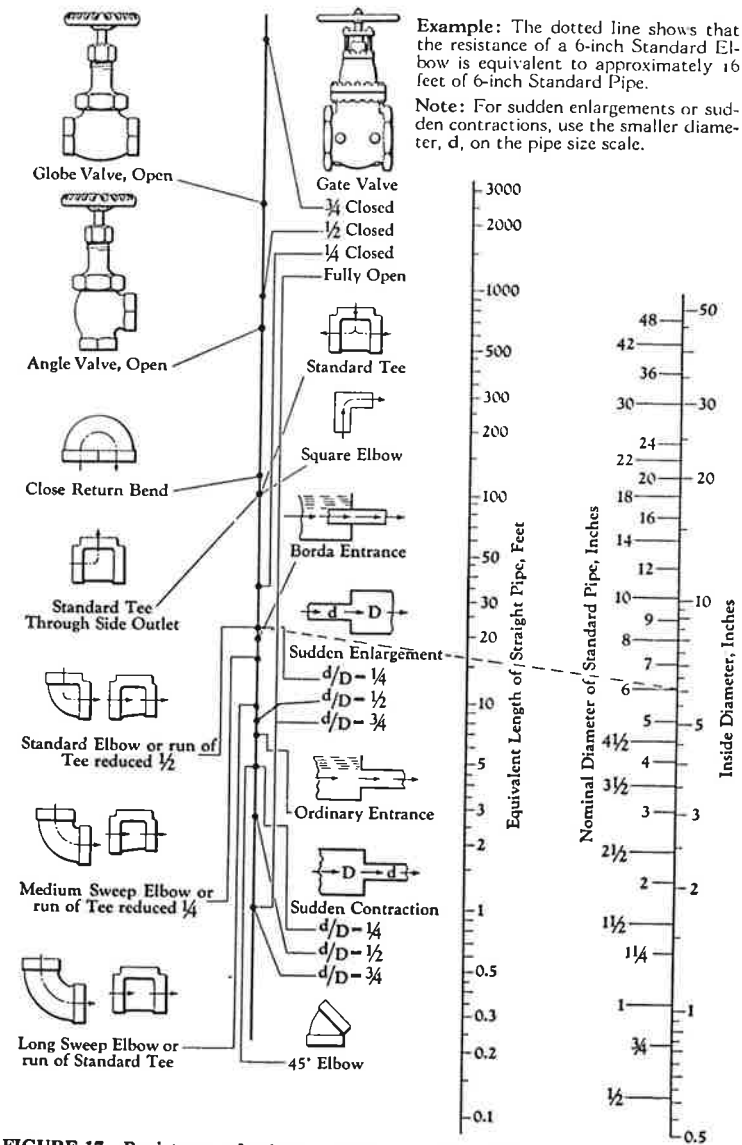



**FIGURE 17** Resistance of valves and fittings to flow of fluid.

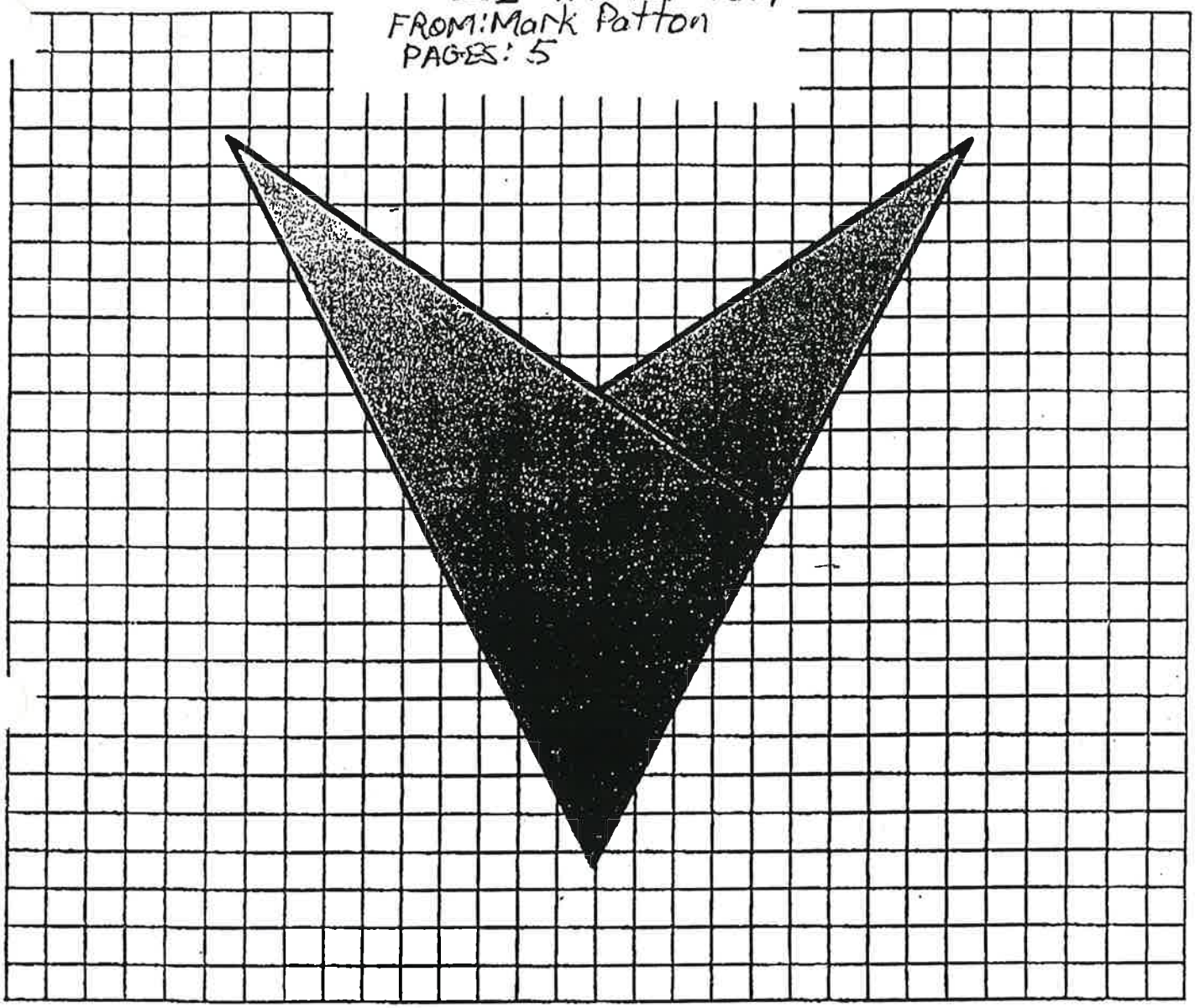
## ORIFICES

**25. High Head.** When the head is relatively large, as compared with the size of the orifice, the following equation will apply:

$$Q = CA\sqrt{2gH} \tag{37}$$

REF 7

TO: Stan Christie
ESI 714-727-7399
FROM: Mark Patton
PAGES: 5



# PROTEC®
# RECIP® PUMP

GROUNDWATER & LEACHATE RECOVERY PUMPS

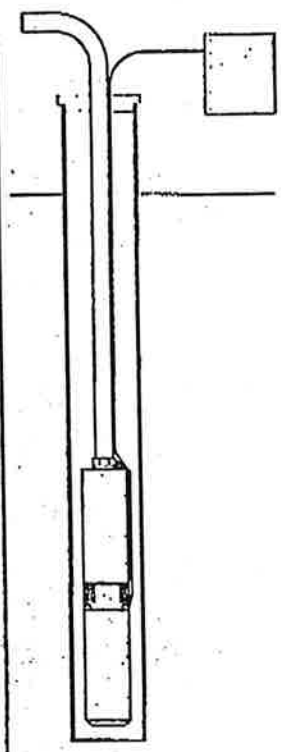
Stan Christie
714-727-7399
For your file
Mark Patton

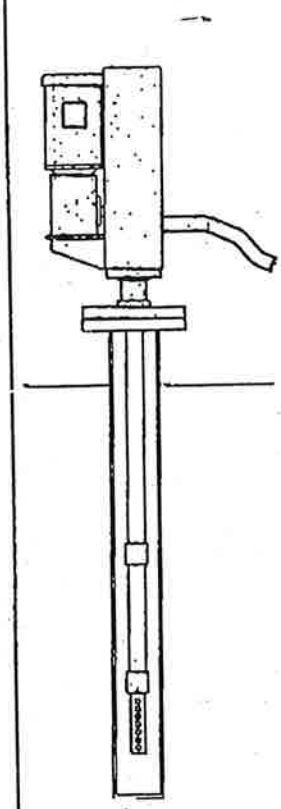
# PROTEC

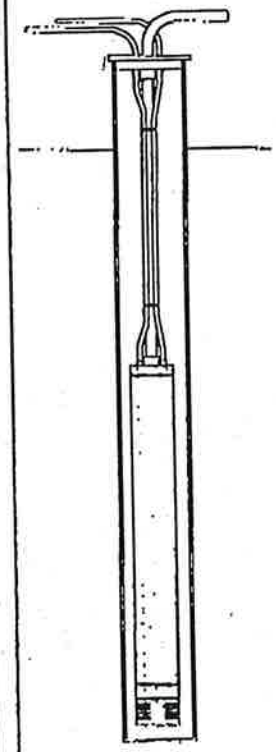
## ENVIRONMENTAL EQUIPMENT

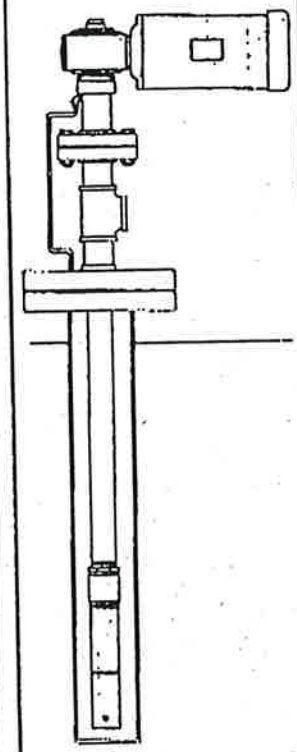
### A COMPLETE LINE OF GROUNDWATER REMEDIATION LANDFILL LEACHATE & SAMPLING PUMPS

| GUARDIAN | RECIP | AUTOPILOT | ROTARY |
|---|---|---|---|
| Variable Speed Electric Submersible Pump | Electric or Air Driven Piston Pump | Automatic "Controllerless" Air Pump | Electric Drive Progressing Cavity Pump |







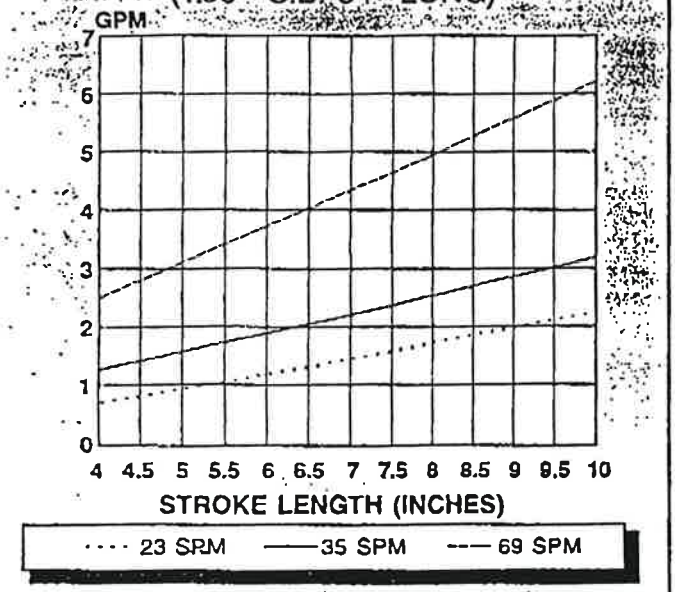


PROTEC

HOUSTON, TX SALES: 918-493-6101 TULSA, OK

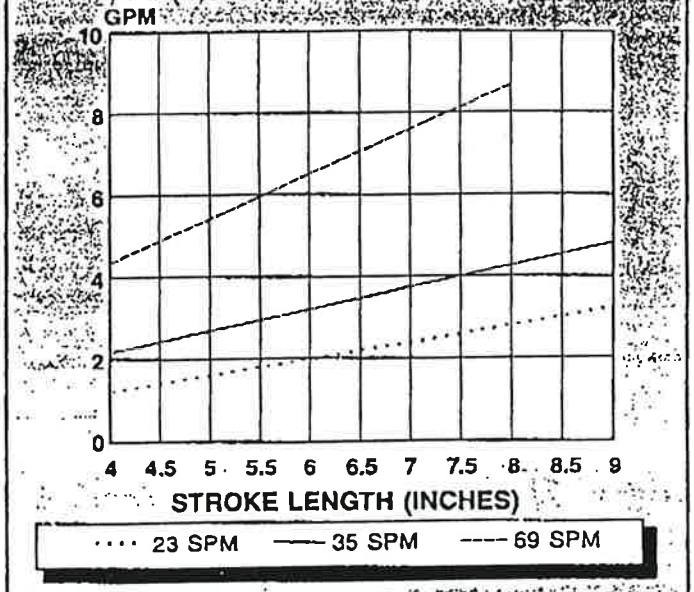
# PROTEC RECIP PUMP CURVES

## MODEL RP1
(1.90" O.D. 34" LONG)



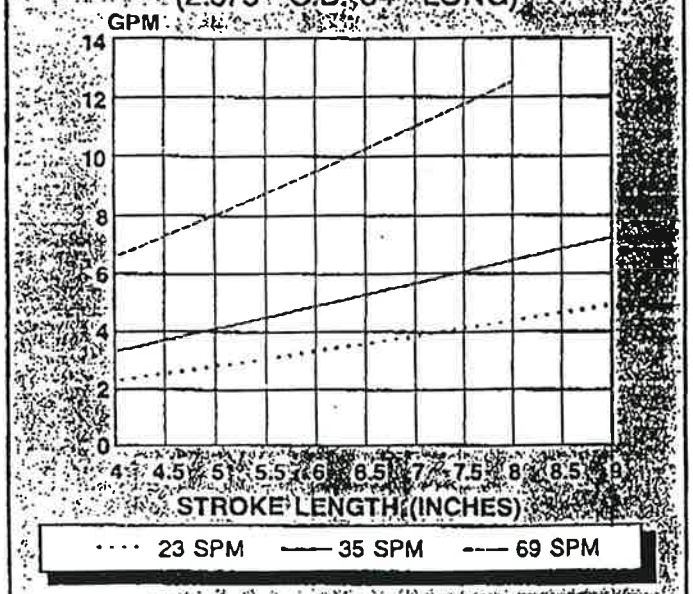

MAXIMUM SETTING DEPTH 200 FT.
1140 RPM MTR = 23 SPM, 1750 RPM MTR = 35 SPM
3450 RPM MTR = 69 SPM, APPROX. 1 HP

## MODEL RP2
(2.375" O.D. 34" LONG)



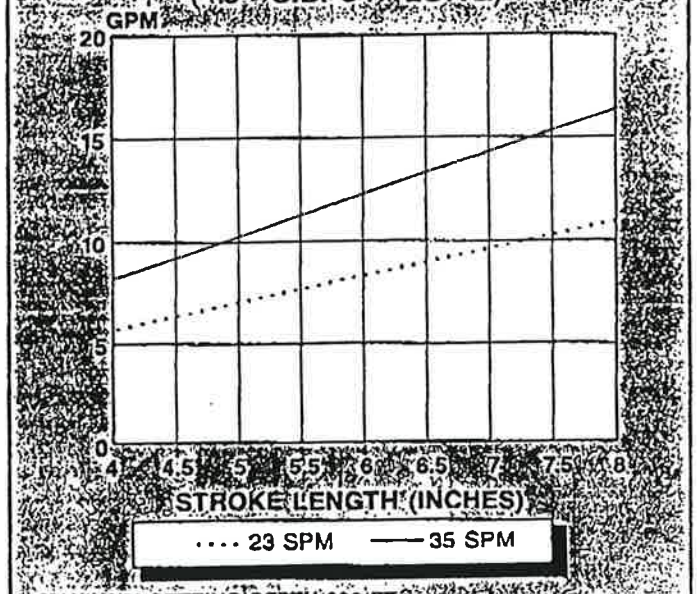

MAXIMUM SETTING DEPTH 200 FT.
1140 RPM MTR = 23 SPM, 1750 RPM MTR = 35 SPM
3450 RPM MTR = 69 SPM, APPROX. 1.5 HP

## MODEL RP3
(2.875" O.D. 34" LONG)




MAXIMUM SETTING DEPTH 200 FT.
1140 RPM MTR = 23 SPM, 1750 RPM MTR = 35 SPM
3450 RPM MTR = 69 SPM, APPROX. 2 HP

## MODEL RP5
(4.5" O.D. 34" LONG)

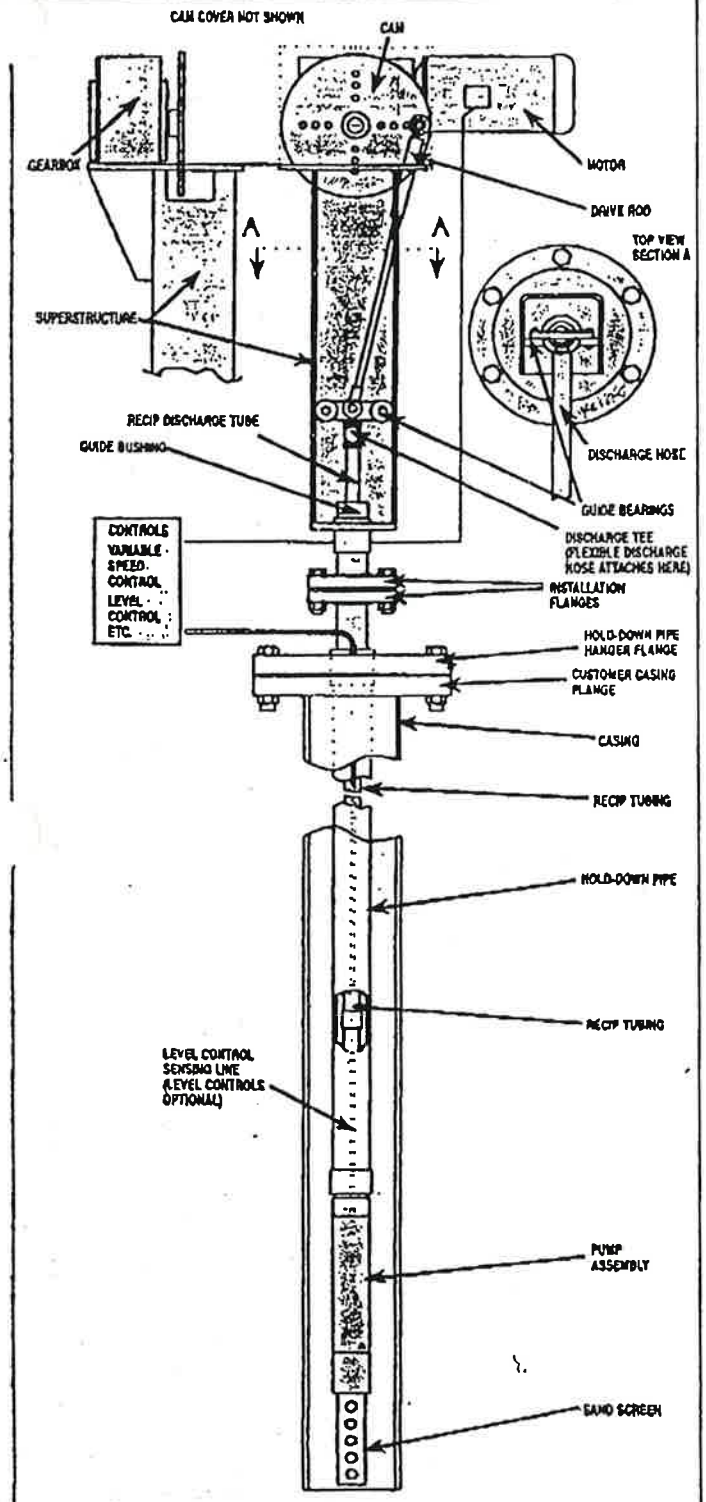



MAXIMUM SETTING DEPTH 200 FT.
1140 RPM MTR = 23 SPM
1750 RPM MTR = 35 SPM, APPROX. 3 HP


**PROTEC**

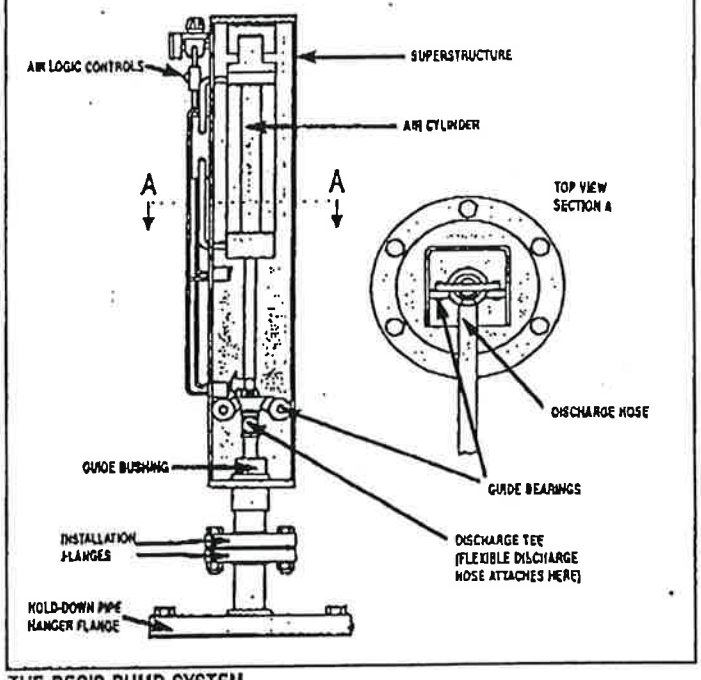
7136 S. Yale, Suite 200 • Tulsa, OK 74136 • 918-493-6101

## ELECTRIC DRIVE




**SURFACE DRIVE ASSY. APPROX. DIMENSIONS**
**ELECTRIC DRIVE: 55" H x 24" W x 12" D**
**AIR DRIVE: 51" H x 8" W x 8" D**
**LOW PROFILE DRIVES ALSO AVAILABLE WHEN THERE ARE HEIGHT LIMITATIONS**

## AIR DRIVE




### THE RECIP PUMP SYSTEM

The **RECIP PUMP** is a surface driven (air drive or electric drive) down-hole reciprocating pump. The down-hole pump assembly is anchored by a string of stationary pipe (Hold-Down Pipe). The pump piston is attached to the bottom of a string of reciprocating tubing (Recip Tubing) which is run inside the Hold-Down Pipe. The surface drive assembly alternately lifts and lowers the Recip Tubing which gives the down-hole pump its reciprocating motion. The surface drive assembly can be either Air Drive or Electric Drive. The Air Drive unit utilizes a dual acting air cylinder. The Electric Drive unit utilizes an electric motor (TEFC or X-Proof) coupled to a gearbox and cam assembly.

The **RECIP PUMP** is a positive displacement piston pump with pumping rate (GPM) directly proportional to strokes per minute (SPM) and stroke length (SL). On the Electric Drive unit SL is adjusted manually by changing the position of the drive rod connection to the cam and SPM depends on the motor RPM selected (1140, 1750 or 3450 RPM). An optional electronic VSD (Variable Speed Drive) is also available to give variable SPM capability to the Electric Drive unit. The Air Drive unit comes standard as a variable speed device since the pump speed (SPM) can be controlled by the pressure regulator on the drive unit. The Recip Pump is currently available in four sizes: RP1, RP2, RP3 and RP5. Pump rates range from fractional GPM to 16 GPM.

Down-hole pump materials of construction are stainless steel and Teflon for service in highly corrosive fluids. **RECIP PUMP** can run dry without causing damage to the pump. Pump operation is easily interfaced with down-hole level controls, tank level controls, safety shut-downs, timers, etc. to meet the operating requirements of any particular site.

### APPLICATIONS

The **RECIP PUMP** is an efficient low-maintenance system for installation in extraction wells and trench sumps associated with groundwater recovery and leachate collection projects. **RECIP PUMP** is also useful as a dedicated sampling pump for groundwater monitoring.

Notice that the Model RP1 can pump up to 6 GPM from wells with 2" casing. This means that often existing monitoring wells can be utilized for continuous recovery when a project advances from the sampling stage to the remedial pump and treat stage. And when new recovery wells must be drilled it is less expensive to drill and equip 2" wells than larger diameter wells.

### IN SUMMARY

High volumetric pump efficiency and low power requirements make the **RECIP PUMP** a highly efficient pumping system. Simple design, rugged construction and low operating speeds result in long operating life and low maintenance. Other features are: low noise, small size, quick installation and all moving parts fully enclosed.

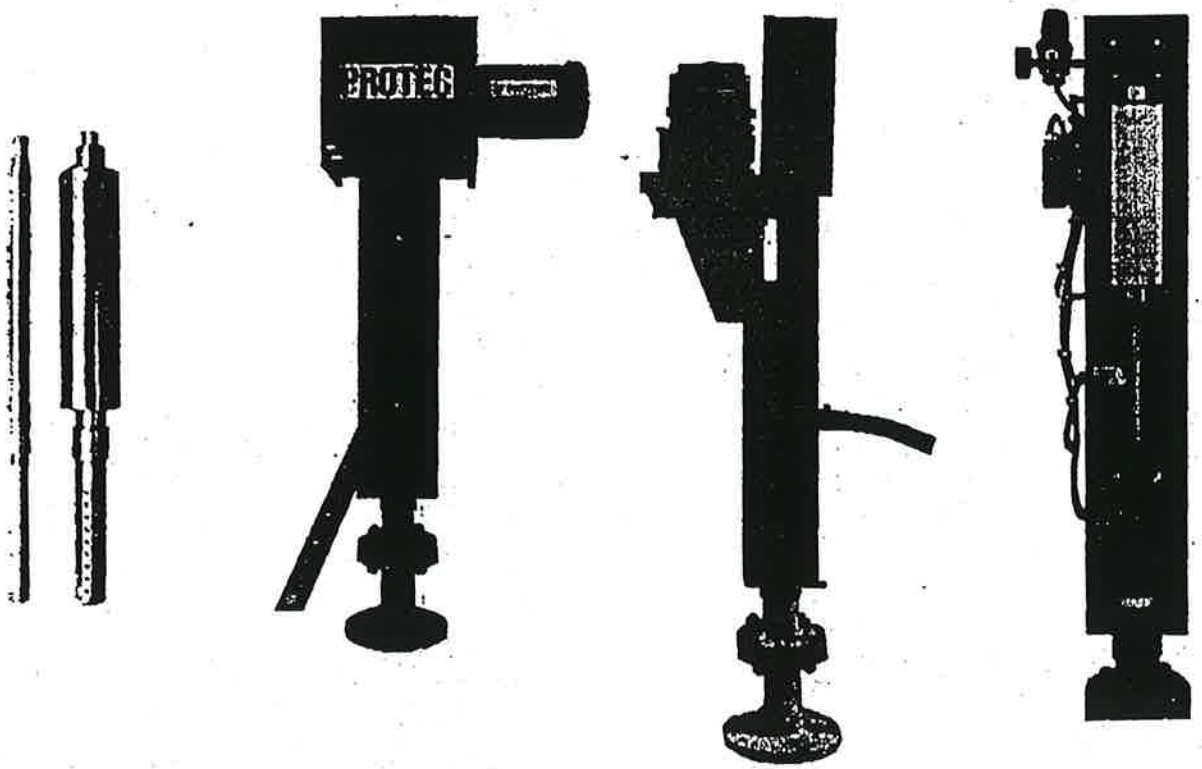
# PUMP SELECTION TABLE

| PUMP MODEL | RP1 | RP2 | RP3 | RP5 |
|---|---|---|---|---|
| PUMP OUTSIDE DIAMETER | 1.90 IN. | 2.375 IN. | 2.875 IN. | 4.50 IN. |
| STANDARD MOTOR HP[1] | 1.0 HP | 1.5 HP | 2.0 HP | 3.0 HP |
| PUMP SETTING DEPTH | 1 - 200 FT. | 1 - 200 FT. | 1 - 200 FT. | 1 - 200 FT. |
| GPM RANGE | | | | |
| 1. FIXED SPEED ELECTRIC ADJUSTABLE SL[2] | | | | |
| DEPENDS ON MOTOR RPM: | | | | |
| WITH 1140 RPM MOTOR[3] | 0.8-2.1 GPM | 1.4-3.2 GPM | 2.2-4.9 GPM | 5-10 GPM |
| WITH 1750 RPM MOTOR | 1.2-3.1 GPM | 2.1-4.8 GPM | 3.2-7.3 GPM | 8-16 GPM |
| WITH 3450 RPM MOTOR | 2.5-6.2 GPM | 4.2-8.5 GPM | 6.5-13.0 GPM | N/A |
| 2. VARIABLE SPEED ELECTRIC ADJUSTABLE SL & SPM[4] | | | | |
| USE ELECTRIC VSD[5] | 0.1-6.2 GPM | 1.4-8.5 GPM | 2.2-13.0 GPM | 5-16 GPM |
| 3. VARIABLE SPEED AIR ADJUSTABLE SPM | 0.1-6.2 GPM | 1.4-8.5 GPM | 2.2-13.0 GPM | 5-16 GPM |

[1] Motor HP can vary depending on operating conditions.
[2] SL: Strokes Length.
[3] 1140 RPM motors available in 3-Phase only.
[4] SPM: Strokes Per Minute.
[5] VSD: Variable Speed Drive.

NOTE: Above data is in accordance with the pump curves on the back cover.

**PUMPS** **ELECTRIC DRIVE** **AIR DRIVE**



**PROTEC RECIP PUMP**
**PRICE LIST**
(FOR ESTIMATING PURPOSES)
November 1994

The following are estimated prices for complete pumping units including down-hole pump, surface drive assembly and 4 feet of cross-linked polyethylene discharge hose. On Electric Drive units for variable SPM (Strokes Per Minute) add VSD (Variable Speed Drive). Air Drive price includes pneumatic controls and Air Drive units are inherently variable SPM.

| PUMP MODEL | AIR DRIVE 316 S.S | FIXED SPM TEFC MOTOR | FIXED SPM X-PROOF MOTOR | VSD ADD-ON NEMA 3R PANEL FOR USE WITH TEFC DC MOTOR | VSD ADD-ON NEMA 3R PANEL FOR USE WITH TEFC OR X-PROOF AC MOTOR |
|---|---|---|---|---|---|
| RP1 | 2,700.00 | 2,975.00 | 3,175.00 | 875.00 | 1,075.00 |
| RP2 | 2,900.00 3100 | 3,175.00 | 3,425.00 | 1,025.00 | 1,175.00 |
| RP3 | 3,100.00 | 3,375.00 | 3,625.00 | N/A | 1,275.00 |
| RP5 | 3,700.00 | 3,975.00 | 4,225.00 | N/A | 1,325.00 |

**ACCESSORIES**

**LEVEL CONTROL:**
Dual Probe Level Control.....286.00
Dual Float Level Control.....305.00
Level Control Signal Wire.....0.50/FT
(22 Gauge, Teflon Jacketed)
Pneumatic Level Control.....275.00
Pneumatic L/C Tube (3/8" Teflon).....2.50/FT
Stroke Counter.....100.00
Pneumatic Dual Timer.....275.00

**WELL TUBULARS**

1. FOR USE IN 2" CASING ( RP1 ONLY ):
RECIP TUBING : 1/2" CS T&C.....1.23/FT
1/2" 304SS T&C.....5.22/FT*
HOLD-DOWN PIPE: 1" CS T&C.....1.81/FT
1" 304 SS T&C.....8.36/FT*

2. FOR USE IN 3" AND LARGER CASING ( RP5 6" AND LARGER CASING ):
RECIP TUBING ( RP1, 2, 3 ) : 3/4" CS T&C.....1.55/FT
3/4" 304 SS T&C.....6.62/FT* ← Add 25 % for 316 S.S.
( RP5 ONLY ): 1" CS T&C.....1.81/FT
1" 304 SS T&C.....8.36/FT* ←
HOLD-DOWN PIPE ( RP1, 2, 3 ): 1 1/4" CS T&C.....2.43/FT
1 1/4" 304 SS T&C.....10.53/FT* ←
( RP5 ONLY ): 2" CS T&C.....2.96/FT
2" 304 SS T&C.....13.86/FT* ←

*SS PRICES CAN FLUCTUATE SIGNIFICANTLY OVER TIME.

**WELL HEAD FITTINGS:** Standard Carbon Steel.....150.00
Stainless Steel.....price on request

**TOP LOADING INLET JACKET (PUMP CAN):** Carbon steel .....150.00

**CONTROL PANELS** - NEMA 3R Enclosure ( Contactor, Adjustable Thermal Overload, HOA. Does not include primary disconnect or short circuit protection.)

FIXED SPM: THRU 10 HP, 1-PHASE OR 3-PHASE.....250.00
FOR VARIABLE SPM: PANEL PRICE INCLUDED IN VSD PRICE ABOVE. CONTROL PANELS CAN ALSO BE CUSTOM BUILT PER CUSTOMER REQUIREMENTS.

**HOUSTON, TX** **SALES: 918-493-6101** **TULSA, OK**

REF 8

CRC

# STANDARD MATHEMATICAL TABLES

28th EDITION

CRC PRESS

REF 8A



*Mensuration Formulas*

**Pyramid**

$S$ of regular pyramid $= \frac{1}{2}$(perimeter of base) $\times$ (slant height)

$V = \frac{1}{3}$(area of base) $\times$ (altitude)

**Frustum of Pyramid**

Let $B_1$ = area of lower base, $B_2$ = area of upper base, $h$ = altitude.

$S$ of regular figure $= \frac{1}{2}$(sum of perimeters of bases) $\times$ (slant height)

$V = \frac{1}{3}h(B_1 + B_2 + \sqrt{B_1B_2})$

**Prismatoid**

A *prismatoid* is a polyhedron having for bases two polygons in parallel planes, and for lateral faces triangles or trapezoids with one side lying in one base, and the opposite vertex or side lying in the other base, of the polyhedron. Let $B_1$ = area of lower base, $M$ = area of midsection, $B_2$ = area of upper base, $h$ = altitude.

$$V = \frac{1}{6}h(B_1 + 4M + B_2) \quad \text{(the } \textit{prismoidal formula}\text{)}$$

*Note:* Since cubes, rectangular parallelepipeds, prisms, pyramids, and frustums of pyramids are all examples of prismatoids, the formula for the volume of a prismatoid subsumes most of the above volume formulae.

**Regular Polyhedra**

Let $v$ = number of vertices, $e$ = number of edges, $f$ = number of faces, $\alpha$ = each dihedral angle, $a$ = length of each edge, $r$ = radius of the inscribed sphere, $R$ = radius of the circumscribed sphere, $A$ = area of each face, $T$ = total area, $V$ = volume.

$v - e + f = 2$ (the *Euler-Descartes formula*—actually holds for *any* convex polyhedron)

$$T = fA$$
$$V = \frac{1}{3}rfA = \frac{1}{3}rT$$

| Name | Nature of Surface | $T$ | $V$ |
|---|---|---|---|
| Tetrahedron | 4 equilateral triangles | $1.73205a^2$ | $0.11785a^3$ |
| Hexahedron (cube) | 6 squares | $6.00000a^2$ | $1.00000a^3$ |
| Octahedron | 8 equilateral triangles | $3.46410a^2$ | $0.47140a^3$ |
| Dodecahedron | 12 regular pentagons | $20.64573a^2$ | $7.66312a^3$ |
| Icosahedron | 20 equilateral triangles | $8.66025a^2$ | $2.18169a^3$ |

| Name | $v$ | $e$ | $f$ | $\alpha$ | $a$ | $r$ |
|---|---|---|---|---|---|---|
| Tetrahedron | 4 | 6 | 4 | 70° 32′ | $1.633R$ | $0.333R$ |
| Hexahedron | 8 | 12 | 6 | 90° | $1.155R$ | $0.577R$ |
| Octahedron | 6 | 12 | 8 | 109° 28′ | $1.414R$ | $0.577R$ |
| Dodecahedron | 20 | 30 | 12 | 116° 34′ | $0.714R$ | $0.795R$ |
| Icosahedron | 12 | 30 | 20 | 138° 11′ | $1.051R$ | $0.795R$ |

# C.9 LDS FLOW CAPACITY AND PUMP SIZING

# Environmental Solutions, Inc.

By: RVH
Date: 1/19/96

Subject: Beatty Landfill-Cell 12
American Ecology Corporation

Checked By: EC
Checked On: 3/8/96

## LDS Flow Capacity (Action Leakage Rate) and Pump Sizing

### Purpose:

1. Determination of the action leakage rate (ALR) for each of the three sumps proposed for Cell 12 in the Beatty Landfill.

### Method:

1. Identify applicable regulations and standards.
2. Generate conceptual sump geometry.
3. Determine flow capacities of the selected drainage elements
4. Refine sump geometry as necessary, and identify controlling flow section.
5. Calculate ALR for Cell 12 sumps.

### Summary

1. Double layers of geocomposite are required for each sump at the grade break to provide sufficient flow capacity.
2. Sumps in cell corners require geonet rather than geocomposite to provide adequate flow capacity due to reduction of effective perimeter.
3. ALR's for sumps are as follows:

| Sump | Total Area | ALR |
|---|---|---|
| 12A | 3.49 ac | 200 gpad |
| 12B | 2.45 ac | 185 gpad |
| 12C | 2.87 ac | 212 gpad |

4. Variable speed recipricating pumps, identical to those used in the LCRS are recommended for use.

### Analysis:

#### Applicable Regulations

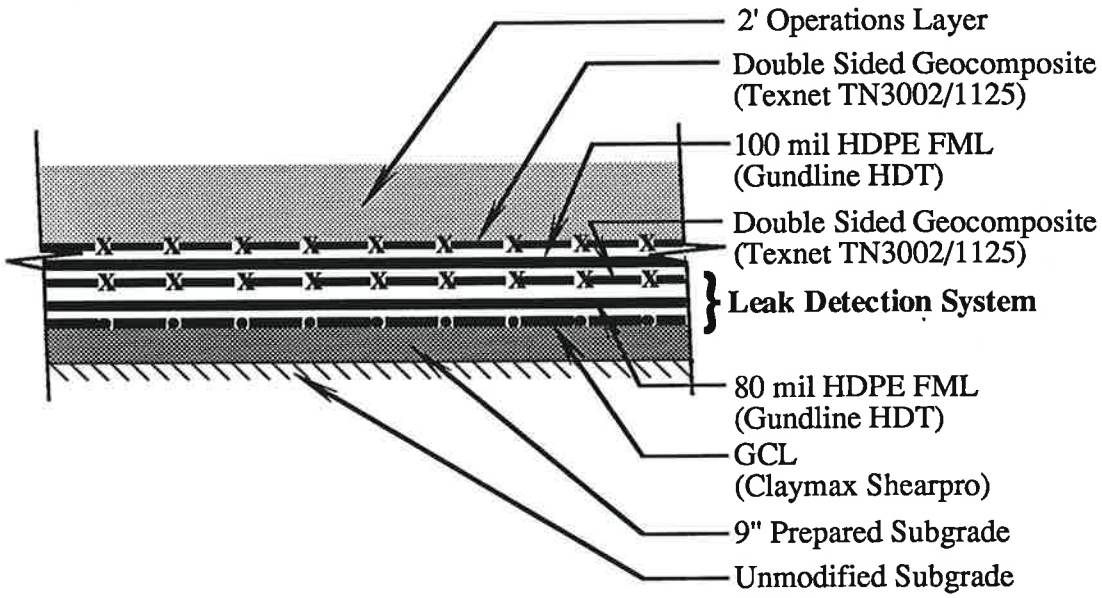
1. By regulatory definition (Reference 1A) the ALR is the maximum design flow rate the leak detection system (LDS) can convey without the fluid head on the secondary, composite, liner exceeding one foot.

#### Typical Geometry

1. The typical bottom liner is shown below in Figure 1:

## LDS Flow Capacity (Action Leakage Rate) and Pump Sizing

**Figure 1**




2. The sump locations are shown on Figure 2 (Attached). There are three sumps proposed for Cell 12. The typical geometry is shown in Figure 3 (Attached).

Flow Capacities

1. The following flow elements will be used in the LDS

   - Double sided geocomposite (Texnet TN3002/1125)
   - Geonet (Polynet PN3000)-used in the sump areas if additional flow capacity is required.
   - Gravel (clean, poorly graded, nominal 3/4" diameter)-used in the sump proper only.

2. The double sided geocomposite is used in the slope liner LDS as well as in the bottom liner as shown in Figure 1. A continuous strip of geocomposite will be used for each system. Therefore flow within the LDS will be controlled by the minimum bottom slope.

3. Flow within the geosynthetics is calculated using Darcy's Equation (which assumes laminar flow within the net) as follows (Reference 2A):

# Environmental Solutions, Inc.

By: RVH
Date: 1/19/96

Subject: Beatty Landfill-Cell 12
American Ecology Corporation

Checked By: EC
Checked On: 3/6/96

## LDS Flow Capacity (Action Leakage Rate) and Pump Sizing

$$q = \Theta_{eff} i \qquad (1)$$

Where:

$q$ = flow per unit width

$\Theta_{eff}$ = effective transmissivity

$i$ = hydraulic gradient

Flow within the gravel also uses Darcy's formula, however, transmissivity is replaced with hydraulic conductivity and the thickness of the flow area.

4. Effective transmissivity for the geosynthetics is calculated by applying several safety factors to the published transmissivity value. The following formula (Reference 2B) is used for that calculation, definitions are provided in the attached reference:

$$\Theta_{eff} = \frac{\Theta}{(FS_{CR} \times FS_{IN} \times FS_{CC} \times FS_{BC})} \qquad (2)$$

5. The following table shows the unit flow capacity for the geosynthetic elements and the gravel based on the applicable transmissivity, or hydraulic conductivities, hydraulic gradients, and safety factors:

**Table 1**

| Flow Element | $\Theta$ | $FS_{CR}$ | $FSI_N$ | $FS_{CC}$ | $FS_{BC}$ | $\Theta_{eff}$[1] | $i$ | q | q |
|---|---|---|---|---|---|---|---|---|---|
| Units: | m2/s | NA | NA | NA | NA | m2/s | NA | m3/m-s | gal/ft-day |
| TN3002/1125 | 2.20E-04 | 1.4 | 1.5 | 1.5 | 1.5 | 4.66E-05 | 0.01 | 4.66E-07 | 3.24 |
| PN3000 | 2.00E-03 | 1.4 | 1.5 | 1.5 | 1.5 | 4.23E-04 | 0.01 | 4.23E-06 | 29.45 |
| TN3002/1125 | 2.20E-04 | 1.4 | 1.5 | 1.5 | 1.5 | 4.66E-05 | 0.1 | 4.66E-06 | 32.39 |
| PN3000 | 2.00E-03 | 1.4 | 1.5 | 1.5 | 1.5 | 4.23E-04 | 0.1 | 4.23E-05 | 294.47 |

Transmissivity values are provided by the manufacturer (Reference 3A and 3B). Safety factors are taken from the literature and are attached (Reference 2C). Flow capacities are shown at hydraulic gradients of 1 percent and 10 percent for the nominal cell bottom slope and the minimum sump slope, respectively. Since the gravel is used only within the minimum sump boundaries around the riser pipes, flow capacities are calculated for the geosynthethics only.

Controlling Section

1. As shown in Figure 3 there are two potentially controlling sections: 1) the 7'x7' LDS perimeter; and 2) the perimeter at the grade break between the 1 and 10 percent slopes. In the controlling sections multiple layers of geocomposite or geonet alone may be used to provide sufficient flow capacity.

---

[1] The effective transmissivity of the geocomposite (using the factors of safety listed in Table 1) still exceeds the minimum transmissivity requirement ($3 \times 10^{-5}$ $m^2/s$) of 40 CFR § 264.301(c)(3)(ii).

# Environmental Solutions, Inc.

By: RVH
Date: 1/19/96

Subject: Beatty Landfill-Cell 12
American Ecology Corporation

Checked By: EC
Checked On: 3/8/96

## LDS Flow Capacity (Action Leakage Rate) and Pump Sizing

2. Using EPA guidelines (Reference 4A) the sumps will be designed for a nominal ALR of 200 gallons per acre per day (gpad). Total flow for each cell is shown in Table 2, below:

**Table 2**

| Sump | Tributary Area (ac) | Total Flow (gpd) |
|---|---|---|
| 12A | 3.49 | 698 |
| 12B | 2.45 | 490 |
| 12C | 2.87 | 574 |

Since the maximum flow to the sump is only about 0.5 gpm (Sump 12A ALR), the Protec recipricating pumps recommended in the LCRS calculation are acceptable units. It is anticipated that liquid will be allowed to accumulate in the LDS until sufficient volume is present to remove efficiently with a pump. At no time however, would more than 12 inches of fluid be allowed to collect over the liner system.

2. Each sump has a nominal perimeter of 70 feet (Figure 3), however sumps 1 and 3 have one short edge near the slope. Therefore, as a conservatism, the effective perimeter for each cell will be reduced by 25 percent to 53 feet. Table 3 shows the minimum length (i.e., perimeter) required to accomdate the total flow for each sump using the various flow elements. Since gravel is not used outside of the sumps, perimeters are calculated for only the geosynthetics.

**Table 3**

| | | Min. Perimeter Length (ft) Required for Flow at i=.01 | | | |
|---|---|---|---|---|---|
| | | No. of Geocomposites | | | |
| Sump | Design Flow (gpd) | 1 | 2 | 3 | Geonet |
| 12A | 698 | 215 | 108 | 72 | 24 |
| 12B | 490 | 151 | 76 | 50 | 17 |
| 12C | 574 | 177 | 89 | 59 | 19 |

3. Given the minimum grade break perimeter of 53 feet triple geocomposite layers are adequate for only sump 12B. Also note that geonet is adequate in all cases. Hence, geonet will be used between the grade break and the perimeter. Double geocomposite provides adequate flow capacity beyond the grade break perimeter.

4. Within the 7'x7' area, the minimum perimeter is 21 feet (Figure 3). As shown in Table 4, below, a single geocomposite layer has sufficient flow capacity to accomodate the design ALR. However geonet will also be used in the 10 percent slope areas, to facilitate construction and prevent capacity losses in the transition between the higher capacity geonet and the double geocomposite.

# Environmental Solutions, Inc.

By: RVH
Date: 1/19/96

Subject: Beatty Landfill-Cell 12
American Ecology Corporation

Checked By: EC
Checked On: 3/8/96

## LDS Flow Capacity (Action Leakage Rate) and Pump Sizing

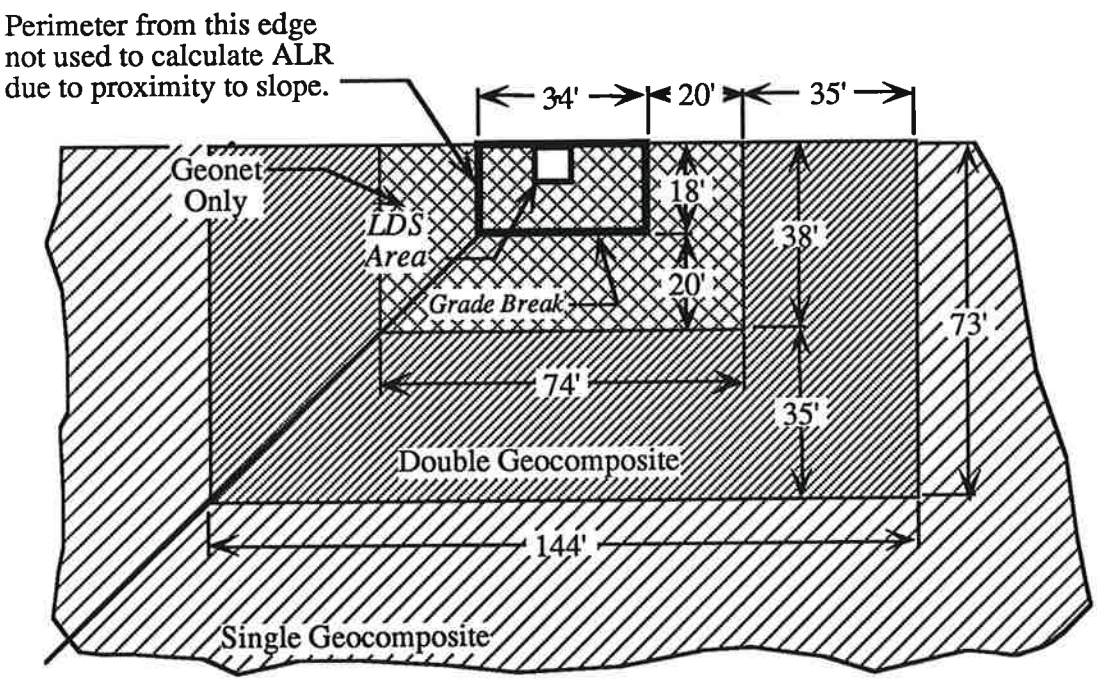
**Table 4**

| Sump | Design Flow (gpd) | Min. Perimeter Length (ft) Required for Flow at i=.10 — No. of Geocomposites 1 | 2 | 3 | Geonet |
|---|---|---|---|---|---|
| 12A | 698 | 22 | 11 | 7 | 2 |
| 12B | 490 | 15 | 8 | 5 | 2 |
| 12C | 574 | 18 | 9 | 6 | 2 |

ALR

1. Using the perimeters identified above, flow elements are arranged for each sump. Figure 4 is the arrangement for Sump 12A.

**Figure 4**




**Sump 12A**

The ALR for Sump 12A is controlled by the minimum perimeter of the single geocomposite layer. The perimeter and resulting ALR are:

# Environmental Solutions, Inc.

By: RVH
Date: 1/19/96

Subject: Beatty Landfill-Cell 12
American Ecology Corporation

Checked By: EC
Checked On: 2/8/96

## LDS Flow Capacity (Action Leakage Rate) and Pump Sizing

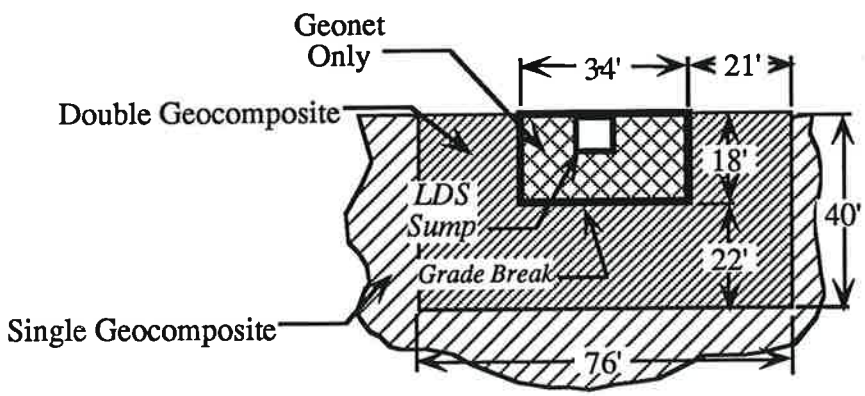
$$Perim = 144\,ft + 73\,ft$$
$$= 217\,ft$$

$$Capacity = 217\,ft\left(3.24\frac{\text{gal}}{\text{ft - day}}\right)$$
$$= 703\text{gpd}$$

$$\text{ALR} = \frac{703\text{gpd}}{3.49\text{ac}} = 201\text{gpad} \approx 200\text{gpad}$$

2. None of theSump 12B flow edges are located next to a slope. Therefore the entire perimeter is available to collect leakage. As shown in Table 3 the double geocomposite provides nearly enough capacity for the design ALR of 200 gpad at thegrade break. On the 1 percent side of the grade break double and single geocomposite will be used as shown in Figure 5. On the 10 percent side of thegrade break geonet will be used as for the other sumps.

**Figure 5**




**Sump 12B**

The ALR for Sump 12B is controlled by the minimum perimeter of the double geocomposite layer. The perimeter and resulting ALR are:

$$Perim = 34\,ft + 18\,ft + 18\,ft$$
$$= 70\,ft$$

$$Capacity = 70\,ft \times 2\left(3.24\frac{\text{gal}}{\text{ft - day}}\right)$$
$$= 453\text{gpd}$$

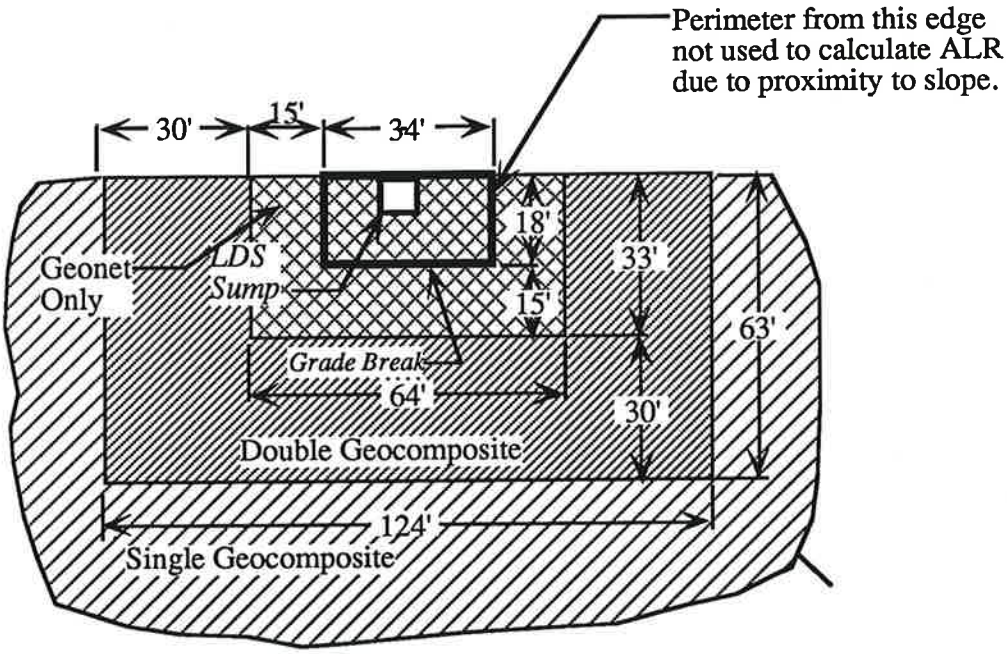
$$\text{ALR} = \frac{453\text{gpd}}{2.45\text{ac}} = 185\text{gpad}$$

## LDS Flow Capacity (Action Leakage Rate) and Pump Sizing

Although slightly less than the design ALR, it is within 5 percent and therefore adequate.

3. Sump 12C is situated similar to Sump 12A. Flow elements are arranged as shown in Figure 6.

**Figure 6**




**Sump 12C**

The ALR for Sump 12C is controlled by the minimum perimeter of the single geocomposite layer. The perimeter and resulting ALR are:

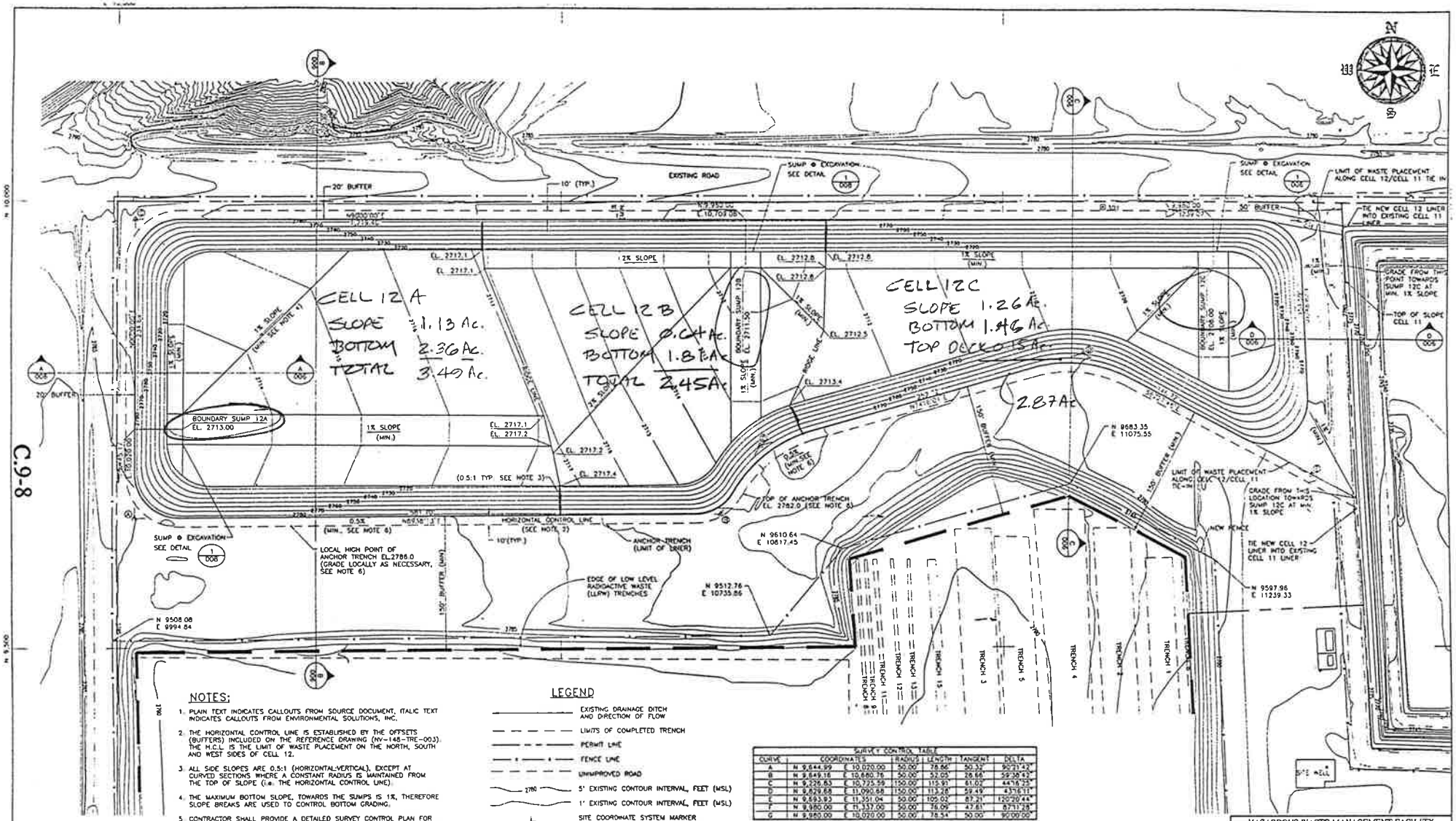
$$Perim = 124\,ft + 64\,ft$$
$$= 188\,ft$$
$$Capacity = 188\,ft\left(3.24\frac{\text{gal}}{\text{ft - day}}\right)$$
$$= 609\,\text{gpd}$$
$$\text{ALR} = \frac{609\,\text{gpd}}{2.87\,\text{ac}} = 212\,\text{gpad}$$

4. Double layers of geocomposite will consist of a double-sided geocomposite overlain by a single-sided geocomposite with the geotextile up. At the transition between the single layer to the double layer, a one foot width of the upper geotextile of the double-sided geocomposite will be removed so that the geonet elements of each are in contact.

~35'

2 MIDHEIGHT PERIMETER (MHP)

35' x MHP = SLOPE AREA




## NOTES:

1. PLAIN TEXT INDICATES CALLOUTS FROM SOURCE DOCUMENT, ITALIC TEXT INDICATES CALLOUTS FROM ENVIRONMENTAL SOLUTIONS, INC.
2. THE HORIZONTAL CONTROL LINE IS ESTABLISHED BY THE OFFSETS (BUFFERS) INCLUDED ON THE REFERENCE DRAWING (NV-148-TRE-003). THE H.C.L. IS THE LIMIT OF WASTE PLACEMENT ON THE NORTH, SOUTH AND WEST SIDES OF CELL 12.
3. ALL SIDE SLOPES ARE 0.5:1 (HORIZONTAL-VERTICAL), EXCEPT AT CURVED SECTIONS WHERE A CONSTANT RADIUS IS MAINTAINED FROM THE TOP OF SLOPE (i.e. THE HORIZONTAL CONTROL LINE).
4. THE MAXIMUM BOTTOM SLOPE, TOWARDS THE SUMPS IS 1%, THEREFORE SLOPE BREAKS ARE USED TO CONTROL BOTTOM GRADING.
5. CONTRACTOR SHALL PROVIDE A DETAILED SURVEY CONTROL PLAN FOR APPROVAL, BASED ON THE HORIZONTAL CONTROL PRIOR TO COMMENCING WORK.
6. PROVIDE LOCAL GRADING AS NECESSARY TO ESTABLISH MINIMUM ANCHOR TRENCH FLOWLINES, WHICH ARE REQUIRED FOR THE COVER SYSTEM DRAIN PIPE (SEE SHEET 4)

## LEGEND

- EXISTING DRAINAGE DITCH AND DIRECTION OF FLOW
- LIMITS OF COMPLETED TRENCH
- PERMIT LINE
- FENCE LINE
- UNIMPROVED ROAD
- 2780 5' EXISTING CONTOUR INTERVAL, FEET (MSL)
- 1' EXISTING CONTOUR INTERVAL, FEET (MSL)
- SITE COORDINATE SYSTEM MARKER LOCATION IN NORTHING AND EASTING
- + 302 MONITOR WELL LOCATION AND NUMBER

SURVEY CONTROL TABLE

| CURVE | COORDINATES | | RADIUS | LENGTH | TANGENT | DELTA |
|---|---|---|---|---|---|---|
| A | N 9,644.99 | E 10,070.00 | 50.00' | 78.66' | 50.32' | 90°21'42" |
| B | N 9,649.16 | E 10,660.76 | 50.00' | 52.03' | 28.66' | 59°36'42" |
| C | N 9,726.63 | E 10,725.59 | 150.00' | 115.90' | 61.02' | 44°16'25" |
| D | N 9,829.68 | E 11,090.88 | 150.00' | 113.28' | 59.49' | 43°16'11" |
| E | N 9,893.93 | E 11,351.04 | 50.00' | 105.02' | 87.21' | 120°29'44" |
| F | N 9,980.00 | E 11,337.00 | 50.00' | 76.09' | 47.61' | 87°17'28" |
| G | N 9,980.00 | E 10,070.00 | 50.00' | 78.54' | 50.00' | 90°00'00" |

SCALE 50 25 0 50 FEET

DATE OF AERIAL TOPOGRAPHY: JUNE 1994

| Rev. | Drawn | Check | Appr | Date | Description |
|---|---|---|---|---|---|
| 3 | DKK | | | 5/95 | REVISION OF CELL 12 ABOVE GRADE |
| 2 | DKK | JGS | SDD | 10/94 | ADDED COORDINATES TO LLRW FENCELINE |
| 1 | DKK | JGS | SDD | 7/94 | SITE TOPO/LLRW CAP REVISED FOR SURVEY 6/94 |

REVISIONS

American Ecology
5333 WESTHEIMER
Suite 1000
Houston, Texas 77056

Reference Drawings
NV-148-TRE-003

HAZARDOUS WASTE MANAGEMENT FACILITY
U.S. ECOLOGY
BEATTY NEVADA

EXCAVATION PLAN AND LIMITS OF GRADING CELL 12

ENVIRONMENTAL SOLUTIONS, INC.

JOB NO. 95284 | Scale AS NOTED | D-95284-BL003 | SHEET 3

Drawn by BOB JONES

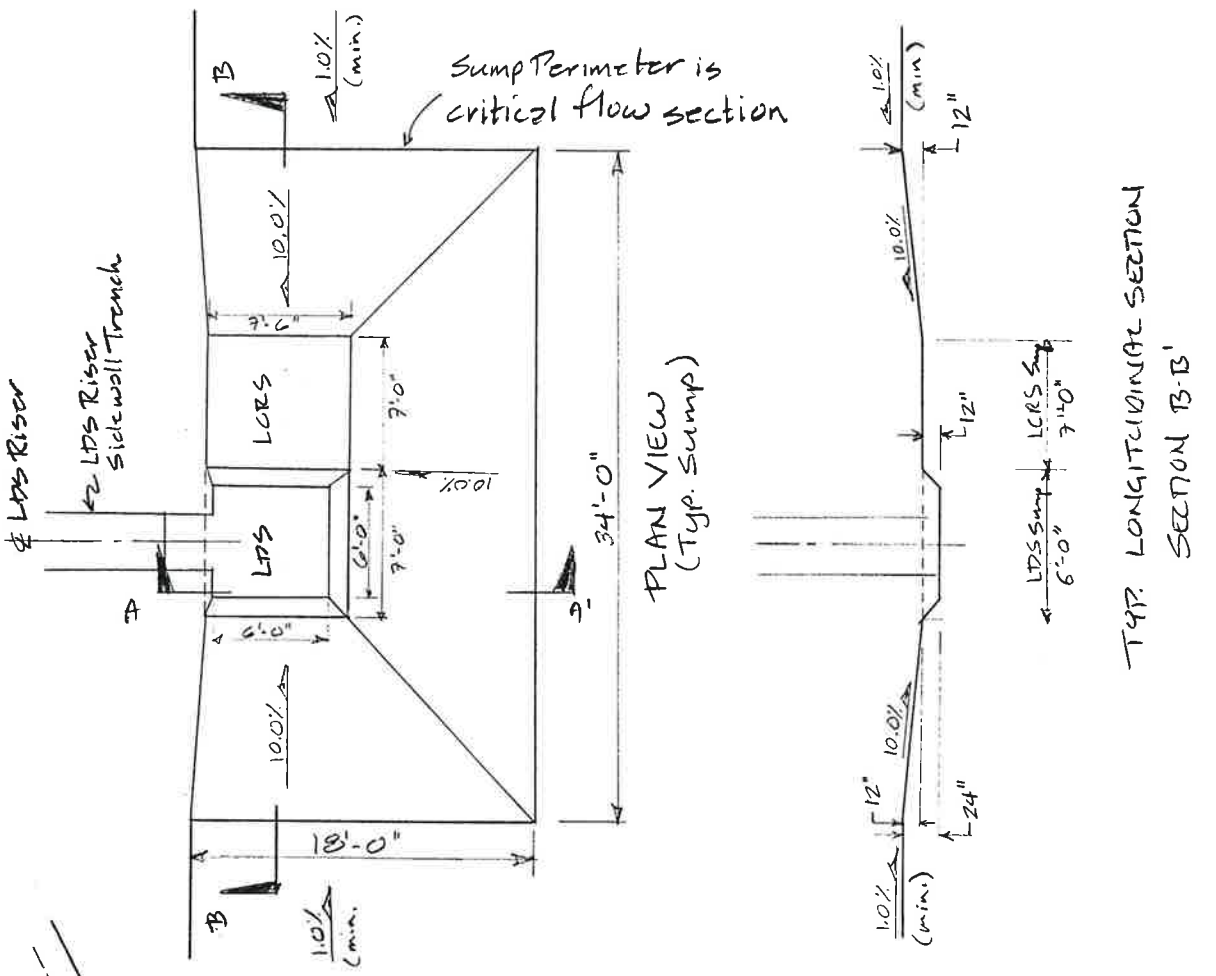
35.284
Q-1 of 20

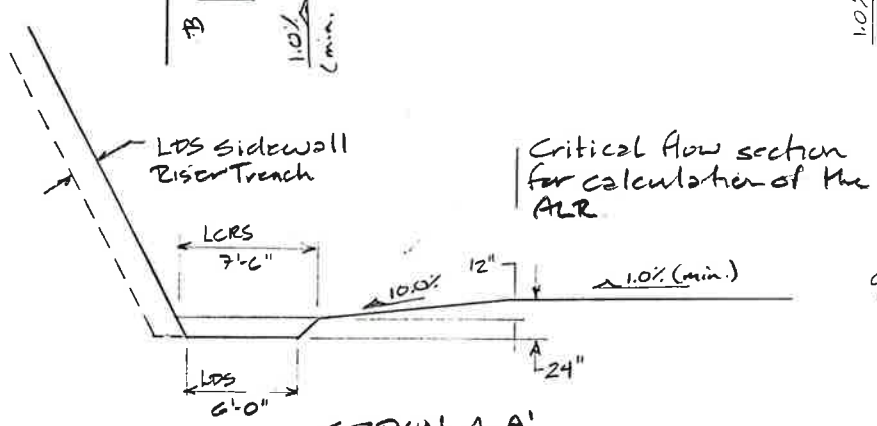
Figure 2
Proposed Sump Location

# ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/20/96 Subject BEATTY LANDFILL-CELL 12 Sheet No. 9 of 20

Chkd. By LMG Date 1/23/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284

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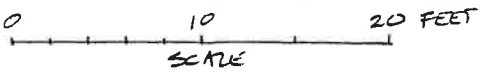


FIGURE 3

TYPICAL SUMP GEOMETRY

Notes:

1) Liner elements & riser pipes not shown for clarity.



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(j), and (k) as paragraphs (g), (h), (i), (j), (k), and (l), respectively, by revising paragraphs (c) and (d), and by adding new paragraph (f) to read as follows:

**§ 264.301 Design and operating requirements.**

LANDFILLS

* * * * *

(c) The owner or operator of each new landfill unit on which construction commences after January 29, 1992, each lateral expansion of a landfill unit on which construction commences after July 29, 1992, and each replacement of an existing landfill unit that is to commence reuse after July 29, 1992 must install two or more liners and a leachate collection and removal system above and between such liners. "Construction commences" is as defined in § 260.10 of this chapter under "existing facility".

(1)(i) The *liner system* must include:

(A) A top liner designed and constructed of materials (e.g., a geomembrane) to prevent the migration of hazardous constituents into such liner during the active life and post-closure care period; and

(B) A composite bottom liner, consisting of at least two components. The upper component must be designed and constructed of materials (e.g., a geomembrane) to prevent the migration of hazardous constituents into this component during the active life and post-closure care period. The lower component must be designed and constructed of materials to minimize the migration of hazardous constituents if a breach in the upper component were to occur. The lower component must be constructed of at least 3 feet (91 cm) of compacted soil material with a hydraulic conductivity of no more than $1\times10^{-7}$ cm/sec.

(ii) The liners must comply with paragraphs (a)(1) (i), (ii), and (iii) of this section.

(2) The *leachate collection and removal system* immediately above the top liner must be designed, constructed, operated, and maintained to collect and remove leachate from the landfill during the active life and post-closure care period. The Regional Administrator will specify design and operating conditions in the permit to ensure that the leachate depth over the liner does not exceed 30 cm (one foot). The leachate collection and removal system must comply with paragraphs (3)(c) (iii) and (iv) of this section.

(3) The *leachate collection and removal system* between the liners, and immediately above the bottom composite liner in the case of multiple leachate collection and removal systems, is also a *leak detection system*. This leak detection system must be capable of detecting, collecting, and removing leaks of hazardous constituents at the earliest practicable time through all areas of the top liner likely to be exposed to waste or leachate during the active life and post-closure care period. The requirements for a leak detection system in this paragraph are satisfied by installation of a system that is, at a minimum:

(i) Constructed with a bottom slope of one percent or more;

(ii) Constructed of granular drainage materials with a hydraulic conductivity of $1\times10^{-2}$ cm/sec or more and a thickness of 12 inches (30.5 cm) or more; or constructed of synthetic or geonet drainage materials with a transmissivity of $3\times10^{-5}$ $m^2$/sec or more;

(iii) Constructed of materials that are chemically resistant to the waste managed in the landfill and the leachate expected to be generated, and of sufficient strength and thickness to prevent collapse under the pressures exerted by overlying wastes, waste cover materials, and equipment used at the landfill;

(iv) Designed and operated to minimize clogging during the active life and post-closure care period; and

(v) Constructed with sumps and liquid removal methods (e.g., pumps) of sufficient size to collect and remove liquids from the sump and prevent liquids from backing up into the drainage layer. Each unit must have its own sump(s). The design of each sump and removal system must provide a method for measuring and recording the volume of liquids present in the sump and of liquids removed.

(4) The owner or operator shall collect and remove pumpable liquids in the leak detection system sumps to minimize the head on the bottom liner.

(5) The owner or operator of a leak detection system that is not located completely above the seasonal high water table must demonstrate that the operation of the leak detection system will not be adversely affected by the presence of ground water.

(d) The Regional Administrator may approve alternative design or operating practices to those specified in paragraph (c) of this section if the owner or operator demonstrates to the Regional Administrator that such design and operating practices, together with location characteristics:

(1) Will prevent the migration of any hazardous constituent into the ground water or surface water at least as effectively as the liners and leachate collection and removal systems specified in paragraph (c) of this section; and

(2) Will allow detection of leaks of hazardous constituents through the top liner at least as effectively.

* * * * *

(f) The owner or operator of any replacement landfill unit is exempt from paragraph (c) of this section if:

(1) The existing unit was constructed in compliance with the design standards of section 3004(o)(1)(A)(i) and (o)(5) of the Resource Conservation and Recovery Act; and

(2) There is no reason to believe that the liner is not functioning as designed.

* * * * *

13. New § 264.302 is added to read as follows:

**§ 264.302 Action leakage rate.**

(a) The Regional Administrator shall approve an action leakage rate for surface impoundment units subject to § 264.301(c) or (d). The action leakage rate is the maximum design flow rate that the leak detection system (LDS) can remove without the fluid head on the bottom liner exceeding 1 foot. The action leakage rate must include an adequate safety margin to allow for uncertainties in the design (e.g., slope, hydraulic conductivity, thickness of drainage material), construction, operation, and location of the LDS, waste and leachate characteristics, likelihood and amounts of other sources of liquids in the LDS, and proposed response actions (e.g., the action leakage rate must consider decreases in the flow capacity of the system over time resulting from siltation and clogging, rib layover and creep of synthetic components of the system, overburden pressures, etc.).

(b) To determine if the action leakage rate has been exceeded, the owner or operator must convert the weekly or monthly flow rate from the monitoring data obtained under § 264.303(c), to an average daily flow rate (gallons per acre per day) for each sump. Unless the Regional Administrator approves a different calculation, the average daily flow rate for each sump must be calculated weekly during the active life and closure period, and monthly during the post-closure care period when monthly monitoring is required under § 264.303(c).

14. Section 264.303 is amended by adding new paragraph (c) to read as follows:

**§ 264.303 Monitoring and inspection.**

* * * * *

(c)(1) An owner or operator required to have a leak detection system under § 264.301(c) or (d) must record the amount of liquids removed from each leak detection system sump at least

KOERNER

DESIGNING WITH GEOSYNTHETICS

Third Edition

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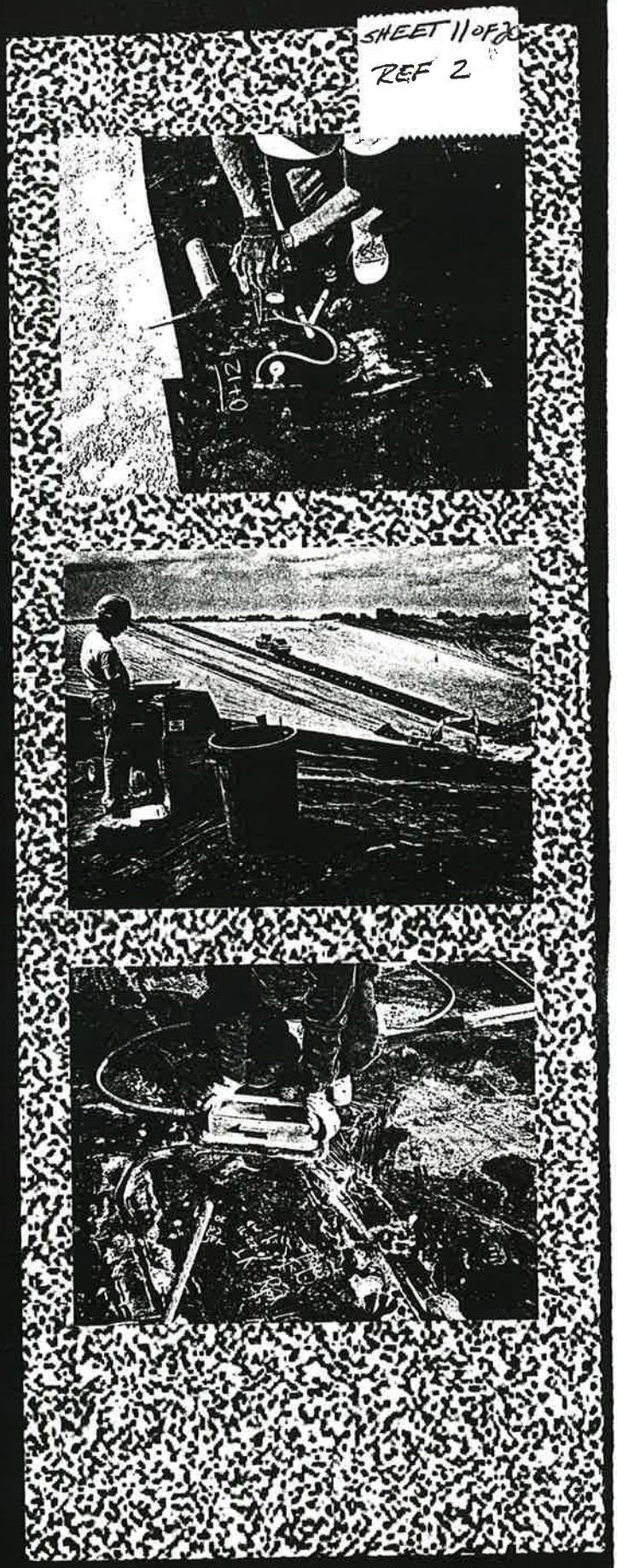
# DESIGNING WITH GEOSYNTHETICS

ROBERT M. KOERNER

SHEET 11 OF 20
REF 2



SHEET 12 OF 20
REF 2A



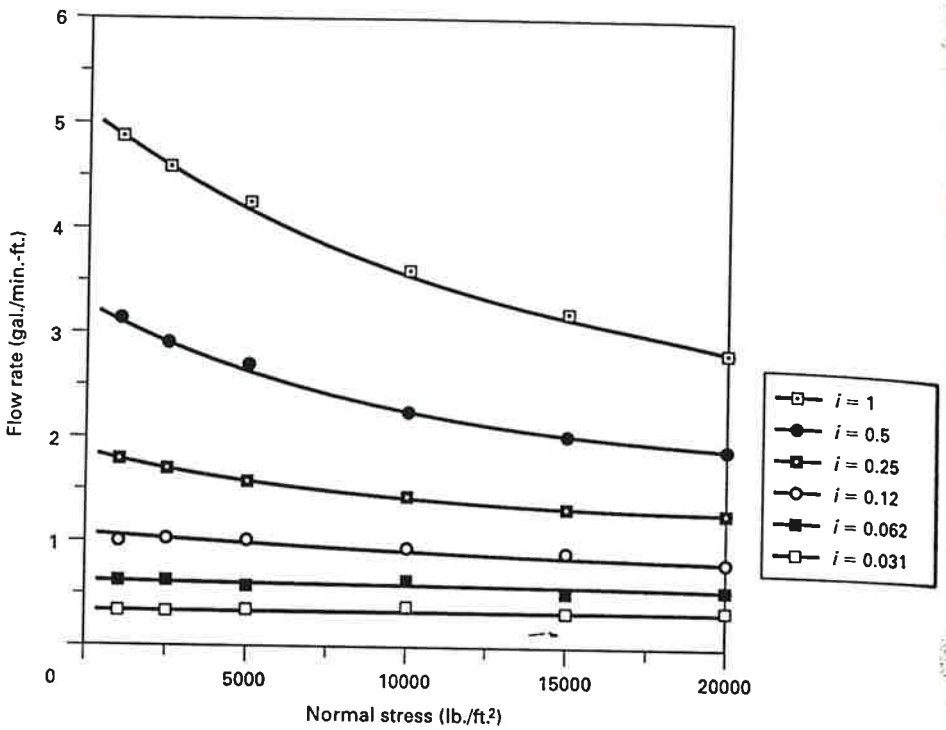



**Figure 4.8** Flow rate behavior of a 0.25-in. (6.3-mm) geonet sandwiched between a 16-oz./yd.² (540-g/m²) nonwoven needle-punched geotextile with clay above and a 60-mil (1.5-mm) HDPE geomembrane below.

met with the typical flow regime in a geonet. Yet current EPA Leak Detection regulations [2] state the following:

- For landfills and waste piles, the geonet's transmissivity must be

$$\theta \geq 3 \times 10^{-5}\ \text{m}^2/\text{sec}.$$

- For surface impoundments, the geonet's transmissivity must be

$$\theta \geq 3 \times 10^{-4}\ \text{m}^2/\text{sec}.$$

One converts from flow rate per unit width to transmissivity as follows:

$$q = kiA \tag{4.1}$$

$$q = ki(t \times W)$$

$$q/W = i(k \times t)$$

$$q/W = i\theta \tag{4.2}$$

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REF 26



generated value is an ultimate value which, using ASTM D4716 for flow rate determination, must be reduced before use in design; that is,

$$q_{\text{allow}} < q_{\text{ult}}$$

One way of doing this is to ascribe partial factors of safety on each of the items not adequately assessed in the laboratory test. For example,

$$q_{\text{allow}} = q_{\text{ult}}\left[\frac{1}{FS_{IN} \times FS_{CR} \times FS_{CC} \times FS_{BC}}\right] \tag{4.5}$$

or if all of the partial factors of safety are lumped together,

$$q_{\text{allow}} = q_{\text{ult}}\left[\frac{1}{\Sigma\, FS_p}\right] \tag{4.6}$$

where $q_{\text{ult}}$ = the flow rate determined from ASTM D4716 for short-term tests between solid plates using water as the transported liquid under laboratory test temperatures,

$q_{\text{allow}}$ = the allowable flow rate to be used in Equation 4.3 for final design purposes,

$FS_{IN}$ = the factor of safety for elastic deformation, or intrusion, of the adjacent geosynthetics into the geonet's core space,

$FS_{CR}$ = the factor of safety for creep deformation of the geonet and/or adjacent geosynthetics into the geonet's core space,

$FS_{CC}$ = the factor of safety for chemical clogging and/or precipitation of chemicals in the geonet's core space,

$FS_{BC}$ = the factor of safety for biological clogging in the geonet's core space, and

$\Sigma\, FS_p$ = the *product* of all partial factors of safety for the site-specific conditions.

Some guidelines for various factors of safety to be used in different situations are given in Table 4.2. Example problems follow, which illustrate the use of geonets and point out that high factors of safety are warranted in critical situations. Please note that these values are based on preliminary and relatively sparse information. Other factors of safety, such as installation damage, temperature effects, and liquid turbidity, could also have been included. If needed they can be included on a site-specific basis. On the other hand, if the test method has included the particular item, it would appear in the foregoing formulation as a value of unity.

**Example:**

What is the allowable geonet flow rate to be used in the design of a capillary break beneath a roadway to prevent frost heave? Assume that laboratory testing was

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REF 2C



**Table 4.2** Recommended preliminary factor of safety values for determining allowable flow rate or transmissivity of geonets

| | *Partial Factor of Safety Value in Equation 4.5* | | | |
|---|---|---|---|---|
| *Application Area* | $FS_{IN}$ | $FS_{CR}$* | $FS_{CC}$ | $FS_{BC}$ |
| Sport fields | 1.0 to 1.2 | 1.0 to 1.5 | 1.0 to 1.2 | 1.1 to 1.3 |
| Capillary breaks | 1.1 to 1.3 | 1.0 to 1.2 | 1.1 to 1.5 | 1.1 to 1.3 |
| Roof and plaza decks | 1.2 to 1.4 | 1.0 to 1.2 | 1.0 to 1.2 | 1.1 to 1.3 |
| Retaining walls, seeping rock and soil slopes | 1.3 to 1.5 | 1.2 to 1.4 | 1.1 to 1.5 | 1.0 to 1.5 |
| Drainage blankets | 1.3 to [illegible] | 1.2 to 1.4 | 1.0 to 1.2 | 1.0 to 1.2 |
| Surface water drains for landfill caps | 1.3 to 1.5 | 1.2 to 1.4 | 1.0 to 1.2 | 1.2 to 1.5 |
| Secondary leachate collection (landfills) | 1.5 to 2.0 | 1.4 to 2.0 | 1.5 to 2.0 | 1.5 to 2.0 |
| Primary leachate collection (landfills) | 1.5 to 2.0 | 1.4 to 2.0 | 1.5 to 2.0 | 1.5 to 2.0 |

*These values assume that the $q_{ult}$ value was obtained using an applied normal pressure of 1.5 to 2 times the field-anticipated maximum value. If not, values must be increased.

done at the proper design load and hydraulic gradient and that this testing yielded a short-term between-rigid-plates value of 1.2 gal./min.-ft.

**Solution:** Since better information is not known, average values from Table 4.2 are used.

$$q_{allow} = q_{ult}\left[\frac{1}{FS_{IN} \times FS_{CR} \times FS_{CC} \times FS_{BC}}\right] \tag{4.5}$$

$$= 1.2\left[\frac{1}{1.1 \times 1.1 \times 1.1 \times 1.2}\right]$$

$$= 1.2\left[\frac{1}{1.60}\right]$$

$$= 0.75 \text{ gal./min.-ft.}$$

**Example:**

What is the allowable geonet flow rate to be used in the design of a secondary leachate collection system? Assume that laboratory testing at proper design load and proper hydraulic gradient gave a short-term between-rigid-plates value of 1.2 gal./min.-ft.

**Solution:** Average values from Table 4.2 are used; however, note the large reduction.

REF 3
SHEET 15 OF 20

# FSI
# FLUID SYSTEMS, INC.

## POLY-NET®

**TRANSMISSIVITY – CHARTS**

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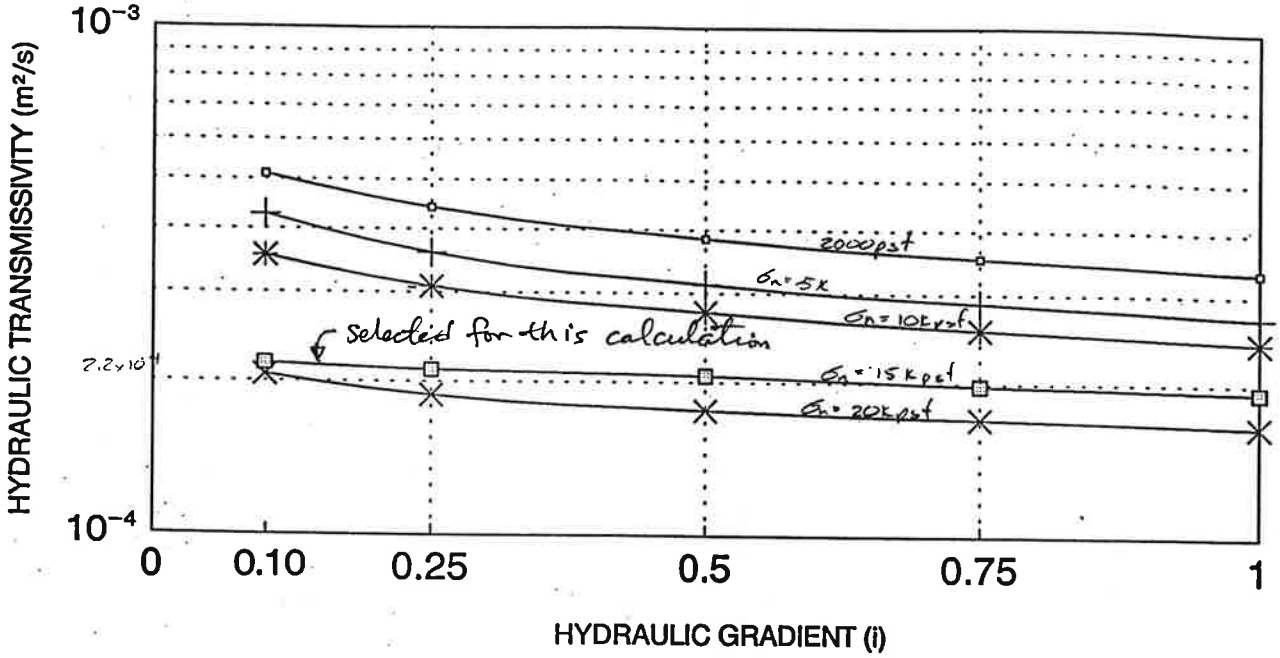
800/346-9107
FAX - 513/771-4844

32 Triangle Park Drive
Suite 3201
Cincinnati, Ohio 45246

REF 3A

# TEX-NET TN3002/1125

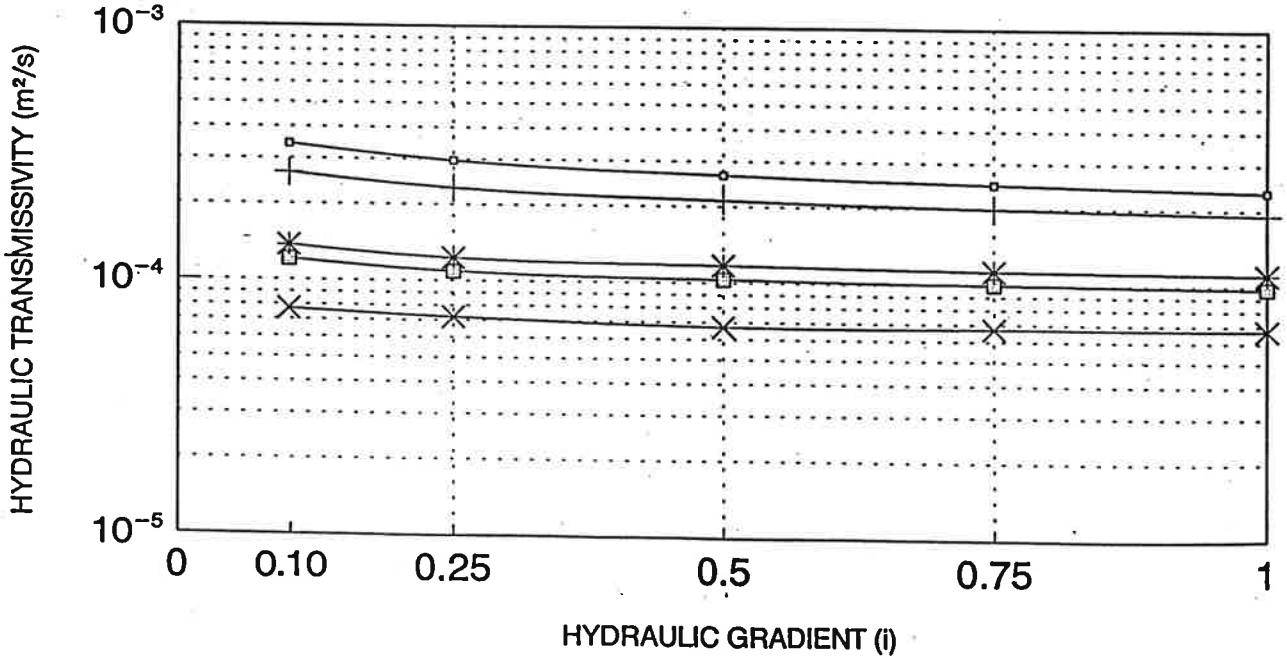
plate/FRICTION SEAL/TN3002/1125/FRICTION SEAL/plate




| | |
|---|---|
| ▫ | 2000 psf |
| + | 5000 psf |
| ✳ | 10000 psf |
| ⊞ | 15000 psf |
| × | 20000 psf |

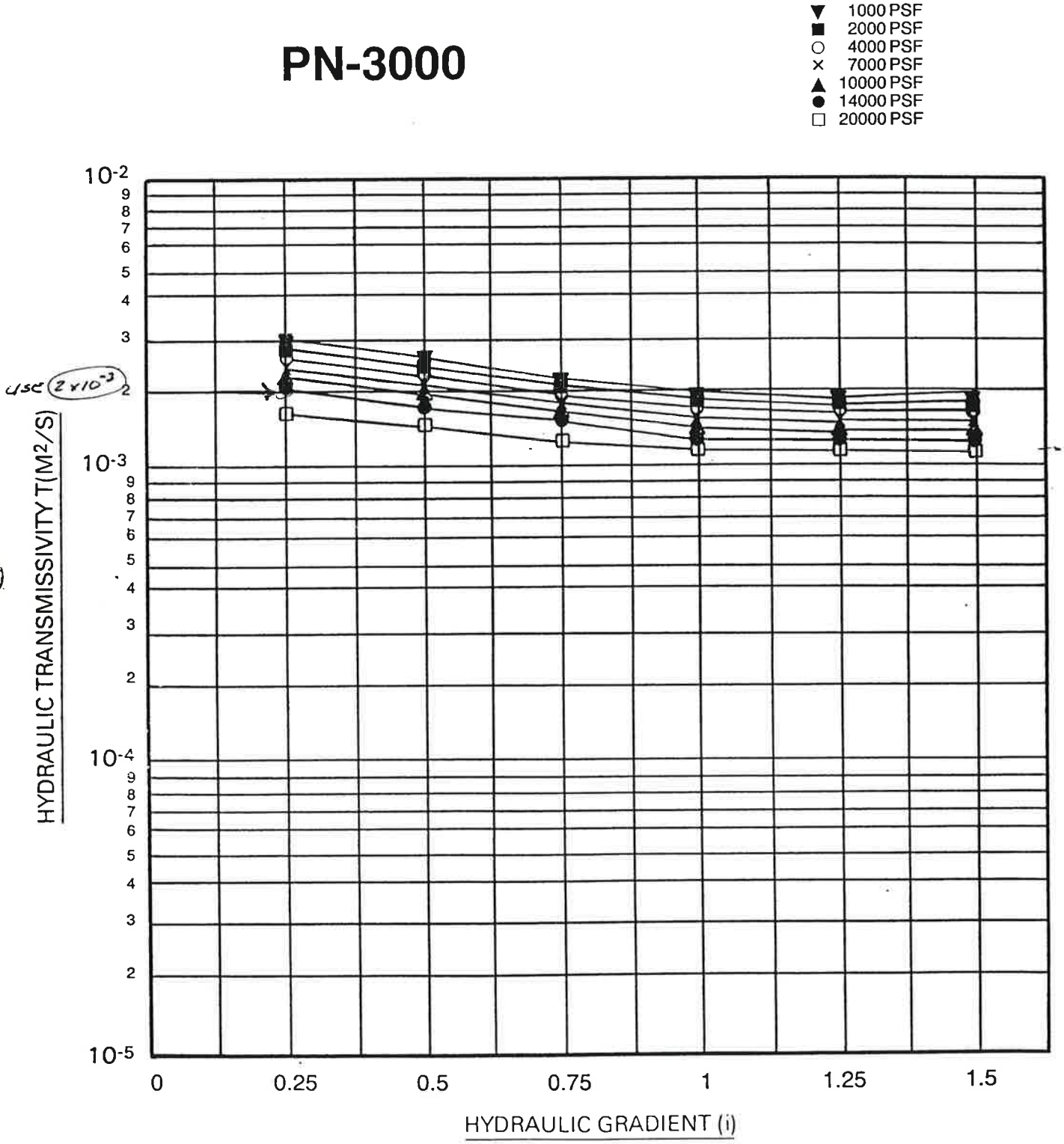
# TEX-NET TN3002CN/1125

plate/FRICTION SEAL/TN3002CN/1125/FRICTION SEAL/plate



| POLY-NET® TRANSMISSIVITY | INSTALLATION METHOD<br>LEAK DETECTION<br>HDPE LINER/POLY-NET/HDPE LINER | HYDRAULIC PRESSURE |
|---|---|---|

# PN-3000

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FSI 6/88

REPORTS
US EPA
530/R-92-004
1992

REF 4
EPA 530-R-92-004
PB92-128214
SHEET 18 OF 20

# ACTION LEAKAGE RATES FOR LEAK DETECTION SYSTEMS

[Supplemental Background Document for
the Final Double Liners and Leak Detection Systems Rule for
Hazardous Waste Landfills, Waste Piles, and Surface Impoundments]

U. S. ENVIRONMENTAL PROTECTION AGENCY
Office of Solid Waste
January 1992



REPRODUCED BY
U.S. DEPARTMENT OF COMMERCE
NATIONAL TECHNICAL
INFORMATION SERVICE
SPRINGFIELD, VA 22161

$B_{avg}$ = average width of the flow in the leak detection system, perpendicular to the flow.

Assumming that the gradient of flow through the hole, at the hole, is sin $\alpha$ and depth of flow at the hole for concentrated flow = the thickness of the drainage layer:

$$B_{avg} = D/\sin \alpha$$

where D = leak detection system thickness.

Then, with D = 1 ft and sin $\alpha$ = 0.01, $B_{avg}$ = 100 ft
0.02, $B_{avg}$ = 50 ft
0.03, $B_{avg}$ = 33 ft.

Using these values for $B_{avg}$ and Equation 1 with h $\approx$ D = 1 ft (h $\approx$ D for small values of $\alpha$), Q in gpad =

| k (cm/sec) | sin $\alpha$ | $B_{avg}$ (ft) | | |
|---|---|---|---|---|
| | | 33 | 50 | 100 |
| 1 | .01 | ---- | ---- | 21,000 |
| | .02 | ---- | 21,000 | ---- |
| | .03 | 21,000 | ---- | ---- |
| .1 | .01 | ---- | ---- | 2,100 |
| | .02 | ---- | 2,100 | ---- |
| | .03 | 2,100 | ---- | ---- |
| .01 | .01 | ---- | ---- | 210 |
| | .02 | ---- | 210 | ---- |
| | .03 | 210 | ---- | ---- |

Thus, using the minimum specifications in today's rule: 1% slope, 12 in thick drainage layer, and 1 X $10^{-1}$ cm/sec hydraulic conductivity for surface impoundments and 1 X $10^{-2}$ cm/sec hydraulic conductivity for landfills and waste piles, and assuming that the head is 1 ft and the average width of flow ($B_{avg}$) is as given above, the results show maximum flow rates of 2,100 gpad for surface impoundments and 210 gpad for landfills and waste piles. Using a safety factor of two, as suggested in the example given in the proposed rule preamble, yields about 1,000 gpad for surface impoundments and 100 gpad for landfills and waste piles as the Agency recommended action leakage rates, for units that are designed to the minimum specifications in today's rule. As listed in the rule and above, the safety factor helps account for uncertainties in the design, construction, operation, and location of the drainage layer and potential decreases in flow over time as a result of overburden compressive forces and clogging caused by fines and biological and chemical actions in any leachate that seeps through. Of course, all of the above mechanisms that could result in potential decreases in flow over time should also be considered when selecting the design, especially the hydraulic conductivity of the drainage layer, and in construction. Because this calculation used the

increasing length of run from 20 ft to 80 ft [Table 1; Figure 4 shows that length of run has negligible effect for slopes at or greater than the 1% minimum]; 43% increase when hole size is increased from .25 $ft^2$ to 1.0 $ft^2$ but a much less significant increase for holes > 3 $ft^2$ [Table 2; Figure 5 graphically shows the effect of leak size on flow rates]). However, the effect of these three variables is relatively insignificant compared to hydraulic conductivity, head, and drainage layer thickness (e.g., ten times increase (900%) when increased from .01 cm/sec to .1 cm/sec hydraulic conductivity [Tables 1, 3-5]; 382% increase when increased from no head to 2 ft head above the top liner, e.g., in a 2 ft deep surface impoundment [Table 3]; and 210% increase when geonet thickness is doubled from 5 mm to 10 mm [Table 5]).

Figures 2a-2d (side view) and 3a-b (top view) show the shape of the saturated zone for various designs, assuming no head above the top liner. These show only small portions of the bottom liner are actually exposed to the 1 ft head (as assumed in the simpler models discussed above). Figures 6-8b, however, show that as the head increases, so does the area of the bottom liner exposed to the greater heads. The graph for 8 ft head for surface impoundments is almost rectangular and therefore is not shown. Table 5 and Figure 10 show the results for geonets, which because of their high hydraulic conductivities have high flow rates.

Table 4 shows flow rates of 204 gpad and 2,040 gpad respectively for the landfill and surface impoundment specifications (i.e., 1% slope and hydraulic conductivity of $10^{-1}$ cm/sec for surface impoundments and $10^{-1}$ cm/sec for landfills, but with 1 ft of head above the top liner, 180 ft length of run, and a 1 $ft^2$ hole size). Comparing the results of the 3-D model to those of Equations 1 and 3, using the 1% slope and $10^{-1}$ cm/sec hydraulic conductivity for surface impoundments, shows that if the hole size is somewhat less than .25 $ft^2$, the flow rate with a 2 ft head would be about 2100 gpad [Table 3]. For 0 ft head above the top liner, the hole would be somewhat larger than 30 $ft^2$, or close to uniform flow [Figure 5].

### 3.2 Alternative Action Leakage Rates

While EPA recommends the above action leakage rates (100 and 1,000 gpad) for units that are built to the minimum design specifications, the Agency recognizes that a number of site-specific factors affect the maximum flow capacity of a leak detection system, and owners and operators may want to propose alternative action leakage rates. For example, the leak detection system design may be different than the minimums specified in the final rule. As indicated above, the hydraulic conductivity is a factor that significantly affects the flow capacity of the system. Since they are directly proportional, a ten times increase in hydraulic conductivity (i.e., from $10^{-2}$ to $10^{-1}$ cm/sec) increases the flow capacity ten times. Therefore, EPA believes that leak detection systems with greater hydraulic conductivities would have higher action leakage rates. In addition, owners or operators may have information to justify a

# C.10 SURFACE WATER MANAGEMENT

# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 1 of 53
Chkd. By EC Date 3/4/96 SURFACE WATER MANAGEMENT Proj. No. 95-284

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| HYDROLOGY MAP | |




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# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 2 of 53

Chkd. By EC Date 3/4/96 SURFACE WATER MANAGEMENT Proj. No. 95-284

PURPOSE:

THE PURPOSE OF THESE CALCULATIONS IS TO DETERMINE THE PEAK FLOW RATE DUE TO A 25-YEAR 24-HOUR STORM EVENT AND SIZE DITCHES TO CONVEY THE FLOW AWAY FROM THE LANDFILL SITE.

METHODOLOGY:

THE HYDROLOGY STUDY WAS PERFORMED USING THE SOIL CONSERVATION SERVICE "URBAN HYDROLOGY FOR SMALL WATERSHEDS TECHNICAL RELEASE 55 (TR 55)" TABULAR HYDROGRAPH METHOD. THIS METHOD IS A PREFERRED METHOD IN STATE OF NEVADA. OPEN CHANNEL FLOW MODULE (C)1991 DEVELOPED BY HAESTAD METHOD WAS USED TO SIZE THE DOWNDRAINS AND THE DITCHES.



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# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 3 of 53

Chkd. By EC Date 3/4/96 SURFACE WATER MANAGEMENT Proj. No. 95-284

REFERENCE

- USDA URBAN HYDROLOGY FOR SMALL WATERSHEDS TECHNICAL RELEASE 55, JUNE 1986.
- USDA - SCS NATIONAL ENGINEERING HANDBOOK, SECTION 4 HYDROLOGY, DATED AUGUST 1972.
- HAESTAD METHODS, INC. - OPEN CHANNEL FLOW MODULE, DATED 1991.



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# HYDROLOGY CALCULATIONS

## Worksheet 2: Runoff curve number and runoff

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location BEATTY NEVADA Checked EC Date 3/4/96

Circle one: Present (Developed)

### 1. Runoff curve number (CN)

| Soil name and hydrologic group (appendix A) | Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio) | CN 1/ Table 2-2 | CN 1/ Fig. 2-3 | CN 1/ Fig. 2-4 | Area ☐acres ☐mi² ☐% | Product of CN x area |
|---|---|---|---|---|---|---|
| SANDY SILT B (SEE PAGE 29 | BARE SOIL | (1) 86 | (2) 98 | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | Totals = | |

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}}$ = ——— = ———; Use CN = 98

### 2. Runoff

$S = \frac{1000}{CN} - 10 = \frac{1000}{98} - 10 = 0.20$ (SEE PAGE 30)

$Q = \frac{(P - 0.2S)^2}{P + 0.8S} = \frac{(2.1 - 0.2 \times 0.2)^2}{2.1 + 0.8 \times 0.2} = 1.88$

| | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|
| Frequency ..... yr | 25 | | |
| Rainfall, P (24-hour) ..(SEE SHT. 31).. in | 2.1 | | |
| Runoff, Q ..... in (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.) | 1.88 | | |

(1) SEE PAGE 32

(2) CONSERVATIVELY, ASSUMED 100% IMPERVIOUS (DUE TO FML CAP LINER) (SEE PAGE 30)



$I_a = 0.2S = 0.2 \times 0.2 = 0.04$

$\frac{I_a}{P} = \frac{0.04}{2.1} = 0.019$ USE: 0.1 (MIN. TABULAR) } (SEE PAGE 31)

DRAINAGE AREA "A"

## Worksheet 5b: Tabular hydrograph discharge summary

Project BEATTY LANDFILL Location _____ By M.V. Date 2/23/96

Circle one: Present (Developed) _____ Frequency (yr) 25 Checked EC Date 3/4/96

| Subarea name | Basic watershed data used 1/ | | | | Select and enter hydrograph times in hours from exhibit 5- 2/ | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sub-area $T_c$ (hr) | $\Sigma T_t$ to outlet (hr) | $I_a/P$ | $A_mQ$ ($mi^2$-in) | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 |
| | | | | | Discharges at selected hydrograph times 3/ (cfs) | | | | | | | | | | | |
| A-1 | 0.10 | 0.1 | 0.1 | $1.8 \times 10^{-3}$ | 0.1 | 0.2 | 0.5 | 0.9 | 1.5 | 1.2 | 0.7 | 0.4 | 0.3 | 0.2 | 0.2 | 0.1 |
| A-2 | 0.10 | 0.0 | 0.1 | $4.7 \times 10^{-3}$ | 0.2 | 1.6 | 3.0 | 4.7 | 3.0 | 1.0 | 0.7 | 0.6 | 0.5 | 0.4 | 0.4 | 0.3 |
| A-3 | 0.10 | 0.0 | 0.1 | $3.7 \times 10^{-3}$ | 0.2 | 1.2 | 2.4 | 3.7 | 2.3 | 0.8 | 0.5 | 0.5 | 0.4 | 0.3 | 0.3 | 0.2 |
| A-4 | 0.10 | 0.0 | 0.1 | $4.3 \times 10^{-3}$ | 0.2 | 1.4 | 2.8 | 4.3 | 2.7 | 0.9 | 0.6 | 0.5 | 0.4 | 0.4 | 0.3 | 0.3 |
| A-5 | 0.10 | 0.0 | 0.1 | $1.8 \times 10^{-3}$ | 0.1 | 0.6 | 1.2 | 1.8 | 1.1 | 0.4 | 0.3 | 0.2 | 0.2 | 0.2 | 0.1 | 0.1 |
| A-6 | 0.10 | 0.0 | 0.1 | $1.5 \times 10^{-3}$ | 0.1 | 0.5 | 1.0 | 1.5 | 0.9 | 0.3 | 0.2 | 0.2 | 0.2 | 0.1 | 0.1 | 0.1 |
| A-7 | 0.10 | 0.0 | 0.1 | $4.3 \times 10^{-3}$ | 0.2 | 1.4 | 2.8 | 4.3 | 2.7 | 0.9 | 0.6 | 0.5 | 0.4 | 0.4 | 0.3 | 0.3 |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| Composite hydrograph at outlet | | | | | 1 | 7 | 14 | 21 | 14 | 5 | 4 | 3 | 2 | 2 | 2 | 1 |

1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is $A_mQ$ multiplied by tabular discharge from appropriate exhibit 5.



5 OF 53

DRAINAGE AREA "A"

## Worksheet 5a: Basic watershed data

Project BEATTY LANDFILL Location _____ By M.V. Date 2/23/96

Circle one: Present Developed ASSUMED DEVELOPED Frequency (yr) 25 Checked EC Date 3/4/96

| Subarea name | Drainage area SEE HYDROLOGY MAP $A_m$ ($mi^2$) | Time of concentration (1) $T_c$ (hr) | Travel time through subarea (1) $T_t$ (hr) | Downstream subarea names | Travel time summation to outlet (1) $\Sigma T_t$ (hr) | 24-hr Rainfall P (in) | Runoff curve number CN | Runoff Q (in) | $A_m Q$ ($mi^2$-in) | Initial abstraction (3) $I_a$ (in) | (2) (3) $I_a/P$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| A-1 | $9.4 \times 10^{-4}$ | 0.1 | 0.0 | A-2, A-4, A-7 | 0.1 | 2.1 | 98 | 1.88 | $1.8 \times 10^{-3}$ | 0.0408 | 0.1 |
| A-2 | $2.5 \times 10^{-3}$ | 0.1 | 0.1 | A-4, A-7 | 0.0 | 2.1 | 98 | 1.88 | $4.7 \times 10^{-3}$ | 0.0408 | 0.1 |
| A-3 | $2.0 \times 10^{-3}$ | 0.1 | 0.0 | A-4, A-7 | 0.0 | 2.1 | 98 | 1.88 | $3.7 \times 10^{-3}$ | 0.0408 | 0.1 |
| A-4 | $2.3 \times 10^{-3}$ | 0.1 | 0.0 | A-7 | 0.0 | 2.1 | 98 | 1.88 | $4.3 \times 10^{-3}$ | 0.0408 | 0.1 |
| A-5 | $9.4 \times 10^{-4}$ | 0.1 | 0.0 | A-7 | 0.0 | 2.1 | 98 | 1.88 | $1.8 \times 10^{-3}$ | 0.0408 | 0.1 |
| A-6 | $7.8 \times 10^{-4}$ | 0.1 | 0.0 | A-7 | 0.0 | 2.1 | 98 | 1.88 | $1.5 \times 10^{-3}$ | 0.0408 | 0.1 |
| A-7 | $2.3 \times 10^{-3}$ | 0.1 | 0.0 | — | 0.0 | 2.1 | 98 | 1.88 | $4.3 \times 10^{-3}$ | 0.0408 | 0.1 |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |

From worksheet 3 From worksheet 2 From table 5-1

(1) ROUNDED VALUE

(2) $\frac{I_a}{P} = 0.0194 \Rightarrow$ USE: $\frac{I_a}{P} = 0.1$

(3) SEE SHEET 4

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SUB-AREA A-1

# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location Checked EC Date 3/4/96

Circle one: Present (Developed)

Circle one: ($T_c$) $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | | Segment ID | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) ..... | | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) .. ASSUMED | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... | ft | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... | in | 1.0 | | |
| 5. Land slope, s ..... | ft/ft | 0.14 | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute $T_t$ ..... | hr | 0.08 | + | = 0.08 |

$S = \frac{2861 - 2819}{300} = 0.14$

| Shallow concentrated flow | | Segment ID | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... | | | | |
| 8. Flow length, L ..... | ft | | | |
| 9. Watercourse slope, s ..... | ft/ft | | | |
| 10. Average velocity, V (figure 3-1) ..... | ft/s | | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | 0.0 | + | = 0.0 |

| Channel flow | | Segment ID | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a ..... | $ft^2$ | | | |
| 13. Wetted perimeter, $P_w$ ..... | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r ..... | ft | | | |
| 15. Channel slope, s ..... | ft/ft | | | |
| 16. Manning's roughness coeff., n ..... | | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... | ft/s | | | |
| 18. Flow length, L ..... | ft | | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.08

# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location Checked EC Date 3/4/96

Circle one: Present (Developed)

Circle one: ($T_c$) $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

**Sheet flow** (Applicable to $T_c$ only)

| | | Segment ID | |
|---|---|---|---|
| 1. Surface description (table 3-1) | | BARE SOIL | |
| 2. Manning's roughness coeff., n (table 3-1) | | 0.025 | |
| 3. Flow length, L (total L ≤ 300 ft) | ft | 300 | |
| 4. Two-yr 24-hr rainfall, $P_2$ | in | 1.0 | |
| 5. Land slope, s | ft/ft | 0.113 | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute $T_t$ | hr | 0.08 + | = 0.08 |

$S = \frac{2819-2785}{300} = 0.113$

**Shallow concentrated flow**

| | | Segment ID | |
|---|---|---|---|
| 7. Surface description (paved or unpaved) ASSUMED PAVED | | PAVED | |
| 8. Flow length, L | ft | 400 | |
| 9. Watercourse slope, s | ft/ft | 0.005 | |
| 10. Average velocity, V (figure 3-1) | ft/s | 1.4 | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ | hr | 0.08 + | = 0.08 |

**Channel flow**

| | | Segment ID | |
|---|---|---|---|
| 12. Cross sectional flow area, a | $ft^2$ | | |
| 13. Wetted perimeter, $p_w$ | ft | | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r | ft | | |
| 15. Channel slope, s | ft/ft | | |
| 16. Manning's roughness coeff., n | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V | ft/s | | |
| 18. Flow length, L | ft | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ | hr | + | = |
| 20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) | hr | | 0.16 |

# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location Checked EC Date 3/4/96

Circle one: Present Developed

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | Segment ID | | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) ..... | | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... ft | | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... in | | 1.0 | | |
| $S = \frac{2861-2821}{300} = .133$ 5. Land slope, s ..... ft/ft | | 0.133 | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5} \cdot s^{0.4}}$ Compute $T_t$ ..... hr | | 0.08 | + | = 0.08 |

| Shallow concentrated flow | Segment ID | | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... | | PAVED | | |
| 8. Flow length, L ..... ft | | 75 | | |
| 9. Watercourse slope, s ..... ft/ft | | 0.005 | | |
| 10. Average velocity, V (figure 3-1) ..... ft/s | | 1.4 | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... hr | | 0.01 | + | = 0.01 |

| Channel flow | Segment ID | | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a ..... $ft^2$ | | | | |
| 13. Wetted perimeter, $P_w$ ..... ft | | | | |
| 14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r ..... ft | | | | |
| 15. Channel slope, s ..... ft/ft | | | | |
| 16. Manning's roughness coeff., n ..... | | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... ft/s | | | | |
| 18. Flow length, L ..... ft | | | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... hr | | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.09

SUB-AREA AREA A-4

# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location _____ Checked EC Date 3/4/96

Circle one: Present (Developed)

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | | Segment ID | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) ..... | | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... | ft | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... | in | 1.0 | | |
| 5. Land slope, s ..... | ft/ft | 0.117 | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute $T_t$ ..... | hr | 0.08 | + | = 0.08 |

$S = \frac{2819-2784}{300} = 0.117$

| Shallow concentrated flow | | Segment ID | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... | | PAVED | | |
| 8. Flow length, L ..... | ft | 260 | | |
| 9. Watercourse slope, s ..... | ft/ft | 0.005 | | |
| 10. Average velocity, V (figure 3-1) ..... | ft/s | 1.4 | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | 0.05 | + | = 0.05 |

| Channel flow | | Segment ID | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a ..... | $ft^2$ | | | |
| 13. Wetted perimeter, $P_w$ ..... | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r ..... | ft | | | |
| 15. Channel slope, s ..... | ft/ft | | | |
| 16. Manning's roughness coeff., n ..... | | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... | ft/s | | | |
| 18. Flow length, L ..... | ft | | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.14

SUB-AREA A-5



# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/28/96

Location _____ Checked EC Date 3/4/96

Circle one: Present Developed

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

**Sheet flow** (Applicable to $T_c$ only)

| | | Segment ID | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) ..... | | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... | ft | 290 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... | in | 1.0 | | |
| 5. Land slope, s ..... | ft/ft | 0.09 | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5} \cdot s^{0.4}}$ Compute $T_t$ ..... | hr | 0.09 | + | = 0.09 |

$S = \frac{2845-2819}{290} = 0.09$

**Shallow concentrated flow**

| | | Segment ID | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... | | | | |
| 8. Flow length, L ..... | ft | | | |
| 9. Watercourse slope, s ..... | ft/ft | | | |
| 10. Average velocity, V (figure 3-1) ..... | ft/s | | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | + | = |

**Channel flow**

| | | Segment ID | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a ..... | $ft^2$ | | | |
| 13. Wetted perimeter, $P_w$ ..... | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r ..... | ft | | | |
| 15. Channel slope, s ..... | ft/ft | | | |
| 16. Manning's roughness coeff., n ..... | | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... | ft/s | | | |
| 18. Flow length, L ..... | ft | | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.09

SUB-AREA A-6



# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location _____ Checked EC Date 3/4/96

Circle one: Present Developed

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | | Segment ID | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) ..... | | BARE SURFACE | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... | ft | 230 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... | in | 1.0 | | |
| 5. Land slope, s ..... | ft/ft | 0.078 | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5} \cdot s^{0.4}}$ Compute $T_t$ ..... | hr | 0.08 | + | = 0.08 |

$s = \frac{2833-2815}{230} = 0.078$

| Shallow concentrated flow | | Segment ID | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... | | | | |
| 8. Flow length, L ..... | ft | | | |
| 9. Watercourse slope, s ..... | ft/ft | | | |
| 10. Average velocity, V (figure 3-1) ..... | ft/s | | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | + | = 0.00 |

| Channel flow | | Segment ID | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a ..... | $ft^2$ | | | |
| 13. Wetted perimeter, $P_w$ ..... | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r ..... | ft | | | |
| 15. Channel slope, s ..... | ft/ft | | | |
| 16. Manning's roughness coeff., n ..... | | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... | ft/s | | | |
| 18. Flow length, L ..... | ft | | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.08

SUB-AREA AREA A-7



## Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location Checked EC Date 3/4/96

Circle one: Present (Developed)

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | Segment ID | | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) ..... | | BARE SOIL | | |
| 2. Manning's roughness coeff., n ~~(table 3-1)~~ ASSUMED .. | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... | ft | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... | in | 1.0 | | |
| 5. Land slope, s ..... | ft/ft | 0.130 | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5} \cdot s^{0.4}}$ Compute $T_t$ ..... | hr | 0.08 | + | = 0.08 |

$S = \frac{2819 - 2780}{300} = 0.13$

| Shallow concentrated flow | Segment ID | | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... ASSUMED PAVED | | PAVED | | |
| 8. Flow length, L ..... | ft | 180 | | |
| 9. Watercourse slope, s ..... | ft/ft | 0.005 | | |
| 10. Average velocity, V (figure 3-1) ..... | ft/s | 1.4 | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | 0.04 | + | = 0.04 |

| Channel flow | Segment ID | | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a ..... | $ft^2$ | | | |
| 13. Wetted perimeter, $p_w$ ..... | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ..... | ft | | | |
| 15. Channel slope, s ..... | ft/ft | | | |
| 16. Manning's roughness coeff., n ..... | | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... | ft/s | | | |
| 18. Flow length, L ..... | ft | | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | + | = |
| 20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... | hr | | | 0.12 |

DRAINAGE AREA "B"

## Worksheet 5b: Tabular hydrograph discharge summary

Project BEATTY LANDFILL Location _____ By M.V. Date 2/23/96

Circle one: Present Developed _____ Frequency (yr) _____ Checked EE Date 3/4/96

| Subarea name | Basic watershed data used 1/ | | | | Select and enter hydrograph times in hours from exhibit 5- 2/ | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sub-area $T_c$ (hr) | $\Sigma T_t$ to outlet (hr) | $I_a/P$ | $A_mQ$ ($mi^2$-in) | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 |
| | | | | | Discharges at selected hydrograph times 3/ -(cfs)- | | | | | | | | | | | |
| B-1 | 0.10 | 0.10 | 0.1 | $1.5\times10^{-3}$ | 0.1 | 0.2 | 0.4 | 0.8 | 1.3 | 1.1 | 0.6 | 0.3 | 0.2 | 0.2 | 0.1 | 0.1 |
| B-2 | 0.10 | 0.10 | 0.1 | $3.0\times10^{-3}$ | 0.1 | 0.4 | 0.8 | 1.6 | 2.5 | 2.1 | 1.1 | 0.7 | 0.5 | 0.4 | 0.3 | 0.2 |
| B-3 | 0.20 | 0.10 | 0.1 | $1.2\times10^{-2}$ | 0.5 | 1.0 | 2.0 | 3.9 | 7.2 | 8.8 | 6.8 | 4.3 | 2.7 | 1.9 | 1.5 | 1.0 |
| B-4 | 0.10 | 0.00 | 0.1 | $1.8\times10^{-3}$ | 0.1 | 0.6 | 1.2 | 1.8 | 1.1 | 0.4 | 0.3 | 0.2 | 0.2 | 0.2 | 0.1 | 0.1 |
| B-5 | 0.20 | 0.00 | 0.1 | $1.1\times10^{-2}$ | 0.5 | 2.3 | 4.5 | 8.3 | 8.8 | 5.3 | 2.8 | 1.8 | 1.4 | 1.1 | 0.9 | 0.8 |
| B-6 | 0.10 | 0.00 | 0.1 | $3.6\times10^{-3}$ | 0.2 | 1.2 | 2.3 | 3.6 | 2.3 | 0.8 | 0.5 | 0.4 | 0.4 | 0.3 | 0.3 | 0.2 |
| B-7 | 0.20 | 0.00 | 0.1 | $4.4\times10^{-3}$ | 0.2 | 0.9 | 1.8 | 3.3 | 3.5 | 2.1 | 1.1 | 0.7 | 0.6 | 0.4 | 0.4 | 0.3 |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| Composite hydrograph at outlet | | | | | 2 | 7 | 13 | 23 | 27 | 21 | 13 | 8 | 6 | 5 | 4 | 3 |

1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is $A_mQ$ multiplied by tabular discharge from appropriate exhibit 5.

DRAINAGE AREA "B"

## Worksheet 5a: Basic watershed data

Project BEATTY LANDFILL Location _____ By M.V. Date 2/23/96

Circle one: Present (Developed) _____ Frequency (yr) 25 Checked EC Date 3/4/96

| Subarea name | Drainage area $A_m$ ($mi^2$) | Time of concentration $T_c$ (hr) | Travel time through subarea $T_t$ (hr) | Downstream subarea names | Travel time summation to outlet $\Sigma T_t$ (hr) | 24-hr Rainfall P (in) | Runoff curve number CN | Runoff Q (in) | $A_m Q$ ($mi^2$-in) | Initial abstraction $I_a$ (in) | $I_a/P$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| B-1 | $7.8 \times 10^{-4}$ | 0.10 | 0.00 | B-2, B-5, B-7 | 0.10 | 2.1 | 98 | 1.88 | $1.5 \times 10^{-3}$ | 0.0408 | 0.1 |
| B-2 | $1.6 \times 10^{-3}$ | 0.10 | 0.00 | B-5, B-7 | 0.10 | 2.1 | 98 | 1.88 | $3.0 \times 10^{-3}$ | 0.0408 | 0.1 |
| B-3 | $6.4 \times 10^{-3}$ | 0.20 | 0.10 | B-5, B-7 | 0.10 | 2.1 | 98 | 1.88 | $1.2 \times 10^{-2}$ | 0.0408 | 0.1 |
| B-4 | $9.4 \times 10^{-4}$ | 0.10 | 0.00 | B-7 | 0.00 | 2.1 | 98 | 1.88 | $1.8 \times 10^{-3}$ | 0.0408 | 0.1 |
| B-5 | $6.1 \times 10^{-3}$ | 0.20 | 0.10 | B-7 | 0.00 | 2.1 | 98 | 1.88 | $1.1 \times 10^{-2}$ | 0.0408 | 0.1 |
| B-6 | $1.9 \times 10^{-3}$ | 0.10 | 0.00 | B-7 | 0.00 | 2.1 | 98 | 1.88 | $3.6 \times 10^{-3}$ | 0.0408 | 0.1 |
| B-7 | $2.3 \times 10^{-3}$ | 0.20 | 0.00 | – | 0.00 | 2.1 | 98 | 1.88 | $4.4 \times 10^{-3}$ | 0.0408 | 0.1 |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |

From worksheet 3 From worksheet 2 From table 5-1

SUB_AREA B-1



# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location _____ Checked EC Date 3/4/96

Circle one: Present Developed

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | | Segment ID | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) ..... | | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... | ft | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... | in | 1.0 | | |
| 5. Land slope, s ..... $\frac{2861-2840}{300}$ | ft/ft | 0.070 | | |
| 6. $T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\,s^{0.4}}$ Compute $T_t$ ..... | hr | 0.10 | + | = 0.10 |

| Shallow concentrated flow | | Segment ID | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... | | | | |
| 8. Flow length, L ..... | ft | | | |
| 9. Watercourse slope, s ..... | ft/ft | | | |
| 10. Average velocity, V (figure 3-1) ..... | ft/s | | | |
| 11. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ ..... | hr | | + | = 0.0 |

| Channel flow | | Segment ID | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a ..... | $ft^2$ | | | |
| 13. Wetted perimeter, $p_w$ ..... | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ..... | ft | | | |
| 15. Channel slope, s ..... | ft/ft | | | |
| 16. Manning's roughness coeff., n ..... | | | | |
| 17. $V = \frac{1.49\,r^{2/3}\,s^{1/2}}{n}$ Compute V ..... | ft/s | | | |
| 18. Flow length, L ..... | ft | | | |
| 19. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ ..... | hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.10

SUB-AREA AREA B-2

# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location Checked EC Date 3/4/96

Circle one: Present Developed

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | Segment ID | | |
|---|---|---|---|
| 1. Surface description (table 3-1) ..... | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... ft | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... in | 1.0 | | |
| 5. Land slope, s = (2847.5 − 2809) / 300 ..... ft/ft | 0.128 | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5} \cdot s^{0.4}}$ Compute $T_t$ ..... hr | 0.08 | + | = 0.08 |

| Shallow concentrated flow | Segment ID | | |
|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... | | | |
| 8. Flow length, L ..... ft | | | |
| 9. Watercourse slope, s ..... ft/ft | | | |
| 10. Average velocity, V (figure 3-1) ..... ft/s | | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... hr | | + | = 0.00 |

| Channel flow | Segment ID | | |
|---|---|---|---|
| 12. Cross sectional flow area, a ..... $ft^2$ | | | |
| 13. Wetted perimeter, $P_w$ ..... ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r ..... ft | | | |
| 15. Channel slope, s ..... ft/ft | | | |
| 16. Manning's roughness coeff., n ..... | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... ft/s | | | |
| 18. Flow length, L ..... ft | | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.08

SUB-AREA B-3

## Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location _____ Checked EC Date 3/4/96

Circle one: Present Developed

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

**Sheet flow** (Applicable to $T_c$ only)

| | | Segment ID | | | |
|---|---|---|---|---|---|
| 1. Surface description (table 3-1) | | BARE SOIL | | | |
| 2. Manning's roughness coeff., n (table 3-1) | | 0.025 | | | |
| 3. Flow length, L (total L ≤ 300 ft) | ft | 300 | | | |
| 4. Two-yr 24-hr rainfall, $P_2$ | in | 1.0 | | | |
| 5. Land slope, s $\frac{2792.5 - 2786.5}{300}$ | ft/ft | 0.02 | | | |
| 6. $T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5} \cdot s^{0.4}}$ Compute $T_t$ | hr | 0.17 | + | = | 0.17 |

**Shallow concentrated flow**

| | | Segment ID | | | |
|---|---|---|---|---|---|
| 7. Surface description (paved or unpaved) | | PAVED | | | |
| 8. Flow length, L | ft | 430 | | | |
| 9. Watercourse slope, s | ft/ft | 0.005 | | | |
| 10. Average velocity, V (figure 3-1) | ft/s | 1.4 | | | |
| 11. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ | hr | 0.09 | + | = | 0.09 |

**Channel flow**

| | | Segment ID | | | |
|---|---|---|---|---|---|
| 12. Cross sectional flow area, a | $ft^2$ | | | | |
| 13. Wetted perimeter, $p_w$ | ft | | | | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r | ft | | | | |
| 15. Channel slope, s | ft/ft | | | | |
| 16. Manning's roughness coeff., n | | | | | |
| 17. $V = \frac{1.49\, r^{2/3}\, s^{1/2}}{n}$ Compute V | ft/s | | | | |
| 18. Flow length, L | ft | | | | |
| 19. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ | hr | | + | = | |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.26

SUB-AREA B-4

# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location _____ Checked EC Date 3/4/96

Circle one: Present Developed

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

**Sheet flow** (Applicable to $T_c$ only)

| | | Segment ID | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) | | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) | ft | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ | in | 1.0 | | |
| 5. Land slope, s $\frac{2840-2811}{300}$ | ft/ft | 0.097 | | |
| 6. $T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\,s^{0.4}}$ Compute $T_t$ | hr | 0.09 | + | = 0.09 |

**Shallow concentrated flow**

| | | Segment ID | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) | | | | |
| 8. Flow length, L | ft | | | |
| 9. Watercourse slope, s | ft/ft | | | |
| 10. Average velocity, V (figure 3-1) | ft/s | | | |
| 11. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ | hr | | + | = 0.00 |

**Channel flow**

| | | Segment ID | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a | $ft^2$ | | | |
| 13. Wetted perimeter, $P_w$ | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r | ft | | | |
| 15. Channel slope, s | ft/ft | | | |
| 16. Manning's roughness coeff., n | | | | |
| 17. $V = \frac{1.49\,r^{2/3}\,s^{1/2}}{n}$ Compute V | ft/s | | | |
| 18. Flow length, L | ft | | | |
| 19. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ | hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.09

SUB-AREA B-5

# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location _____ Checked EC Date 3/4/96

Circle one: Present Developed

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | Segment ID | | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) ..... | | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... ft | | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... in | | 1.0 | | |
| 5. Land slope, s = (2192.5 − 2188) / 300 ..... ft/ft | | 0.015 | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute $T_t$ ..... hr | | 0.19 | + | = 0.19 |

| Shallow concentrated flow | Segment ID | | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... ASSUMED PAVED | | PAVED | | |
| 8. Flow length, L ..... ft | | 530 | | |
| 9. Watercourse slope, s ..... ft/ft | | 0.005 | | |
| 10. Average velocity, V (figure 3-1) ..... ft/s | | 1.4 | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... hr | | 0.11 | + | = 0.11 |

| Channel flow | Segment ID | | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a ..... $ft^2$ | | | | |
| 13. Wetted perimeter, $p_w$ ..... ft | | | | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ..... ft | | | | |
| 15. Channel slope, s ..... ft/ft | | | | |
| 16. Manning's roughness coeff., n ..... | | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... ft/s | | | | |
| 18. Flow length, L ..... ft | | | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... hr | | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.30

SUB-AREA B-6

## Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location Checked EC Date 3/4/96

Circle one: Present Developed

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | Segment ID | | |
|---|---|---|---|
| 1. Surface description (table 3-1) ..... | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... ft | 240 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... in | 1.0 | | |
| 5. Land slope, s = (2833 − 2811) / 240 ..... ft/ft | 0.092 | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute $T_t$ ..... hr | 0.08 | + | = 0.08 |

| Shallow concentrated flow | Segment ID | | |
|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... | | | |
| 8. Flow length, L ..... ft | | | |
| 9. Watercourse slope, s ..... ft/ft | | | |
| 10. Average velocity, V (figure 3-1) ..... ft/s | | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... hr | | + | = 0.00 |

| Channel flow | Segment ID | | |
|---|---|---|---|
| 12. Cross sectional flow area, a ..... $ft^2$ | | | |
| 13. Wetted perimeter, $p_w$ ..... ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ..... ft | | | |
| 15. Channel slope, s ..... ft/ft | | | |
| 16. Manning's roughness coeff., n ..... | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... ft/s | | | |
| 18. Flow length, L ..... ft | | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.08

SUBAREA B-7



# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location _____ Checked EC Date 3/4/96

Circle one: Present Developed

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | Segment ID | | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) ..... | | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... | ft | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... | in | 1.0 | | |
| 5. Land slope, s = (2891 − 2882) / 300 ..... | ft/ft | 0.030 | | |
| 6. $T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\,s^{0.4}}$ Compute $T_t$ ..... | hr | 0.14 | + | = 0.14 |

| Shallow concentrated flow | Segment ID | | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... | | PAVED | | |
| 8. Flow length, L ..... | ft | 300 | | |
| 9. Watercourse slope, s ..... | ft/ft | 0.01 | | |
| 10. Average velocity, V (figure 3-1) ..... | ft/s | 2.0 | | |
| 11. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ ..... | hr | 0.04 | + | = 0.04 |

| Channel flow | Segment ID | | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a ..... | $ft^2$ | | | |
| 13. Wetted perimeter, $P_w$ ..... | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r ..... | ft | | | |
| 15. Channel slope, s ..... | ft/ft | | | |
| 16. Manning's roughness coeff., n ..... | | | | |
| 17. $V = \frac{1.49\,r^{2/3}\,s^{1/2}}{n}$ Compute V ..... | ft/s | | | |
| 18. Flow length, L ..... | ft | | | |
| 19. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ ..... | hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.18

DRAINAGE AREA "C"

## Worksheet 5b: Tabular hydrograph discharge summary

Project BEATTY LANDFILL Location _____ By M.V. Date 2/28/96

Circle one: Present (Developed) _____ Frequency (yr) 25 Checked EC Date 3/4/96

| Subarea name | Basic watershed data used 1/ | | | | Select and enter hydrograph times in hours from exhibit 5- 2/ | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sub-area $T_c$ (hr) | $\Sigma T_t$ to outlet (hr) | $I_a/P$ | $A_mQ$ ($mi^2$-in) | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 |
| | | | | | Discharges at selected hydrograph times 3/ -(cfs)- | | | | | | | | | | | |
| C-1 | 0.1 | 0.0 | 0.1 | $1.5 \times 10^{-3}$ | 0.1 | 0.5 | 1.0 | 1.5 | 0.9 | 0.3 | 0.2 | 0.2 | 0.2 | 0.1 | 0.1 | 0.1 |
| C-2 | 0.1 | 0.0 | 0.1 | $1.2 \times 10^{-3}$ | 0.1 | 0.4 | 0.8 | 1.2 | 0.8 | 0.3 | 0.2 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 |
| C-3 | 0.2 | 0.0 | 0.1 | $9.8 \times 10^{-3}$ | 0.5 | 2.0 | 3.9 | 7.2 | 7.8 | 4.7 | 2.5 | 1.6 | 1.3 | 1.0 | 0.8 | 0.7 |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| Composite hydrograph at outlet | | | | | 1 | 3 | 6 | 10 | 10 | 5 | 3 | 2 | 2 | 1 | 1 | 1 |

1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is $A_mQ$ multiplied by tabular discharge from appropriate exhibit 5.

DRAINAGE AREA "C"

## Worksheet 5a: Basic watershed data

Project BEATTY LANDFILL Location _____ By M.V. Date 2/23/96

Circle one: Present Developed _____ Frequency (yr) 25 Checked EC Date 3/4/96

| Subarea name | Drainage area $A_m$ ($mi^2$) | Time of concentration $T_c$ (hr) | Travel time through subarea $T_t$ (hr) | Downstream subarea names | Travel time summation to outlet $\Sigma T_t$ (hr) | 24-hr Rainfall P (in) | Runoff curve number CN | Runoff Q (in) | $A_mQ$ ($mi^2$-in) | Initial abstraction $I_a$ (in) | $I_a/P$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| C-1 | $7.8\times10^{-4}$ | 0.1 | 0.0 | C-2, C-3 | 0.0 | 2.1 | 98 | 1.88 | $1.5\times10^{-3}$ | 0.0408 | 0.1 |
| C-2 | $6.3\times10^{-4}$ | 0.1 | 0.0 | C-3 | 0.0 | 2.1 | 98 | 1.88 | $1.2\times10^{-3}$ | 0.0408 | 0.1 |
| C-3 | $5.2\times10^{-3}$ | 0.2 | 0.0 | — | 0.0 | 2.1 | 98 | 1.88 | $9.8\times10^{-3}$ | 0.0408 | 0.1 |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |

From worksheet 3 From worksheet 2 From table 5-1

SUB-AREA "C-1"

# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location Checked EC Date 3/4/96

Circle one: Present Developed

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

**Sheet flow** (Applicable to $T_c$ only)

| | | Segment ID | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) | | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) | ft | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ | in | 1.0 | | |
| 5. Land slope, s = (2861-2819)/300 | ft/ft | 0.140 | | |
| 6. $T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\,s^{0.4}}$ Compute $T_t$ | hr | 0.08 | + | = 0.08 |

**Shallow concentrated flow**

| | | Segment ID | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) | | | | |
| 8. Flow length, L | ft | | | |
| 9. Watercourse slope, s | ft/ft | | | |
| 10. Average velocity, V (figure 3-1) | ft/s | | | |
| 11. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ | hr | | + | = 0.00 |

**Channel flow**

| | | Segment ID | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a | $ft^2$ | | | |
| 13. Wetted perimeter, $P_w$ | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r | ft | | | |
| 15. Channel slope, s | ft/ft | | | |
| 16. Manning's roughness coeff., n | | | | |
| 17. $V = \frac{1.49\,r^{2/3}\,s^{1/2}}{n}$ Compute V | ft/s | | | |
| 18. Flow length, L | ft | | | |
| 19. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ | hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.08

SUB-AREA C-2

## Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location _____ Checked EC Date 3/4/96

Circle one: Present (Developed)

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

**Sheet flow** (Applicable to $T_c$ only)

| | | Segment ID | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) | | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) | ft | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ | in | 1.0 | | |
| 5. Land slope, s $\frac{2848-2809}{300}$ | ft/ft | 0.130 | | |
| 6. $T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\cdot s^{0.4}}$ Compute $T_t$ | hr | 0.08 | + | = 0.08 |

**Shallow concentrated flow**

| | | Segment ID | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) | | | | |
| 8. Flow length, L | ft | | | |
| 9. Watercourse slope, s | ft/ft | | | |
| 10. Average velocity, V (figure 3-1) | ft/s | | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ | hr | | + | = 0.00 |

**Channel flow**

| | | Segment ID | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a | $ft^2$ | | | |
| 13. Wetted perimeter, $p_w$ | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r | ft | | | |
| 15. Channel slope, s | ft/ft | | | |
| 16. Manning's roughness coeff., n | | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V | ft/s | | | |
| 18. Flow length, L | ft | | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ | hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.08

# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project BEATTY LANDFILL By M.V. Date 2/23/96

Location _____ Checked EC Date 3/4/96

Circle one: Present Developed

Circle one: $T_c$ $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

**Sheet flow** (Applicable to $T_c$ only)

| | | Segment ID | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) | | BARE SOIL | | |
| 2. Manning's roughness coeff., n (table 3-1) | | 0.025 | | |
| 3. Flow length, L (total L ≤ 300 ft) | ft | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ | in | 1.0 | | |
| 5. Land slope, s = (2792 − 2786) / 300 = | ft/ft | 0.02 | | |
| 6. $T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\,s^{0.4}}$ Compute $T_t$ | hr | 0.17 | + | = 0.17 |

**Shallow concentrated flow**

| | | Segment ID | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) | | PAVED | | |
| 8. Flow length, L | ft | 320 | | |
| 9. Watercourse slope, s | ft/ft | 0.005 | | |
| 10. Average velocity, V (figure 3-1) | ft/s | 1.4 | | |
| 11. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ | hr | 0.06 | + | = 0.06 |

**Channel flow**

| | | Segment ID | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a | $ft^2$ | | | |
| 13. Wetted perimeter, $P_w$ | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r | ft | | | |
| 15. Channel slope, s | ft/ft | | | |
| 16. Manning's roughness coeff., n | | | | |
| 17. $V = \frac{1.49\,r^{2/3}\,s^{1/2}}{n}$ Compute V | ft/s | | | |
| 18. Flow length, L | ft | | | |
| 19. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ | hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr 0.23

0.1 0.7
0.1 0.0

# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 28 of 53

Chkd. By EC Date 3/4/96 SURFACE WATER MANAGEMENT Proj. No. 95-284

REFERENCE MATERIALS



Recyclable and made from recycled paper.

A. (Low runoff potential). Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.

B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

D. (High runoff potential). Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

## The Soil List

The list at the end of this chapter contains the names of more than 4,000 soils in the United States and Puerto Rico. The capital letter following a name designates the hydrologic soil group classification.

The original classifications were based on the use of rainfall-runoff data from small watersheds or infiltrometer plots, but the majority are based on the judgments of soil scientists and correlators who used physical properties of the soil in making their decisions. They classified a soil in a particular group by comparing its profile with profiles of soils already classified. They assumed that the soil surfaces were bare, maximum swelling had taken place, and rainfall rates exceeded surface intake rates. Thus, most of the classifications are based on the premise that similar soils (similar in depth, organic-matter content, structure, and degree of swelling when saturated) will respond in an essentially similar manner during a rainstorm having excessive intensities.

The classification of a soil in the list can be checked by using the procedure of example 5.4. The soil in question must be the only one



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SECTION 4 - HYDROLOGY
CHAPTER 7. HYDROLOGIC SOIL
GROUPS. DATED AUGUST 1972.

## Graphs and Tables for the Solution of Equation 10.10

Sheets 1 and 2 of figure 10.1 contain graphs for the rapid solution of equation 10.10. The parameter CN (runoff curve number or hydrologic soil-cover complex number) is a transformation of S, and it is used to make interpolating, averaging, and weighting operations more nearly linear. The transformation is:

$$CN = \frac{1000}{S + 10} \qquad (10.11)$$

or

$$S = \frac{1000}{CN} - 10 \qquad (10.12)$$

Tables for the solution of equation 10.10 are given in SCS Technical Release 16 for P from zero to 40.9 inches by steps of 0.1-inch and for all whole-numbered CN in the range from 55 through 98.

USE OF S AND CN. It is more convenient to use CN on figure 10.1, but it will generally be necessary to use S for other applications such as the analysis of runoff data or the development of supplementary runoff relationships. Example 5.5 and figure 5.6(b) illustrate a typical use of S. The relationship is developed using S, but a scale for CN is added later to the graph for ease of application.

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RUNOFF FROM STORM RAINFALL
DATED AUGUST 1972.

July, 1969

which is the rainfall-runoff relation with the initial abstraction taken into account.

The initial abstraction consists mainly of interception, infiltration, and surface storage, all of which occur before runoff begins. The insert on figure 10.1 shows the position of $I_a$ in a typical storm. To remove the necessity for estimating these variables in equation 10.8, the relation between $I_a$ and S (which includes $I_a$) was developed by means of rainfall and runoff data from experimental small watersheds. The relation is discussed later in connection with figure 10.2. The empirical relationship is:

$$I_a = 0.2\ S \qquad (10.9)$$

Substituting 10.9 in 10.8 gives:

$$Q = \frac{(P - 0.2\ S)^2}{P + 0.8\ S} \qquad (10.10)$$

which is the rainfall-runoff relation used in the SCS method of estimating direct runoff from storm rainfall.

## Retention Parameters

Equation 10.9 states that 20 percent (an average, as figure 10.2 shows) of the potential maximum retention S is the initial abstraction $I_a$, which is the interception, infiltration, and surface storage occurring before runoff begins. The remaining 80 percent is mainly the infiltration occurring after runoff begins. This later infiltration is controlled by the rate of infiltration at the soil surface or by the rate of transmission in the soil profile or by the water-storage capacity of the profile, whichever is the limiting factor. A succession of storms, such as one a day for a week, reduces the magnitude of S each day because the limiting factor does not have the opportunity to completely recover its rate or capacity through weathering, evapotranspiration, or drainage. But there is enough recovery, depending on the soil-cover complex, to limit the reduction. During such a storm period the magnitude of S remains virtually the same after the second or third day even if the rains are large so that there is, from a practical viewpoint, a lower limit to S for a given soil-cover complex. Similarly there is a practical upper limit to S, again depending on the soil-cover complex, beyond which the recovery cannot take S unless the complex is altered.

In the SCS method the change in S (actually in CN) is based on an antecedent moisture condition (AMC) determined by the total rainfall

Table 2-2b.—Runoff curve numbers for cultivated agricultural lands[1]

| Cover description | | | Curve numbers for hydrologic soil group— | | | |
|---|---|---|---|---|---|---|
| Cover type | Treatment[2] | Hydrologic condition[3] | A | B | C | D |
| Fallow | Bare soil | | 77 | 86 | 91 | 94 |
| | Crop residue cover (CR) | Poor | 76 | 85 | 90 | 93 |
| | | Good | 74 | 83 | 88 | 90 |
| Row crops | Straight row (SR) | Poor | 72 | 81 | 88 | 91 |
| | | Good | 67 | 78 | 85 | 89 |
| | SR + CR | Poor | 71 | 80 | 87 | 90 |
| | | Good | 64 | 75 | 82 | 85 |
| | Contoured (C) | Poor | 70 | 79 | 84 | 88 |
| | | Good | 65 | 75 | 82 | 86 |
| | C + CR | Poor | 69 | 78 | 83 | 87 |
| | | Good | 64 | 74 | 81 | 85 |
| | Contoured & terraced (C&T) | Poor | 66 | 74 | 80 | 82 |
| | | Good | 62 | 71 | 78 | 81 |
| | C&T + CR | Poor | 65 | 73 | 79 | 81 |
| | | Good | 61 | 70 | 77 | 80 |
| Small grain | SR | Poor | 65 | 76 | 84 | 88 |
| | | Good | 63 | 75 | 83 | 87 |
| | SR + CR | Poor | 64 | 75 | 83 | 86 |
| | | Good | 60 | 72 | 80 | 84 |
| | C | Poor | 63 | 74 | 82 | 85 |
| | | Good | 61 | 73 | 81 | 84 |
| | C + CR | Poor | 62 | 73 | 81 | 84 |
| | | Good | 60 | 72 | 80 | 83 |
| | C&T | Poor | 61 | 72 | 79 | 82 |
| | | Good | 59 | 70 | 78 | 81 |
| | C&T + CR | Poor | 60 | 71 | 78 | 81 |
| | | Good | 58 | 69 | 77 | 80 |
| Close-seeded or broadcast legumes or rotation meadow | SR | Poor | 66 | 77 | 85 | 89 |
| | | Good | 58 | 72 | 81 | 85 |
| | C | Poor | 64 | 75 | 83 | 85 |
| | | Good | 55 | 69 | 78 | 83 |
| | C&T | Poor | 63 | 73 | 80 | 83 |
| | | Good | 51 | 67 | 76 | 80 |

[1]Average runoff condition, and $I_a = 0.2S$.
[2]*Crop residue cover* applies only if residue is on at least 5% of the surface throughout the year.
[3]Hydrologic condition is based on combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes in rotations, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface roughness.
*Poor:* Factors impair infiltration and tend to increase runoff.
*Good:* Factors encourage average and better than average infiltration and tend to decrease runoff.

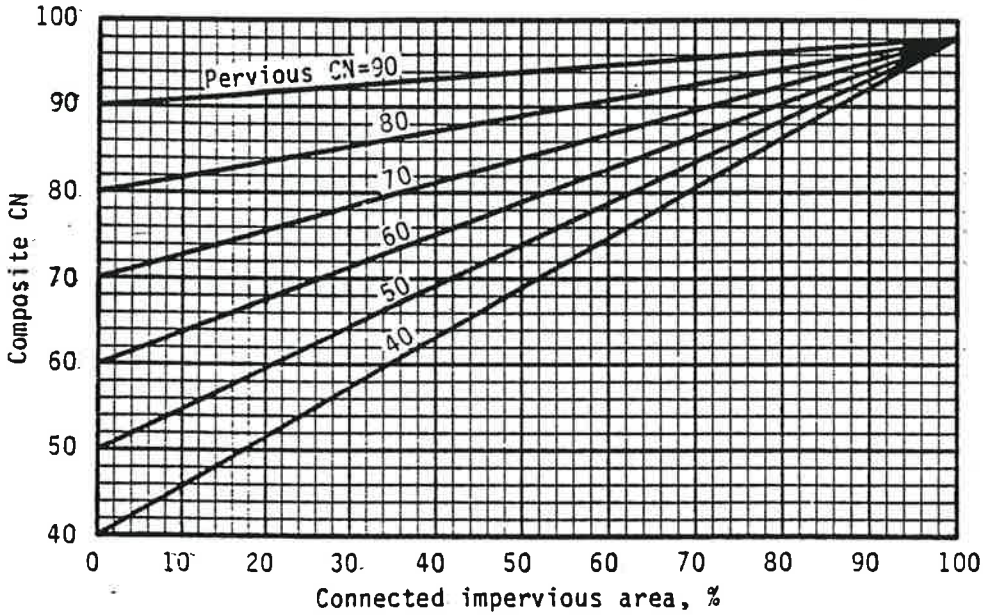


**Figure 2-3.—Composite CN with connected impervious area.**

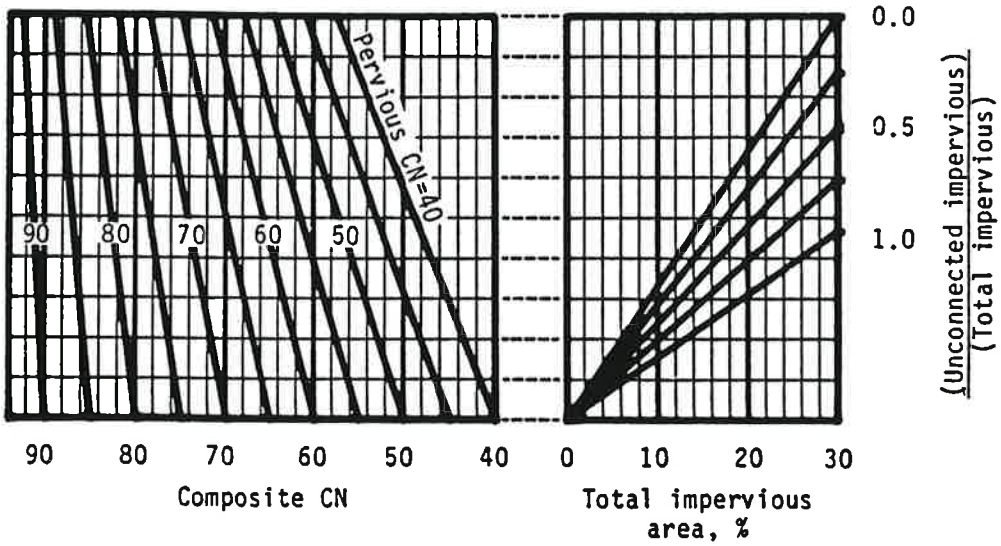


**Figure 2-4.—Composite CN with unconnected impervious areas and total impervious area less than 30%.**

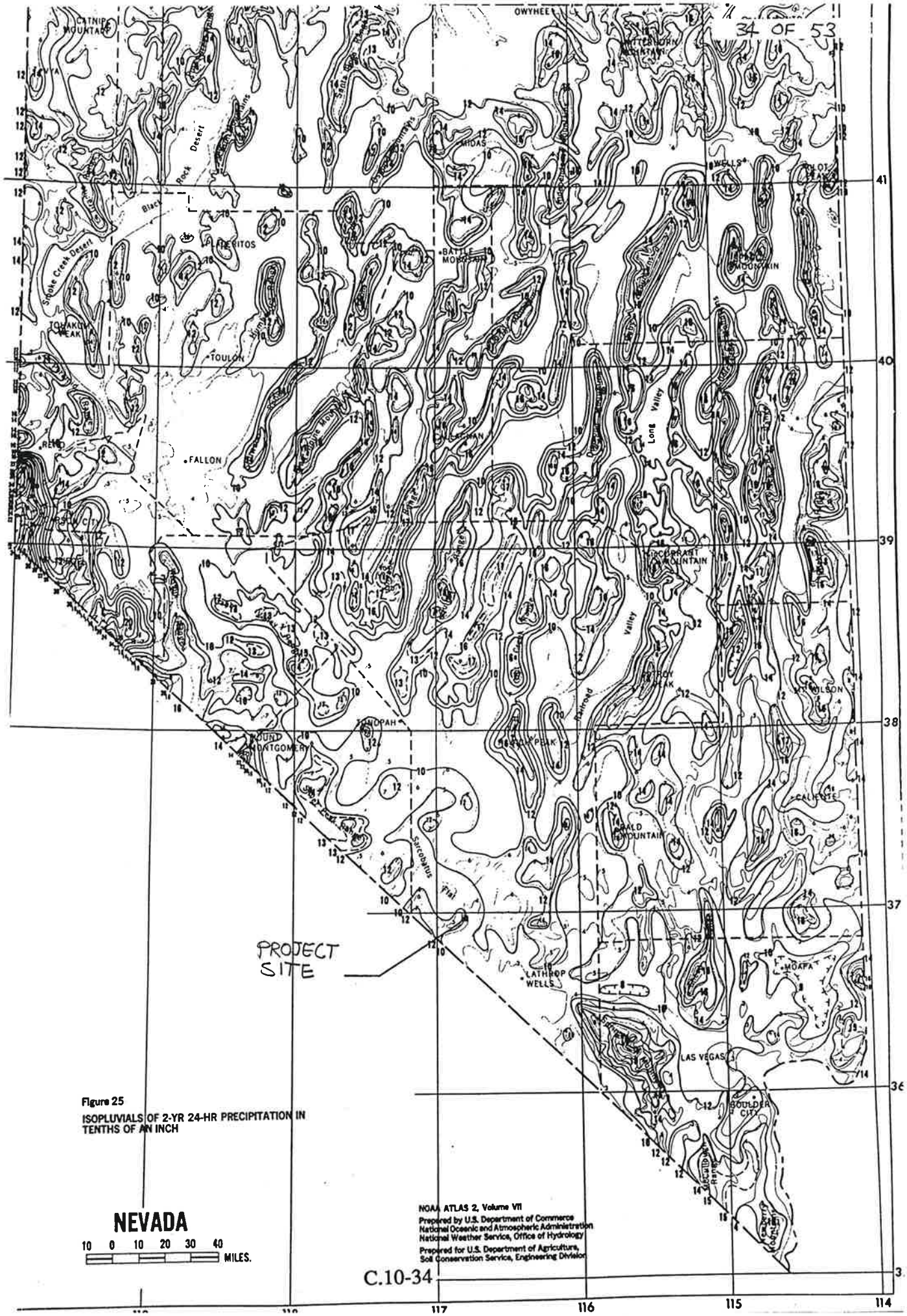


Figure 25

ISOPLUVIALS OF 2-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

NEVADA

10 0 10 20 30 40 MILES.

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National Oceanic and Atmospheric Administration
National Weather Service, Office of Hydrology

Prepared for U.S. Department of Agriculture,
Soil Conservation Service, Engineering Division

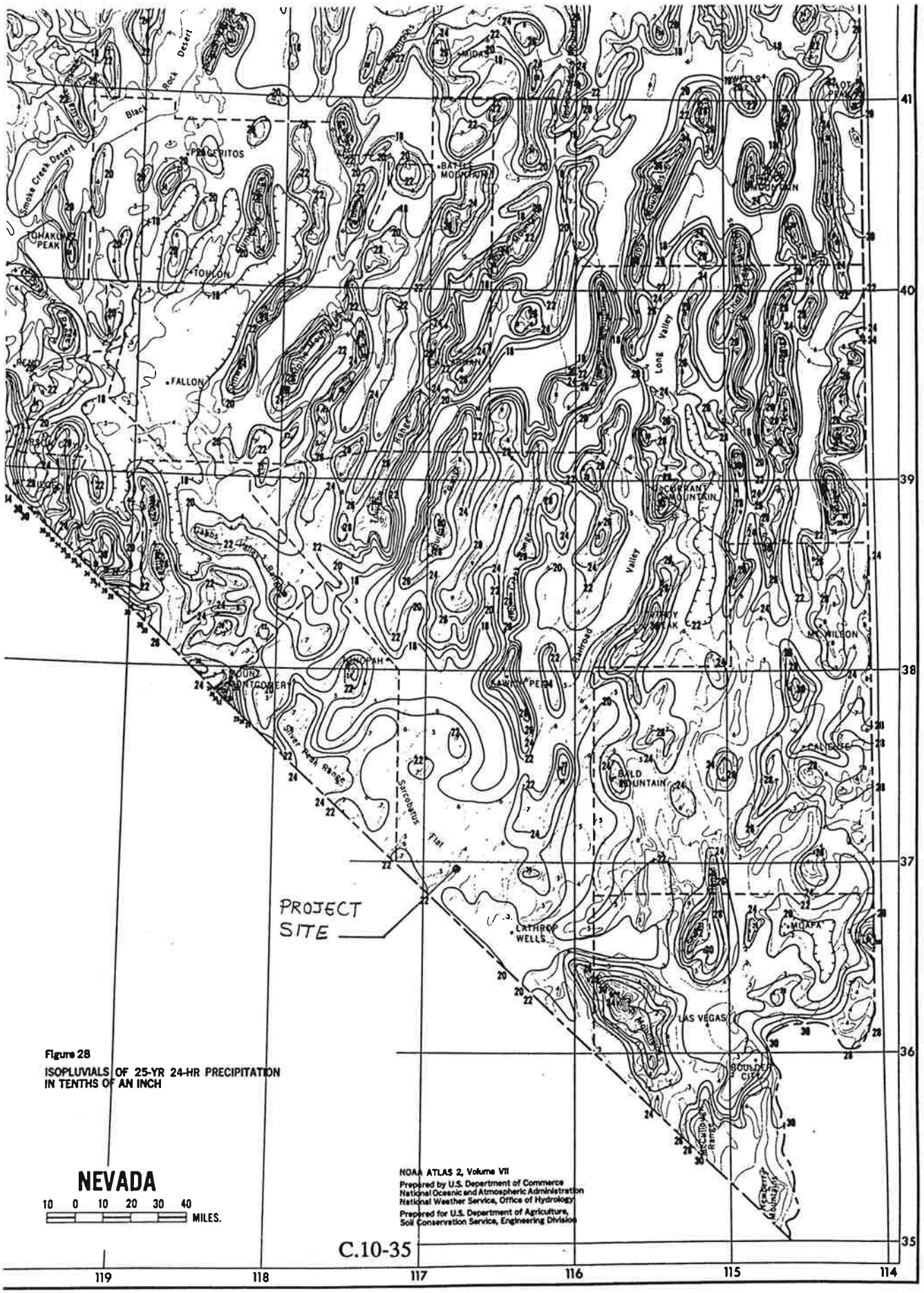


Figure 28

ISOPLUVIALS OF 25-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

NEVADA

10 0 10 20 30 40 MILES.

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Prepared by U.S. Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service, Office of Hydrology

Prepared for U.S. Department of Agriculture,
Soil Conservation Service, Engineering Division

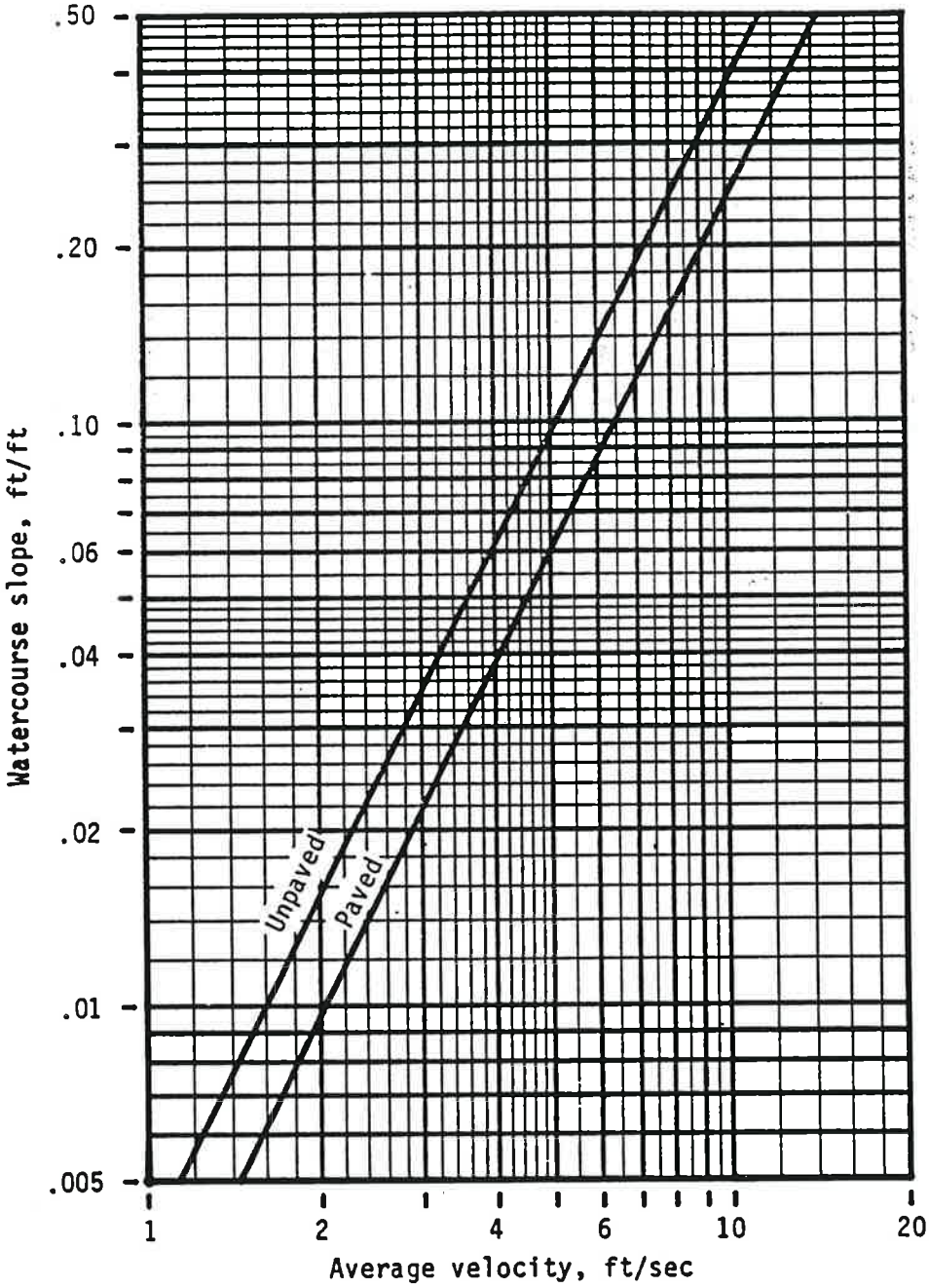



Figure 3-1.—Average velocities for estimating travel time for shallow concentrated flow.

## Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

| TRVL TIME (HR) | 11.0 | 11.3 | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 | 16.0 | 16.5 | 17.0 | 17.5 | 18.0 | 19.0 | 20.0 | 22.0 | 26.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | | | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | |
| IA/P = 0.10 | | | | | | | | | | | | | | * * * TC =0.1 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 24 | 34 | 53 | 334 | 647 | 1010 | 628 | 217 | 147 | 123 | 104 | 86 | 76 | 66 | 57 | 51 | 46 | 42 | 38 | 34 | 32 | 29 | 26 | 23 | 21 | 20 | 19 | 18 | 15 | 13 | 12 | 0 |
| .10 | 21 | 29 | 43 | 134 | 267 | 520 | 847 | 701 | 378 | 224 | 157 | 122 | 98 | 75 | 64 | 56 | 50 | 45 | 41 | 36 | 33 | 30 | 27 | 24 | 21 | 20 | 19 | 18 | 16 | 13 | 12 | 0 |
| .20 | 18 | 25 | 35 | 61 | 110 | 215 | 418 | 704 | 702 | 486 | 312 | 209 | 151 | 94 | 73 | 62 | 54 | 49 | 44 | 38 | 34 | 31 | 28 | 25 | 22 | 21 | 19 | 18 | 16 | 14 | 12 | 0 |
| .30 | 17 | 23 | 33 | 56 | 92 | 174 | 337 | 582 | 662 | 545 | 359 | 269 | 190 | 109 | 79 | 65 | 56 | 50 | 45 | 39 | 35 | 32 | 29 | 25 | 22 | 21 | 20 | 18 | 16 | 14 | 12 | 0 |
| .40 | 15 | 20 | 28 | 41 | 51 | 78 | 142 | 272 | 478 | 601 | 563 | 447 | 328 | 172 | 104 | 76 | 63 | 55 | 49 | 42 | 37 | 33 | 29 | 26 | 23 | 21 | 20 | 19 | 17 | 14 | 12 | 0 |
| .50 | 14 | 19 | 26 | 39 | 47 | 68 | 117 | 220 | 392 | 531 | 553 | 482 | 380 | 209 | 121 | 84 | 67 | 57 | 51 | 43 | 38 | 33 | 30 | 27 | 23 | 21 | 20 | 19 | 17 | 14 | 12 | 0 |
| .75 | 12 | 15 | 21 | 29 | 33 | 38 | 49 | 73 | 126 | 224 | 343 | 432 | 464 | 385 | 252 | 156 | 103 | 76 | 62 | 50 | 43 | 36 | 31 | 28 | 25 | 22 | 21 | 19 | 17 | 15 | 12 | 0 |
| 1.0 | 9 | 12 | 15 | 21 | 23 | 26 | 29 | 33 | 40 | 55 | 86 | 148 | 238 | 406 | 434 | 317 | 205 | 130 | 89 | 62 | 50 | 41 | 34 | 30 | 27 | 24 | 22 | 20 | 18 | 16 | 12 | 0 |
| 1.5 | 7 | 8 | 10 | 14 | 15 | 16 | 18 | 20 | 22 | 25 | 29 | 34 | 45 | 101 | 220 | 339 | 373 | 320 | 234 | 131 | 80 | 53 | 40 | 34 | 30 | 27 | 24 | 21 | 19 | 17 | 12 | 2 |
| 2.0 | 4 | 6 | 7 | 9 | 9 | 10 | 11 | 12 | 13 | 15 | 16 | 18 | 20 | 25 | 37 | 72 | 150 | 252 | 336 | 312 | 216 | 109 | 58 | 42 | 34 | 30 | 27 | 24 | 20 | 18 | 13 | 8 |
| 2.5 | 3 | 4 | 5 | 6 | 7 | 7 | 8 | 8 | 9 | 10 | 11 | 12 | 13 | 16 | 19 | 25 | 39 | 75 | 142 | 262 | 303 | 229 | 108 | 58 | 41 | 34 | 30 | 27 | 22 | 19 | 14 | 11 |
| 3.0 | 1 | 2 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 8 | 10 | 12 | 14 | 17 | 22 | 31 | 76 | 169 | 288 | 236 | 122 | 64 | 43 | 35 | 30 | 24 | 20 | 16 | 11 |
| IA/P = 0.30 | | | | | | | | | | | | | | * * * TC =0.1 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 154 | 568 | 936 | 524 | 217 | 172 | 149 | 126 | 107 | 97 | 86 | 76 | 69 | 63 | 58 | 53 | 48 | 46 | 42 | 38 | 34 | 31 | 30 | 28 | 27 | 24 | 20 | 19 | 0 |
| .10 | 0 | 0 | 0 | 19 | 109 | 415 | 762 | 603 | 346 | 230 | 176 | 143 | 119 | 96 | 84 | 74 | 68 | 62 | 57 | 50 | 47 | 44 | 40 | 35 | 32 | 30 | 29 | 27 | 24 | 21 | 19 | 0 |
| .20 | 0 | 0 | 0 | 0 | 13 | 77 | 302 | 609 | 605 | 432 | 297 | 217 | 167 | 115 | 94 | 81 | 73 | 66 | 60 | 53 | 48 | 45 | 41 | 37 | 33 | 31 | 29 | 28 | 25 | 21 | 19 | 0 |
| .30 | 0 | 0 | 0 | 0 | 9 | 54 | 219 | 479 | 563 | 476 | 357 | 263 | 199 | 129 | 99 | 85 | 75 | 68 | 62 | 54 | 49 | 45 | 41 | 37 | 33 | 31 | 29 | 28 | 25 | 21 | 19 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 6 | 38 | 159 | 372 | 500 | 484 | 399 | 309 | 183 | 123 | 96 | 82 | 73 | 66 | 58 | 51 | 46 | 42 | 38 | 34 | 31 | 30 | 28 | 25 | 22 | 19 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 4 | 27 | 115 | 287 | 429 | 465 | 421 | 346 | 213 | 138 | 103 | 86 | 76 | 68 | 59 | 52 | 47 | 43 | 39 | 34 | 32 | 30 | 29 | 25 | 22 | 19 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 10 | 46 | 132 | 246 | 338 | 381 | 341 | 243 | 165 | 119 | 94 | 80 | 67 | 58 | 50 | 45 | 41 | 37 | 33 | 31 | 29 | 26 | 23 | 19 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 22 | 69 | 149 | 241 | 357 | 331 | 246 | 170 | 122 | 96 | 76 | 64 | 54 | 47 | 42 | 38 | 34 | 32 | 30 | 27 | 24 | 19 | 0 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 41 | 142 | 258 | 310 | 285 | 224 | 142 | 97 | 71 | 55 | 47 | 43 | 39 | 35 | 32 | 29 | 25 | 20 | 4 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 10 | 49 | 130 | 221 | 279 | 255 | 182 | 108 | 70 | 55 | 47 | 42 | 38 | 34 | 30 | 27 | 20 | 11 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 14 | 52 | 119 | 224 | 256 | 193 | 107 | 70 | 55 | 47 | 42 | 38 | 32 | 28 | 22 | 17 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 52 | 141 | 240 | 199 | 117 | 74 | 56 | 48 | 43 | 35 | 30 | 24 | 18 |
| IA/P = 0.50 | | | | | | | | | | | | | | * * * TC =0.1 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 70 | 539 | 377 | 196 | 171 | 154 | 134 | 117 | 108 | 99 | 89 | 83 | 77 | 72 | 67 | 61 | 59 | 56 | 51 | 46 | 43 | 42 | 40 | 38 | 34 | 30 | 28 | 0 |
| .10 | 0 | 0 | 0 | 0 | 47 | 375 | 376 | 256 | 199 | 169 | 146 | 126 | 114 | 102 | 92 | 85 | 79 | 73 | 68 | 62 | 59 | 56 | 52 | 47 | 43 | 42 | 40 | 38 | 34 | 30 | 28 | 0 |
| .20 | 0 | 0 | 0 | 0 | 0 | 31 | 260 | 338 | 283 | 227 | 189 | 160 | 138 | 112 | 99 | 90 | 83 | 77 | 72 | 64 | 60 | 57 | 53 | 48 | 44 | 42 | 41 | 39 | 35 | 30 | 28 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 21 | 180 | 285 | 284 | 246 | 208 | 176 | 131 | 110 | 97 | 88 | 82 | 76 | 68 | 62 | 59 | 54 | 50 | 45 | 43 | 41 | 39 | 36 | 31 | 28 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 14 | 125 | 232 | 266 | 253 | 223 | 192 | 142 | 115 | 100 | 91 | 83 | 77 | 69 | 63 | 59 | 55 | 50 | 45 | 43 | 41 | 40 | 36 | 31 | 28 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 9 | 86 | 183 | 239 | 248 | 231 | 205 | 154 | 122 | 104 | 93 | 85 | 79 | 71 | 64 | 59 | 55 | 51 | 46 | 43 | 41 | 40 | 36 | 32 | 28 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 31 | 87 | 147 | 190 | 211 | 213 | 184 | 147 | 121 | 103 | 92 | 84 | 75 | 67 | 61 | 57 | 52 | 47 | 44 | 42 | 40 | 37 | 32 | 28 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 13 | 45 | 92 | 141 | 205 | 197 | 165 | 134 | 112 | 98 | 84 | 75 | 65 | 57 | 55 | 50 | 46 | 43 | 41 | 38 | 34 | 28 | 0 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 51 | 118 | 170 | 183 | 167 | 143 | 111 | 92 | 77 | 65 | 59 | 54 | 50 | 45 | 43 | 39 | 35 | 28 | 2 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 15 | 51 | 103 | 148 | 168 | 156 | 127 | 96 | 76 | 65 | 58 | 54 | 49 | 45 | 41 | 37 | 29 | 12 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 31 | 69 | 131 | 159 | 140 | 101 | 78 | 66 | 59 | 54 | 50 | 43 | 39 | 31 | 24 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 11 | 46 | 101 | 151 | 134 | 99 | 77 | 65 | 59 | 54 | 45 | 41 | 33 | 26 |
| RAINFALL TYPE = II | | | | | | | | | | | | | | * * * TC = 0.1 HR * * * | | | | | | | | | | | | | | | | | | SHEET 1 OF 10 |

## Exhibit 5-II, continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

HYDROGRAPH TIME (HOURS)

| TRVL TIME (HR) | 11.0 | 11.3 | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 | 16.0 | 16.5 | 17.0 | 17.5 | 18.0 | 19.0 | 20.0 | 22.0 | 26.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| * * * TC =0.2 HR * * * IA/P = 0.10 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 0.0 | 23 | 31 | 47 | 209 | 403 | 739 | 800 | 481 | 250 | 166 | 128 | 102 | 86 | 70 | 61 | 54 | 49 | 44 | 40 | 35 | 33 | 30 | 27 | 24 | 21 | 20 | 19 | 18 | 16 | 13 | 12 | 0 |
| .10 | 19 | 26 | 39 | 86 | 168 | 325 | 601 | 733 | 565 | 355 | 229 | 161 | 122 | 83 | 69 | 59 | 53 | 47 | 43 | 37 | 34 | 31 | 28 | 25 | 22 | 21 | 19 | 18 | 16 | 14 | 12 | 0 |
| .20 | 17 | 23 | 32 | 49 | 74 | 136 | 262 | 488 | 652 | 594 | 435 | 298 | 207 | 115 | 81 | 67 | 58 | 51 | 46 | 40 | 35 | 32 | 29 | 26 | 23 | 21 | 20 | 19 | 16 | 14 | 12 | 0 |
| .30 | 16 | 22 | 30 | 46 | 64 | 112 | 212 | 396 | 566 | 585 | 485 | 360 | 258 | 139 | 90 | 71 | 60 | 53 | 48 | 41 | 36 | 32 | 29 | 26 | 23 | 21 | 20 | 19 | 16 | 14 | 12 | 0 |
| .40 | 14 | 19 | 25 | 37 | 43 | 57 | 94 | 173 | 322 | 485 | 551 | 507 | 409 | 227 | 129 | 87 | 68 | 58 | 52 | 44 | 38 | 33 | 30 | 27 | 24 | 21 | 20 | 19 | 17 | 14 | 12 | 0 |
| .50 | 13 | 18 | 24 | 35 | 40 | 52 | 80 | 142 | 262 | 410 | 504 | 506 | 441 | 269 | 153 | 98 | 73 | 61 | 53 | 45 | 39 | 34 | 30 | 27 | 24 | 22 | 20 | 19 | 17 | 15 | 12 | 0 |
| .75 | 10 | 13 | 17 | 23 | 26 | 30 | 34 | 40 | 55 | 86 | 150 | 247 | 349 | 438 | 360 | 240 | 151 | 101 | 75 | 57 | 47 | 39 | 33 | 29 | 26 | 23 | 21 | 20 | 18 | 15 | 12 | 0 |
| 1.0 | 9 | 11 | 14 | 19 | 21 | 24 | 26 | 30 | 35 | 44 | 62 | 101 | 167 | 337 | 413 | 353 | 245 | 157 | 104 | 68 | 53 | 42 | 35 | 31 | 28 | 24 | 22 | 20 | 18 | 16 | 12 | 0 |
| 1.5 | 6 | 8 | 10 | 13 | 14 | 15 | 17 | 19 | 21 | 23 | 26 | 30 | 37 | 73 | 166 | 288 | 356 | 337 | 264 | 154 | 91 | 57 | 42 | 35 | 30 | 27 | 24 | 22 | 19 | 17 | 13 | 3 |
| 2.0 | 4 | 5 | 7 | 8 | 9 | 10 | 10 | 11 | 12 | 14 | 15 | 16 | 18 | 23 | 31 | 55 | 114 | 206 | 291 | 324 | 239 | 125 | 63 | 44 | 35 | 31 | 28 | 24 | 20 | 18 | 14 | 9 |
| 2.5 | 3 | 4 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 11 | 12 | 15 | 18 | 22 | 32 | 58 | 111 | 227 | 298 | 246 | 122 | 63 | 43 | 35 | 31 | 27 | 22 | 19 | 15 | 11 |
| 3.0 | 1 | 2 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 11 | 13 | 16 | 19 | 27 | 59 | 138 | 280 | 248 | 137 | 70 | 46 | 36 | 31 | 25 | 21 | 16 | 11 |
| * * * TC =0.2 HR * * * IA/P = 0.30 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 0.0 | 0 | 0 | 0 | 39 | 180 | 545 | 697 | 497 | 276 | 198 | 158 | 130 | 110 | 93 | 81 | 73 | 67 | 61 | 56 | 49 | 46 | 43 | 39 | 35 | 32 | 30 | 29 | 27 | 24 | 21 | 19 | 0 |
| .10 | 0 | 0 | 0 | 2 | 27 | 129 | 407 | 600 | 532 | 361 | 252 | 190 | 150 | 108 | 90 | 79 | 71 | 65 | 59 | 52 | 48 | 44 | 41 | 36 | 32 | 31 | 29 | 28 | 25 | 21 | 19 | 0 |
| .20 | 0 | 0 | 0 | 2 | 19 | 92 | 302 | 501 | 521 | 415 | 306 | 228 | 176 | 119 | 95 | 82 | 73 | 67 | 61 | 53 | 48 | 45 | 41 | 37 | 33 | 31 | 29 | 28 | 25 | 21 | 19 | 0 |
| .30 | 0 | 0 | 0 | 0 | 1 | 13 | 66 | 223 | 408 | 484 | 438 | 350 | 269 | 163 | 114 | 93 | 80 | 72 | 65 | 57 | 51 | 46 | 42 | 38 | 34 | 31 | 30 | 28 | 25 | 22 | 19 | 0 |
| .40 | 0 | 0 | 0 | 0 | 1 | 9 | 47 | 164 | 327 | 431 | 436 | 379 | 306 | 189 | 127 | 98 | 83 | 74 | 67 | 58 | 52 | 47 | 43 | 38 | 34 | 31 | 30 | 28 | 25 | 22 | 19 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 6 | 33 | 120 | 258 | 374 | 415 | 391 | 271 | 173 | 121 | 95 | 81 | 72 | 62 | 55 | 48 | 44 | 40 | 35 | 32 | 30 | 29 | 26 | 22 | 19 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 13 | 50 | 126 | 221 | 302 | 345 | 323 | 240 | 167 | 121 | 96 | 81 | 68 | 59 | 50 | 45 | 41 | 37 | 33 | 31 | 29 | 26 | 23 | 19 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 24 | 69 | 139 | 285 | 331 | 280 | 204 | 145 | 109 | 82 | 68 | 56 | 48 | 43 | 39 | 35 | 32 | 30 | 27 | 24 | 19 | 0 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 16 | 79 | 186 | 271 | 288 | 247 | 165 | 110 | 76 | 58 | 49 | 44 | 40 | 35 | 32 | 29 | 26 | 20 | 5 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 24 | 80 | 163 | 235 | 262 | 202 | 123 | 76 | 58 | 49 | 43 | 39 | 35 | 30 | 27 | 21 | 13 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 28 | 77 | 179 | 242 | 207 | 120 | 75 | 57 | 48 | 43 | 39 | 32 | 29 | 22 | 17 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 30 | 101 | 207 | 227 | 130 | 80 | 59 | 49 | 44 | 35 | 30 | 24 | 18 |
| * * * TC =0.2 HR * * * IA/P = 0.50 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 0.0 | 0 | 0 | 0 | 0 | 7 | 98 | 371 | 322 | 221 | 182 | 158 | 137 | 120 | 104 | 94 | 86 | 80 | 74 | 69 | 62 | 60 | 57 | 52 | 47 | 44 | 42 | 40 | 39 | 35 | 30 | 28 | 0 |
| .10 | 0 | 0 | 0 | 0 | 4 | 67 | 270 | 305 | 249 | 204 | 174 | 149 | 130 | 108 | 97 | 88 | 82 | 76 | 71 | 64 | 60 | 57 | 53 | 48 | 44 | 42 | 41 | 39 | 35 | 30 | 28 | 0 |
| .20 | 0 | 0 | 0 | 0 | 0 | 3 | 45 | 195 | 268 | 255 | 221 | 189 | 163 | 125 | 106 | 95 | 87 | 80 | 75 | 67 | 62 | 58 | 54 | 49 | 45 | 43 | 41 | 39 | 35 | 31 | 28 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 2 | 31 | 140 | 226 | 245 | 229 | 203 | 176 | 134 | 111 | 98 | 89 | 82 | 76 | 68 | 62 | 59 | 55 | 50 | 45 | 43 | 41 | 39 | 36 | 31 | 28 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 1 | 21 | 101 | 184 | 225 | 228 | 211 | 193 | 144 | 117 | 101 | 91 | 84 | 78 | 69 | 63 | 59 | 55 | 50 | 45 | 43 | 41 | 40 | 36 | 31 | 28 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 14 | 72 | 146 | 199 | 218 | 213 | 175 | 137 | 113 | 99 | 89 | 82 | 73 | 66 | 60 | 56 | 52 | 47 | 43 | 42 | 40 | 36 | 32 | 28 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 5 | 28 | 71 | 121 | 162 | 186 | 193 | 161 | 133 | 112 | 98 | 88 | 78 | 70 | 62 | 57 | 53 | 48 | 44 | 42 | 41 | 37 | 33 | 28 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 13 | 38 | 77 | 154 | 186 | 174 | 147 | 122 | 105 | 89 | 78 | 68 | 60 | 56 | 51 | 46 | 43 | 42 | 38 | 34 | 28 | 0 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 26 | 71 | 129 | 163 | 168 | 150 | 120 | 98 | 80 | 67 | 60 | 55 | 51 | 46 | 43 | 40 | 36 | 28 | 4 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 5 | 25 | 65 | 112 | 146 | 157 | 134 | 103 | 79 | 67 | 60 | 55 | 50 | 46 | 41 | 38 | 29 | 14 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 26 | 60 | 117 | 148 | 136 | 101 | 79 | 66 | 59 | 54 | 50 | 43 | 39 | 31 | 24 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 40 | 90 | 142 | 130 | 99 | 78 | 66 | 59 | 54 | 45 | 41 | 33 | 26 |

RAINFALL TYPE = II  * * * TC = 0.2 HR * * *  SHEET 2 OF 10



38 OF 53

# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 39 of 53

Chkd. By EC Date 3/4/96 SURFACE WATER MANAGEMENT Proj. No. 95-284

HYDRAULIC CALCULATIONS



Recyclable and made from recycled paper.

# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 40 of 53

Chkd. By EC Date 3/4/96 SURFACE WATER MANAGEMENT Proj. No. 95-284

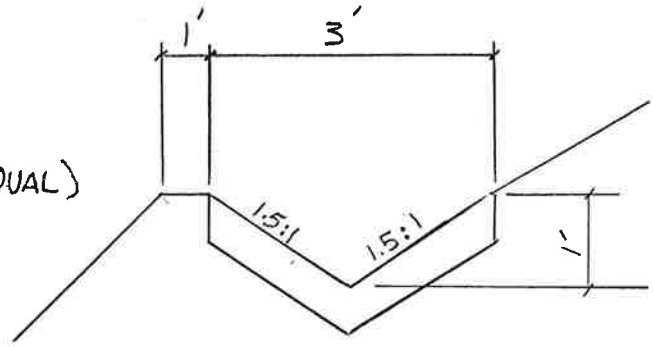
BENCH DRAIN DESIGN:

$Q_{25} = 4$ CFS (MAX. FLOW)

$S = 0.03$

$n = 0.032$ (ARMOFLEX OR EQUAL)

$Z = 1.5$




BENCH DRAIN CROSS-SECTION

$D_h = 0.82'$

$V = 3.9$ FPS

SEE NEXT PAGE



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Triangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name:

Comment: BENCH DESIGN

Solve For Depth

Given Input Data:

| | |
|---|---|
| Left Side Slope.. | 1.50:1 (H:V) |
| Right Side Slope. | 1.50:1 (H:V) |
| Manning's n..... | 0.032 |
| Channel Slope.... | 0.0300 ft/ft |
| Discharge..... | 4.00 cfs |

Computed Results:

| | |
|---|---|
| Depth..... | 0.82 ft |
| Velocity..... | 3.94 fps |
| Flow Area..... | 1.02 sf |
| Flow Top Width... | 2.47 ft |
| Wetted Perimeter. | 2.97 ft |
| Critical Depth... | 0.85 ft |
| Critical Slope... | 0.0254 ft/ft |
| Froude Number.... | 1.08 (flow is Supercritical) |

Open Channel Flow Module, Version 3.41 (c) 1991
Haestad Methods, Inc. * 37 Brookside Rd * Waterbury, Ct 06708

# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 42 of 53

Chkd. By EC Date 3/4/96 SURFACE WATER MANAGEMENT Proj. No. 95-284

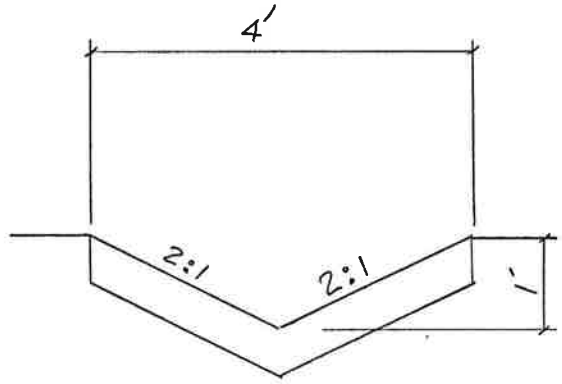
DOWNDRAIN DESIGN:

$Q_{25}$ = 4CFS (MAX. FLOW)

S = 0.30

n = 0.032 (ARMORFLEX OR EQUAL)

z = 2




DOWNDRAIN CROSS SECTION

$D_n$ = 0.47'

V = 9.0 FPS

(SEE NEXT PAGE)



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Triangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name:

Comment: DOWNDRAIN DESIGN

Solve For Depth

Given Input Data:

| | |
|---|---|
| Left Side Slope.. | 2.00:1 (H:V) |
| Right Side Slope. | 2.00:1 (H:V) |
| Manning's n..... | 0.032 |
| Channel Slope.... | 0.3000 ft/ft |
| Discharge..... | 4.00 cfs |

Computed Results:

| | |
|---|---|
| Depth..... | 0.47 ft |
| Velocity..... | 9.01 fps |
| Flow Area..... | 0.44 sf |
| Flow Top Width... | 1.88 ft |
| Wetted Perimeter. | 2.11 ft |
| Critical Depth... | 0.76 ft |
| Critical Slope... | 0.0240 ft/ft |
| Froude Number.... | 3.27 (flow is Supercritical) |

Open Channel Flow Module, Version 3.41 (c) 1991
Haestad Methods, Inc. * 37 Brookside Rd * Waterbury, Ct 06708

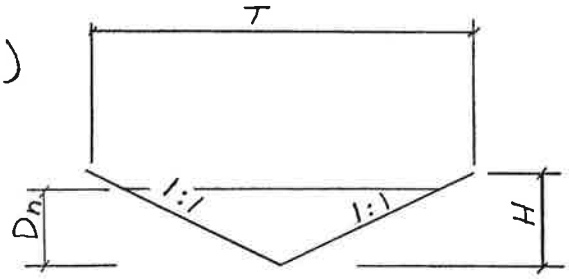
# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 44 of 53

Chkd. By EC Date 3/4/96 SURFACE WATER MANAGEMENT Proj. No. 95-284

LINE A DESIGN:

REACH A-2 (SEE HYDROLOGY MAP)




$Q_{25} = 6$ CFS

$S = 0.005$

$n = 0.015$

$z = 1$

$D_n = 1.22'$, (SEE PAGE 45)
USE: $H = 1.5'$, $T = 3.0'$

REACH A-4

$Q_{25} = 14$ CFS

$S = 0.005$

$n = 0.015$

$z = 1$

$D_n = 1.68'$ (SEE PAGE 46)
USE: $H = 2.0'$, $T = 4.0'$

REACH A-7

$Q_{25} = 21$ CFS

$S = 0.005$

$n = 0.015$

$z = 1$

$D_n = 1.96'$ (SEE PAGE 47)
USE: $H = 2.5'$, $T = 5.0'$



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# Triangular Channel Analysis & Design
## Open Channel - Uniform flow

Worksheet Name:

Comment: REACH A-2

Solve For Depth

Given Input Data:

| | |
|---|---|
| Left Side Slope.. | 1.00:1 (H:V) |
| Right Side Slope. | 1.00:1 (H:V) |
| Manning's n..... | 0.015 |
| Channel Slope.... | 0.0050 ft/ft |
| Discharge..... | 6.00 cfs |

Computed Results:

| | |
|---|---|
| Depth..... | 1.22 ft |
| Velocity..... | 4.01 fps |
| Flow Area..... | 1.50 sf |
| Flow Top Width... | 2.45 ft |
| Wetted Perimeter. | 3.46 ft |
| Critical Depth... | 1.17 ft |
| Critical Slope... | 0.0062 ft/ft |
| Froude Number.... | 0.90 (flow is Subcritical) |

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Triangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name:

Comment: REACH A-4

Solve For Depth

Given Input Data:

| | |
|---|---|
| Left Side Slope.. | 1.00:1 (H:V) |
| Right Side Slope. | 1.00:1 (H:V) |
| Manning's n..... | 0.015 |
| Channel Slope.... | 0.0050 ft/ft |
| Discharge..... | 14.00 cfs |

Computed Results:

| | |
|---|---|
| Depth..... | 1.68 ft |
| Velocity..... | 4.95 fps |
| Flow Area..... | 2.83 sf |
| Flow Top Width... | 3.36 ft |
| Wetted Perimeter. | 4.76 ft |
| Critical Depth... | 1.65 ft |
| Critical Slope... | 0.0056 ft/ft |
| Froude Number.... | 0.95 (flow is Subcritical) |

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Triangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name:

Comment: REACH A-7

Solve For Depth

Given Input Data:

| | |
|---|---|
| Left Side Slope.. | 1.00:1 (H:V) |
| Right Side Slope. | 1.00:1 (H:V) |
| Manning's n..... | 0.015 |
| Channel Slope.... | 0.0050 ft/ft |
| Discharge..... | 21.00 cfs |

Computed Results:

| | |
|---|---|
| Depth..... | 1.96 ft |
| Velocity..... | 5.48 fps |
| Flow Area..... | 3.83 sf |
| Flow Top Width... | 3.91 ft |
| Wetted Perimeter. | 5.54 ft |
| Critical Depth... | 1.94 ft |
| Critical Slope... | 0.0053 ft/ft |
| Froude Number.... | 0.98 (flow is Subcritical) |

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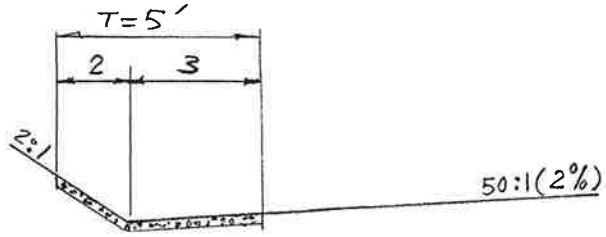
# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 48 of 53

Chkd. By GC Date 3/4/96 SURFACE WATER MANAGEMENT Proj. No. 95-284

LINE B DESIGN:




REACH B-3

$Q_{25} = 12$ CFS

$S = 0.004$
$n = 0.015$
$Z_\ell$ = LEFT SIDE SLOPE = 2
$Z_R$ = RIGHT SIDE SLOPE = 50

$D_n = 0.45'$ (SEE PAGE 49)
USE: $H = 1.0'$, $T = 5.0'$

REACH B-5

$Q = 21$ CFS

$S = 0.004$
$n = 0.015$
$Z_\ell$ = LEFT SIDE SLOPE = 2
$Z_R$ = RIGHT SIDE SLOPE = 50

$D_n = 0.55'$ (SEE PAGE 50)
USE: $H = 1.0'$, $T = 5.0'$

REACH B-7

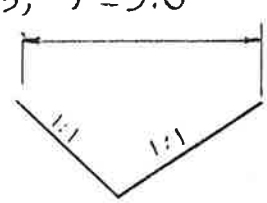
$Q_{25} = 27$ CFS

$S = 0.01$

$n = 0.015$

$Z = 1$

$D_n = 1.89'$ (SEE PAGE 51)
USE: $H = 2.5'$, $T = 5.0'$



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Triangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name:

Comment: REACH B-3

Solve For Depth

Given Input Data:

| | |
|---|---|
| Left Side Slope.. | 2.00:1 (H:V) |
| Right Side Slope. | 50.00:1 (H:V) |
| Manning's n..... | 0.015 |
| Channel Slope.... | 0.0040 ft/ft |
| Discharge..... | 12.00 cfs |

Computed Results:

| | |
|---|---|
| Depth..... | 0.45 ft |
| Velocity..... | 2.30 fps |
| Flow Area..... | 5.21 sf |
| Flow Top Width... | 23.28 ft |
| Wetted Perimeter. | 23.39 ft |
| Critical Depth... | 0.42 ft |
| Critical Slope... | 0.0056 ft/ft |
| Froude Number.... | 0.86 (flow is Subcritical) |

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Triangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name:

Comment: REACH B-5

Solve For Depth

Given Input Data:

| | |
|---|---|
| Left Side Slope.. | 2.00:1 (H:V) |
| Right Side Slope. | 50.00:1 (H:V) |
| Manning's n..... | 0.015 |
| Channel Slope.... | 0.0040 ft/ft |
| Discharge..... | 21.00 cfs |

Computed Results:

| | |
|---|---|
| Depth..... | 0.55 ft |
| Velocity..... | 2.65 fps |
| Flow Area..... | 7.93 sf |
| Flow Top Width... | 28.72 ft |
| Wetted Perimeter. | 28.85 ft |
| Critical Depth... | 0.53 ft |
| Critical Slope... | 0.0052 ft/ft |
| Froude Number.... | 0.89 (flow is Subcritical) |

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Triangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name:

Comment: LINE B (REACH B-7)

Solve For Depth

Given Input Data:

| | |
|---|---|
| Left Side Slope.. | 1.00:1 (H:V) |
| Right Side Slope. | 1.00:1 (H:V) |
| Manning's n..... | 0.015 |
| Channel Slope.... | 0.0100 ft/ft |
| Discharge..... | 27.00 cfs |

Computed Results:

| | |
|---|---|
| Depth..... | 1.89 ft |
| Velocity..... | 7.57 fps |
| Flow Area..... | 3.57 sf |
| Flow Top Width... | 3.78 ft |
| Wetted Perimeter. | 5.34 ft |
| Critical Depth... | 2.14 ft |
| Critical Slope... | 0.0051 ft/ft |
| Froude Number.... | 1.37 (flow is Supercritical) |

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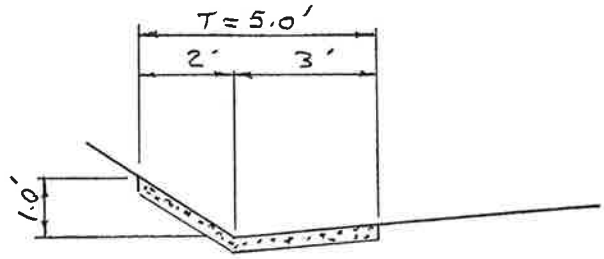
# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 52 of 53

Chkd. By EC Date 3/4/96 SURFACE WATER MANAGEMENT Proj. No. 95-284

LINE C DESIGN:




REACH C-3

$Q_{25} = 10$ CFS

$S = 0.004$

$n = 0.025$

$Z_\ell = 2$

$Z_R = 50$

$D_n = 0.42'$ (SEE PAGE 53)

USE: $H = 1.0'$, $T = 5.0'$



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Triangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name:

Comment: REACH C-3

Solve For Depth

Given Input Data:

| | |
|---|---|
| Left Side Slope.. | 50.00:1 (H:V) |
| Right Side Slope. | 2.00:1 (H:V) |
| Manning's n..... | 0.015 |
| Channel Slope.... | 0.0040 ft/ft |
| Discharge..... | 10.00 cfs |

Computed Results:

| | |
|---|---|
| Depth..... | 0.42 ft |
| Velocity..... | 2.20 fps |
| Flow Area..... | 4.55 sf |
| Flow Top Width... | 21.74 ft |
| Wetted Perimeter. | 21.84 ft |
| Critical Depth... | 0.39 ft |
| Critical Slope... | 0.0057 ft/ft |
| Froude Number.... | 0.85 (flow is Subcritical) |

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# C.11 EROSION CONTROL

# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 1 of 16
Chkd. By EC Date 3/4/96 SURFACE COVER EROSION CALCS Proj. No. 95-284

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| REFERENCE MATERIALS | 9 |
| SURFACE COVER EROSION MAP (FIGURE 2) | 16 |




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# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 2 of 16

Chkd. By EC Date 3/4/96 SURFACE COVER EROSION CALCS Proj. No. 95-284

PURPOSE:

CALCULATE THE RATE OF SOIL COVER EROSION OF THE LANDFILL SURFACE USING THE UNIVERSAL SOIL LOSS EQUATION (USLE).

REFERENCES:

- EROSION AND SEDIMENT CONTROL HANDBOOK COPYRIGHT 1986 BY McGRAW HILL, INC.
- EPA TECHNICAL GUIDANCE DOCUMENT: FINAL COVERS ON HAZARDOUS WASTE LANDFILLS AND SURFACE IMPOUNDMENTS. EPA/530-SW-89-047, DATED JULY 1989.



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# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 3 of 16

Chkd. By EC Date 3/4/96 SURFACE COVER EROSION CALCS Proj. No. 95-284

SURFACE COVER EROSION CALCULATIONS:

USING THE UNIVERSAL SOIL LOSS EQUATION TO ESTIMATE SOIL LOSS, THE GENERAL FORM OF THE UNIVERSAL SOIL LOSS EQUATION IS:

$$A = R \times K \times C \times P \times LS$$

WHERE: A = SOIL LOSS, TONS/ACRE-YEAR

R = RAINFALL EROSION INDEX (RAINFALL AND RUNOFF FACTOR).

K = SOIL ERODIBILITY FACTOR, (TONS/ACRE PER UNIT OF "R").

C = VEGETATIVE COVER FACTOR, (DIMENSIONLESS).

P = EROSION CONTROL PRACTICE FACTOR, (DIMENSIONLESS).

LS = SLOPE LENGTH AND STEEPNESS FACTOR, (DIMENSIONLESS).



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# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 4. of 16
Chkd. By EC Date 3/4/96 SURFACE COVER EROSION CALCULATIONS Proj. No. 95-284

RAINFALL EROSION INDEX, R

THE RAINFALL EROSION INDEX "R" IS A MEASURE OF THE EROSIVE FORCE AND INTENSITY OF RAIN IN A NORMAL YEAR. THE "R" VALUE FOR THE PROJECT SITE IS APPROXIMATELY 20. (SEE FIGURE 1 ON PAGE 10).

$\boxed{R = 20}$

SOIL ERODIBILITY FACTOR, "K"

THE SOIL ERODIBILITY FACTOR "K" IS A MEASURE OF THE SUSCEPTIBILITY OF SOIL PARTICLES TO DETECHMENT AND TRANSPORT BY RAINFALL AND RUNOFF. THE DETERMINATION OF THE "K" VALUE SHOULD BE BASED ON THE SOIL TEXTURE. THE PROPOSED COVER SOIL TEXTURE WOULD BE COMPRISED OF 100 PERCENT GRAVEL (U.S. STANDARD SIEVE OPENING 3" – U.S. STANDARD SIEVE OPENING 3/4"). THIS IS OUTSIDE THE RANGE OF THE TRIANGULAR NOMOGRAPH ON PAGE 13. CONSERVATIVELY USE AN UNADJUSTED K VALUE OF 0.10.



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# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 5 of 16
Chkd. By EC Date 3/4/96 SURFACE COVER EROSION CALCULATIONS Proj. No. 95-284

THE TRIANGULAR NOMOGRAPH ASSUMES 2% ORGANIC MATTER. ASSUMING THE TOPSOIL CONTAINS NO ORGANIC MATTER, ADJUST THE K VALUE PER TABLE 5.3.01 ON PAGE 14. K = 0.10 + 0.06 = 0.16. THE EROSION AND SEDIMENT CONTROL HANDBOOK REQUIRES FURTHER ADJUSTMENT BASED ON THE ROCK CONTENT. THE EROSION AND SEDIMENT CONTROL HANDBOOK DEFINES ROCK CONTENT AS THE PERCENT (BY VOLUME) OF SOIL PARTICLES GREATER THAN 2 mm. ASSUMING THE ROCK CONTENT IN THE RANGE OF 60-75 %, THE ADJUSTED "K" VALUE WOULD BE 0.02 (SEE TABLE 5.3b ON PAGE 14).

STRUCTURE CORRECTION FACTOR = -0.03 (SEE TABLE 5.3C ON PAGE 14).

PERMEABILITY CORRECTION FACTOR = +0.03 (SEE TABLE 5.3D ON PAGE 14).

K = 0.02 - 0.03 + 0.03 = 0.02

$\boxed{K = 0.02}$

## COVER FACTOR "C"

THE COVER FACTOR "C" IS THE RATIO OF SOIL LOSS FROM LAND CROPPED UNDER SPECIFIED CONDITIONS TO THE CORRESPONDING LOSS FROM CLEAN TILLED, CONTINUOUS FALLOW LAND. THE LANDFILL WILL NOT BE VEGETATED.



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FROM TABLE 1 ON PAGE 15, C = 0.5 (CONTINUOUS BARE SOIL SURFACE - UNTILLED). SINCE THE LANDFILL WILL BE COVERED WITH FAIRLY LARGE MATERIAL ($D_{50}$ = 2") USE A "C" VALUE OF 0.1

$$\boxed{C = 0.1}$$

EROSION CONTROL PRACTICE FACTOR "P"

THE EROSION AND SEDIMENT CONTROL HANDBOOK DEFINES THE EROSION CONTROL FACTOR AS THE RATIO OF SOIL LOSS WITH A GIVEN SURFACE CONDITION TO SOIL LOSS WITH UP-AND-DOWN-HILL PLOWING. IN AGRICULTURAL USES OF THE USLE, "P" IS USED TO DESCRIBE PLOWING AND TILLAGE PRACTICES. IN CONSTRUCTION SITE APPLICATIONS, "P" REFLECTS THE ROUGHENING OF THE SOIL SURFACE BY TRACTOR TREAD OR BY ROUGH GRADING. USING TABLE 5.7 ON PAGE 16, THE "P" FACTOR WILL BE 0.9

$$\boxed{P = 0.9}$$

# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 7 of 16
Chkd. By EC Date 3/4/96 SURFACE COVER EROSION CALCULATIONS Proj. No. 95-284

LENGTH-SLOPE FACTOR, "LS"

THE "LS" FACTOR DESCRIBES THE COMBINED EFFECT OF SLOPE LENGTH AND SLOPE GRADIENT. THE LANDFILL SIDE SLOPES HAVE SLOPE RATIOS OF 3:1 AND 2:1. THE TOP DECK GRADE IS FIVE (5) PERCENT. THE SIDE SLOPE LENGTH IS THE DISTANCE OF OVERLAND FLOW TO THE NEAREST DIVERSION CHANNEL.

See Fig. 2 (pg. 16) and Table 5.5 (pg. 15) for longest slope lengths applicable to each slope and their associated LS values

| SLOPE (ft/ft) | LONGEST SLOPE LENGTH | | LENGTH USED (ft) | LS VALUE FROM TABLE 5.5 (PAGE 15) |
|---|---|---|---|---|
| | MAP DISTANCE (ft) (PAGE 16) | CORR. DISTANCE (ft) | | |
| 2:1 | 75 | 84 | 90 | 16.91 |
| 3:1 | 120 | 126 | 150 | 11.55 |
| 5% | 90 | 90 | 90 | 0.51 |

$A = R \times K \times C \times P \times LS$
$= 20 \times 0.02 \times 0.1 \times 0.90 LS = 0.04 LS$

SIDE SLOPES (2:1), $A = 0.04 LS = 0.04 \times 16.91 = 0.68$ TONS/AC/YR

SIDE SLOPES (3:1), $A = 0.04 LS = 0.04 \times 11.55 = 0.46$ TONS/AC/YR

TOP DECK (5%), $A = 0.04 LS = 0.04 \times 0.51 = 0.02$ TONS/AC/YR



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# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 8 of 16
Chkd. By EC Date 3/4/96 SURFACE COVER EROSION CALCULATIONS Proj. No. 95-284

CONCLUSIONS:

THE MAXIMUM AMOUNT OF SOIL EROSION ON SIDE SLOPES IS 0.68 TON/AC/YR (2:1 SIDE SLOPE).

EPA TECHNICAL GUIDANCE DOCUMENT (EPA, 1989) REQUIRES THAT EROSION RATE SHALL NOT EXCEED 2 TONS/AC/YR. THE MAXIMUM CALCULATED EROSION RATE FOR THE SITE IS 0.68 TONS/AC/YR, WHICH IS BELOW THE EROSION CRITERION SUGGESTED BY THE EPA.



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# ENVIRONMENTAL SOLUTIONS, INC.

By M.V. Date 2-23-96 Subject BEATTY LANDFILL Sheet No. 9 of 16

Chkd. By EC Date 3/4/96 SURFACE COVER EROSION CALCS Proj. No. 95-284

REFERENCE MATERIALS



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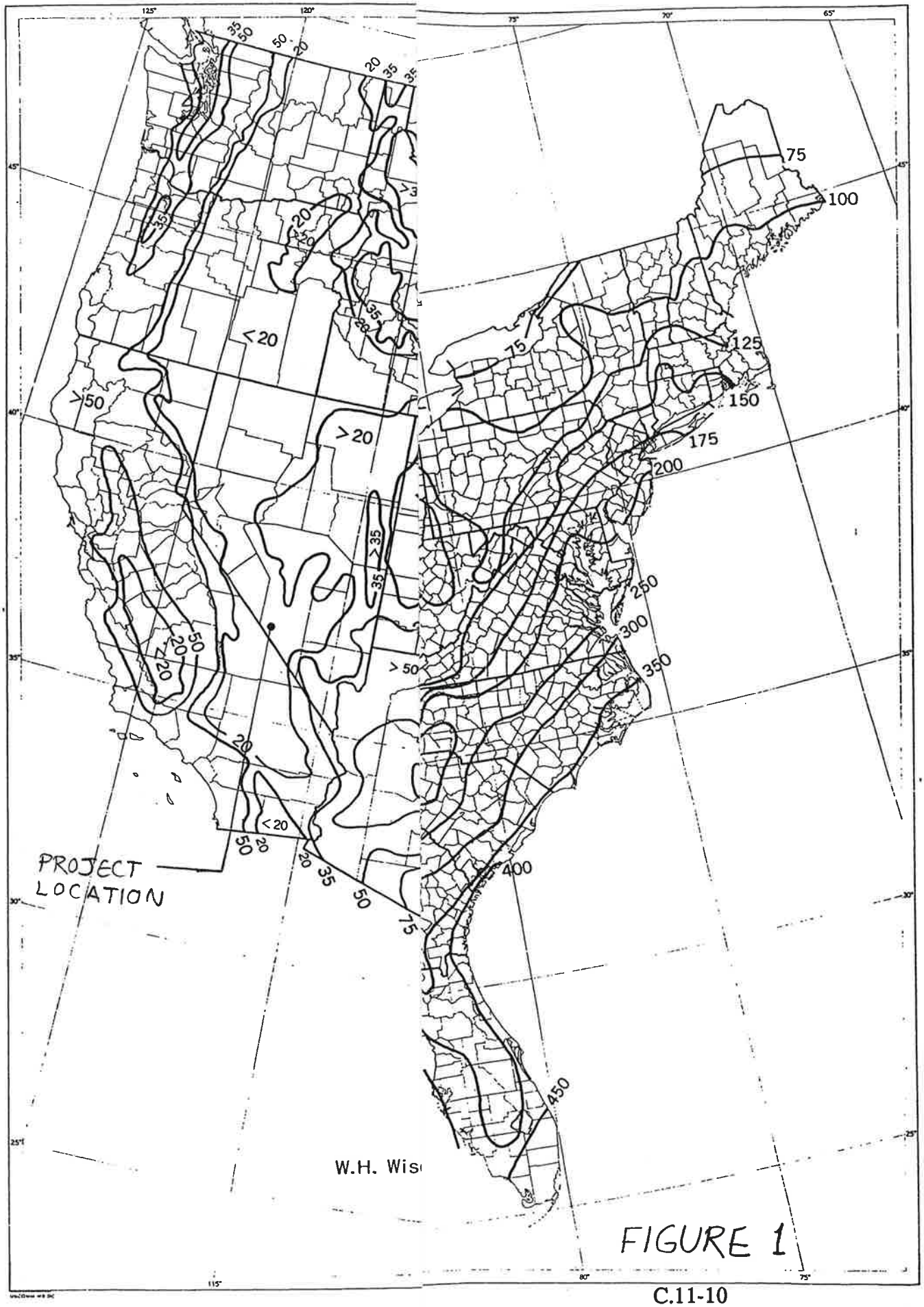
PROJECT LOCATION
W.H. Wis
FIGURE 1
<20
>20
>50
>35
75
100
125
150
175
200
250
300
350
400
450

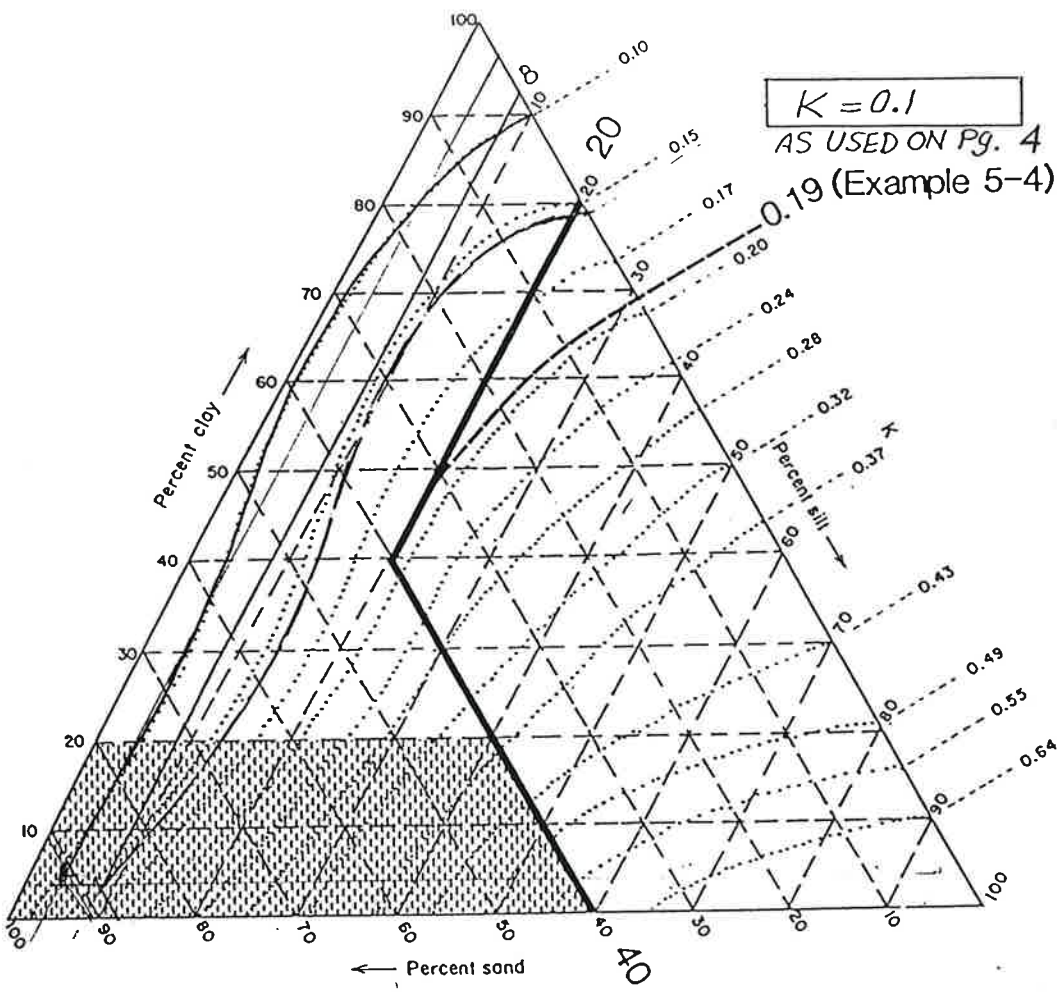


**Fig. 5.6** Triangular nomograph for estimating $K$ value. (6) See Table 5.3 for adjustments to $K$ value under certain conditions.

## EXAMPLE 5.4

**Given:** A soil with the following particle size distribution.

| Component | Size, mm | Fraction, % |
|---|---|---|
| Sand | 2.0–0.1 | 30 |
| Very fine sand | 0.1–0.05 | 10 |
| Silt | 0.05–0.002 | 20 |
| Clay | Less than 0.002 | 40 |

**Find:** Texture and $K$ value.

***Solution:*** Entering Fig. 5.1 with 40 percent total sand and 20 percent silt, the texture is found to be on the border between clay and clay loam. Entering Fig. 5.6 with the same percents (see bold lines), the $K$ value is found to be 0.19.

Table 5.3 describes adjustments to the $K$ factor. Adjustment 1 is a correction for very

**TABLE 5.3** Adjustments to $K$ Value (6)

1. For soils with greater than 15% *very fine sand* (vfs) make the following adjustment:
   - *a.* If texture is *coarser than loam* (shaded area in Fig. 5.6): Subtract 5% from the % vfs and add the difference to the silt content. Consider the remaining 5% vfs to be part of the % total sand.
   - *b.* If texture is *loam and finer* (outside shaded area on Fig. 5.6), subtract 10% from the % vfs and add the difference to the silt content. Consider the remaining 10% vfs to be part of the % total sand.
   - *c.* Find the $K$ value by using the adjusted sand and silt contents.
2. The nomograph assumes 2% organic matter, structure other than granular, and 0–15% rock content. The correction factors are as follows.
   - *a.* Organic matter: Add or subtract correction factor to $K$ value as indicated in the following table.

| $K$ value | Correction factor when percent organic matter is 0 | 1 | 2 | 3 | 4 |
|---|---|---|---|---|---|
| Greater than 0.40 | +0.14 | +0.07 | 0 | −0.07 | −0.14 |
| 0.20–0.40 | +0.10 | +0.05 | 0 | −0.05 | −0.10 |
| Less than 0.20 | +0.06 | +0.03 | 0 | −0.03 | −0.06 |

   - *b.* Rock content: Rock content is defined as the percent (by volume) of soil particles greater than 2 mm.

| Unadjusted $K$ value from Fig. 5.6 | $K$ values adjusted for rock content as follows 15–35% | 35–60% | 60–75% |
|---|---|---|---|
| 0.10 | 0.05 | 0.05 | 0.02 |
| 0.15 | 0.10 | 0.05 | 0.02 |
| 0.17 | 0.10 | 0.05 | 0.02 |
| 0.20 | 0.10 | 0.05 | 0.02 |
| 0.24 | 0.15 | 0.10 | 0.05 |
| 0.28 | 0.15 | 0.10 | 0.05 |
| 0.32 | 0.17 | 0.10 | 0.05 |
| 0.37 | 0.20 | 0.10 | 0.05 |
| 0.43 | 0.24 | 0.15 | 0.10 |
| 0.49 | 0.28 | 0.15 | 0.10 |
| 0.55 | 0.32 | 0.17 | 0.10 |
| 0.64 | 0.37 | 0.20 | 0.15 |

0.16

Add or subtract the correction factors at the right to the $K$ value to correct for the following structures and permeabilities.

   - *c.* Structure:

| | |
|---|---|
| Very fine granular | −0.09 |
| Fine granular | −0.06 |
| Moderate or coarse granular | −0.03 |

   - *d.* Permeability:

| | |
|---|---|
| Compact soil or pH greater than 9.0 | +0.03 |
| Many medium or coarse pores | −0.03 |

EROSION AND SEDIMENT CONTROL HANDBOOK
McGRAW HILL, INC. 1986

## Soil Loss Equation "C" Factors for California Cropping Systems

It is not intended that the tables following include all cropping systems in California. The cropping systems and "C" factors listed are to provide examples for helping conservationists make accurate estimates of "C" values for cropping systems within their work areas.

Table 1, below, provides criteria that should enable most conservationists to make "Average Annual C" estimates for a cropping system with fair accuracy once they know the tillage and crop residue practices for the system and relate these to the periods of precipitation causing erosion.

TABLE 1

| Soil Management Practice | Estimated "C" |
|---|---|
| * 1. Continuous clean tilled fallow (Research plots) | 1.00 |
| 2. Continuous tilled fallow, 1000 lb. straw per acre maintained on soil surface | 0.50 |
| 3. Continuous bare soil surface - untilled | 0.50 |
| 4. Orchard Cover Crop - spring disked | 0.25 |
| 5. Orchard strip Cover Crop, untilled, mowed | 0.10 |
| 6. Continuous annual grass or legume pasture or hay (100% cover) | 0.01 |
| 7. Continuous perennial grass (100% cover) | 0.003 |

* Item 1 represents a condition induced by continuous loose fallow over a period of several years. Thus, for practical purposes, Average Annual "C" on agricultural land would always be less than 1.00.

In general, erosion from rainfall in California occurs during the 6 month period between November 1 and April 30. Thus "C" values in the state relate almost entirely to condition of the soil surface during these winter months. Bare tilled soil surface will give "C" values approaching 1.0. Non-tillage of bare surface, or light surface mulches on a tilled surface will reduce erosion and "C" values to about half (0.5). A complete cover of annual grasses and weeds during the winter months will reduce soil losses to about one-tenth on tilled cropland as compared to clean tilled fallow, giving a "C" value of 0.1.

Excepting non-tilled orchards with cover crop, California cropping systems will usually have average annual "C" values between 0.1 and 0.5 with high crop yield stubble mulch systems producing low values and clean-tilled low crop yield systems the higher values.



USDA - SCS
GUIDES FOR EROSION & SEDIMENT CONTROL, DATED JAN. 1991

TABLE 5.7 *P* Factors for Construction Sites (Adapted from Ref. 15)

| Surface condition | *P* value |
|---|---|
| Compacted and smooth | 1.3 |
| Trackwalked along contour* | 1.2 |
| Trackwalked up and down slope† | 0.9 |
| Punched straw | 0.9 |
| Rough, irregular cut | 0.9 |
| Loose to 12-in (30-cm) depth | 0.8 |

*Tread marks oriented up and down slope.

†Tread marks oriented parallel to contours, as in Figs. 6.9 and 6.10.

*P* values appropriate for construction sites are listed in Table 5.7.

- A surface that is compacted and smoothed by grading equipment is highly susceptible to sheet runoff and is assigned a *P* value of 1.3.
- Trackwalking is given a value of 1.2 if the vehicle traverses along the contour. The *P* value is relatively high because the depressions left by cross-slope tracking resemble up-and-down furrows and worsen runoff conditions.
- Trackwalking up and down slope reduces *P* to 0.9. The tread marks act as slope benches; they reduce runoff velocity and trap soil particles (see Fig. 6.10).
- Punched straw is assigned a *P* value of 0.9 because the action of punching the straw into the soil roughens the surface and creates a trackwalking effect.
- When the soil surface is disked or otherwise loosened to a depth of 1 ft, a slightly lower *P* value of 0.8 may be used. This condition is unlikely to occur on a construction site because compaction, not loosening, is required when fill slopes are constructed.

Clearly, changing the surface condition does not provide much direct reduction in soil loss; all the *P* values are close to 1.0. However, roughening the soil surface is essential before seeding because it greatly increases plant establishment (see Chap. 6) and thus also reduces the *C* factor. Vegetation, mulch, slope length, and gradient have far more significant effects on the erosion process and provide greater opportunities to reduce soil loss.

## 5.2g Combined Effects of LS, *C*, and *P*

Of the five factors in the USLE, the *R*, LS, and *C* factors have the widest range. Although *R* for a site is constant and *K* is essentially a constant, slope length and gradient, cover, and, to a limited extent, surface condition can be manipulated. Slope length and vegetative cover are the most effective and easily implemented measures.

Table 5.8 compares the effect on the soil loss estimates of varying LS, *C*, and *P*. For example, a building pad with a 1 percent slope, smooth surface, and no cover has a fractional soil loss potential. A 2:1 slope, common between terraced

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**TABLE 5.5** LS Values* (10)

| Slope ratio | Slope gradient $s$, % | LS values for following slope lengths $l$, ft (m) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 10 (3.0) | 20 (6.1) | 30 (9.1) | 40 (12.2) | 50 (15.2) | 60 (18.3) | 70 (21.3) | 80 (24.4) | 90 (27.4) | 100 (30.5) |
| | 0.5 | 0.06 | 0.07 | 0.07 | 0.08 | 0.08 | 0.09 | 0.09 | 0.09 | 0.09 | 0.10 |
| 100:1 | 1 | 0.08 | 0.09 | 0.10 | 0.10 | 0.11 | 0.11 | 0.12 | 0.12 | 0.12 | 0.12 |
| | 2 | 0.10 | 0.12 | 0.14 | 0.15 | 0.16 | 0.17 | 0.18 | 0.19 | 0.19 | 0.20 |
| | 3 | 0.14 | 0.18 | 0.20 | 0.22 | 0.23 | 0.25 | 0.26 | 0.27 | 0.28 | 0.29 |
| | 4 | 0.16 | 0.21 | 0.25 | 0.28 | 0.30 | 0.33 | 0.35 | 0.37 | 0.38 | 0.40 |
| 20:1 | 5 | 0.17 | 0.24 | 0.29 | 0.34 | 0.38 | 0.41 | 0.45 | 0.48 | 0.51 | 0.53 |
| | 6 | 0.21 | 0.30 | 0.37 | 0.43 | 0.48 | 0.52 | 0.56 | 0.60 | 0.64 | 0.67 |
| | 7 | 0.26 | 0.37 | 0.45 | 0.52 | 0.58 | 0.64 | 0.69 | 0.74 | 0.78 | 0.82 |
| 12½:1 | 8 | 0.31 | 0.44 | 0.54 | 0.63 | 0.70 | 0.77 | 0.83 | 0.89 | 0.94 | 0.99 |
| | 9 | 0.37 | 0.52 | 0.64 | 0.74 | 0.83 | 0.91 | 0.98 | 1.05 | 1.11 | 1.17 |
| 10:1 | 10 | 0.43 | 0.61 | 0.75 | 0.87 | 0.97 | 1.06 | 1.15 | 1.22 | 1.30 | 1.37 |
| | 11 | 0.50 | 0.71 | 0.86 | 1.00 | 1.12 | 1.22 | 1.32 | 1.41 | 1.50 | 1.58 |
| 8:1 | 12.5 | 0.61 | 0.86 | 1.05 | 1.22 | 1.36 | 1.49 | 1.61 | 1.72 | 1.82 | 1.92 |
| | 15 | 0.81 | 1.14 | 1.40 | 1.62 | 1.81 | 1.98 | 2.14 | 2.29 | 2.43 | 2.56 |
| 6:1 | 16.7 | 0.96 | 1.36 | 1.67 | 1.92 | 2.15 | 2.36 | 2.54 | 2.72 | 2.88 | 3.04 |
| 5:1 | 20 | 1.29 | 1.82 | 2.23 | 2.58 | 2.88 | 3.16 | 3.41 | 3.65 | 3.87 | 4.08 |
| 4½:1 | 22 | 1.51 | 2.13 | 2.61 | 3.02 | 3.37 | 3.69 | 3.99 | 4.27 | 4.53 | 4.77 |
| 4:1 | 25 | 1.86 | 2.63 | 3.23 | 3.73 | 4.16 | 4.56 | 4.93 | 5.27 | 5.59 | 5.89 |
| | 30 | 2.51 | 3.56 | 4.36 | 5.03 | 5.62 | 6.16 | 6.65 | 7.11 | 7.54 | 7.95 |
| 3:1 | 33.3 | 2.98 | 4.22 | 5.17 | 5.96 | 6.67 | 7.30 | 7.89 | 8.43 | 8.95 | 9.43 |
| | 35 | 3.23 | 4.57 | 5.60 | 6.46 | 7.23 | 7.92 | 8.55 | 9.14 | 9.70 | 10.22 |
| 2½:1 | 40 | 4.00 | 5.66 | 6.93 | 8.00 | 8.95 | 9.80 | 10.59 | 11.32 | 12.00 | 12.65 |
| | 45 | 4.81 | 6.80 | 8.33 | 9.61 | 10.75 | 11.77 | 12.72 | 13.60 | 14.42 | 15.20 |
| 2:1 | 50 | 5.64 | 7.97 | 9.76 | 11.27 | 12.60 | 13.81 | 14.91 | 15.94 | 16.91 | 17.82 |
| | 55 | 6.48 | 9.16 | 11.22 | 12.96 | 14.48 | 15.87 | 17.14 | 18.32 | 19.43 | 20.48 |
| 1¾:1 | 57 | 6.82 | 9.64 | 11.80 | 13.63 | 15.24 | 16.69 | 18.03 | 19.28 | 20.45 | 21.55 |
| | 60 | 7.32 | 10.35 | 12.68 | 14.64 | 16.37 | 17.93 | 19.37 | 20.71 | 21.96 | 23.15 |
| 1½:1 | 66.7 | 8.44 | 11.93 | 14.61 | 16.88 | 18.87 | 20.67 | 22.32 | 23.87 | 25.31 | 26.68 |
| | 70 | 8.98 | 12.70 | 15.55 | 17.96 | 20.08 | 21.99 | 23.75 | 25.39 | 26.93 | 28.39 |
| | 75 | 9.78 | 13.83 | 16.94 | 19.56 | 21.87 | 23.95 | 25.87 | 27.66 | 29.34 | 30.92 |
| 1¼:1 | 80 | 10.55 | 14.93 | 18.28 | 21.11 | 23.60 | 25.85 | 27.93 | 29.85 | 31.66 | 33.38 |
| | 85 | 11.30 | 15.98 | 19.58 | 22.61 | 25.27 | 27.69 | 29.90 | 31.97 | 33.91 | 35.74 |
| | 90 | 12.02 | 17.00 | 20.82 | 24.04 | 26.88 | 29.44 | 31.80 | 34.00 | 36.06 | 38.01 |
| | 95 | 12.71 | 17.97 | 22.01 | 25.41 | 28.41 | 31.12 | 33.62 | 35.94 | 38.12 | 40.18 |
| 1:1 | 100 | 13.36 | 18.89 | 23.14 | 26.72 | 29.87 | 32.72 | 35.34 | 37.78 | 40.08 | 42.24 |

*Calculated from

$$LS = \left(\frac{65.41 \times s^2}{s^2 + 10{,}000} + \frac{4.56 \times s}{\sqrt{s^2 + 10{,}000}} + 0.065\right)\left(\frac{l}{72.5}\right)^m$$

LS = topographic factor
$l$ = slope length, ft (m × 0.3048)
$s$ = slope steepness,
$m$ = exponent dependent upon slope steepness (0.2 for slopes < 1%, 0.3 for slopes 1 to 3%, 0.4 for slopes 3.5 to 4.5%, and 0.5 for slopes > 5%)



C.11-15

| Slope ratio | Slope gradient $s$, % | LS values for following slope lengths $l$, ft (m) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 150 (46) | 200 (61) | 250 (76) | 300 (91) | 350 (107) | 400 (122) | 450 (137) | 500 (152) | 600 (183) | 700 (213) | 800 (244) | 900 (274) | 1000 (305) |
| | 0.5 | 0.10 | 0.11 | 0.11 | 0.12 | 0.12 | 0.13 | 0.13 | 0.13 | 0.14 | 0.14 | 0.14 | 0.15 | 0.15 |
| 100:1 | 1 | 0.14 | 0.14 | 0.15 | 0.16 | 0.16 | 0.16 | 0.17 | 0.17 | 0.18 | 0.18 | 0.19 | 0.19 | 0.20 |
| | 2 | 0.23 | 0.25 | 0.26 | 0.28 | 0.29 | 0.30 | 0.32 | 0.33 | 0.34 | 0.36 | 0.37 | 0.39 | 0.40 |
| | 3 | 0.32 | 0.35 | 0.38 | 0.40 | 0.42 | 0.43 | 0.45 | 0.46 | 0.49 | 0.51 | 0.54 | 0.55 | 0.57 |
| | 4 | 0.47 | 0.53 | 0.58 | 0.62 | 0.66 | 0.70 | 0.73 | 0.76 | 0.82 | 0.87 | 0.92 | 0.96 | 1.00 |
| 20:1 | 5 | 0.36 | 0.76 | 0.85 | 0.93 | 1.00 | 1.07 | 1.13 | 1.20 | 1.31 | 1.42 | 1.51 | 1.60 | 1.69 |
| | 6 | 0.82 | 0.95 | 1.06 | 1.16 | 1.26 | 1.34 | 1.43 | 1.50 | 1.65 | 1.78 | 1.90 | 2.02 | 2.13 |
| | 7 | 1.01 | 1.17 | 1.30 | 1.43 | 1.54 | 1.65 | 1.75 | 1.84 | 2.02 | 2.18 | 2.33 | 2.47 | 2.61 |
| 12½:1 | 8 | 1.21 | 1.40 | 1.57 | 1.72 | 1.85 | 1.98 | 2.10 | 2.22 | 2.43 | 2.62 | 2.80 | 2.97 | 3.13 |
| | 9 | 1.44 | 1.66 | 1.85 | 2.03 | 2.19 | 2.35 | 2.49 | 2.62 | 2.87 | 3.10 | 3.32 | 3.52 | 3.71 |
| 10:1 | 10 | 1.68 | 1.94 | 2.16 | 2.37 | 2.56 | 2.74 | 2.90 | 3.06 | 3.35 | 3.62 | 3.87 | 4.11 | 4.33 |
| | 11 | 1.93 | 2.23 | 2.50 | 2.74 | 2.95 | 3.16 | 3.35 | 3.53 | 3.87 | 4.18 | 4.47 | 4.74 | 4.99 |
| 8:1 | 12.5 | 2.35 | 2.72 | 3.04 | 3.33 | 3.59 | 3.84 | 4.08 | 4.30 | 4.71 | 5.08 | 5.43 | 5.76 | 6.08 |
| | 15 | 3.13 | 3.62 | 4.05 | 4.43 | 4.79 | 5.12 | 5.43 | 5.72 | 6.27 | 6.77 | 7.24 | 7.68 | 8.09 |
| 6:1 | 16.7 | 3.72 | 4.30 | 4.81 | 5.27 | 5.69 | 6.08 | 6.45 | 6.80 | 7.45 | 8.04 | 8.60 | 9.12 | 9.62 |
| 5:1 | 20 | 5.00 | 5.77 | 6.45 | 7.06 | 7.63 | 8.16 | 8.65 | 9.12 | 9.99 | 10.79 | 11.54 | 12.24 | 12.90 |
| 4½:1 | 22 | 5.84 | 6.75 | 7.54 | 8.26 | 8.92 | 9.54 | 10.12 | 10.67 | 11.68 | 12.62 | 13.49 | 14.31 | 15.08 |
| 4:1 | 25 | 7.21 | 8.33 | 9.31 | 10.20 | 11.02 | 11.78 | 12.49 | 13.17 | 14.43 | 15.58 | 16.66 | 17.67 | 18.63 |
| | 30 | 9.74 | 11.25 | 12.57 | 13.77 | 14.88 | 15.91 | 16.87 | 17.78 | 19.48 | 21.04 | 22.49 | 23.86 | 25.15 |
| 3:1 | 33.3 | 11.55 | 13.34 | 14.91 | 16.33 | 17.64 | 18.86 | 20.00 | 21.09 | 23.10 | 24.95 | 26.67 | 28.29 | 29.82 |
| | 35 | 12.52 | 14.46 | 16.16 | 17.70 | 19.12 | 20.44 | 21.68 | 22.86 | 25.04 | 27.04 | 28.91 | 30.67 | 32.32 |
| 2½:1 | 40 | 15.50 | 17.89 | 20.01 | 21.91 | 23.67 | 25.30 | 26.84 | 28.29 | 30.99 | 33.48 | 35.79 | 37.96 | 40.01 |
| | 45 | 18.62 | 21.50 | 24.03 | 26.33 | 28.44 | 30.40 | 32.24 | 33.99 | 37.23 | 40.22 | 42.99 | 45.60 | 48.07 |
| 2:1 | 50 | 21.83 | 25.21 | 28.18 | 30.87 | 33.34 | 35.65 | 37.81 | 39.85 | 43.66 | 47.16 | 50.41 | 53.47 | 56.36 |
| | 55 | 25.09 | 28.97 | 32.39 | 35.48 | 38.32 | 40.97 | 43.45 | 45.80 | 50.18 | 54.20 | 57.94 | 61.45 | 64.78 |
| 1¾:1 | 57 | 26.40 | 30.48 | 34.08 | 37.33 | 40.32 | 43.10 | 45.72 | 48.19 | 52.79 | 57.02 | 60.96 | 64.66 | 68.15 |
| | 60 | 28.35 | 32.74 | 36.60 | 40.10 | 43.31 | 46.30 | 49.11 | 51.77 | 56.71 | 61.25 | 65.48 | 69.45 | 73.21 |
| 1½:1 | 66.7 | 32.68 | 37.74 | 42.19 | 46.22 | 49.92 | 53.37 | 56.60 | 59.66 | 65.36 | 70.60 | 75.47 | 80.05 | 84.38 |
| | 70 | 34.77 | 40.15 | 44.89 | 49.17 | 53.11 | 56.78 | 60.23 | 63.48 | 69.54 | 75.12 | 80.30 | 85.17 | 89.78 |
| | 75 | 37.87 | 43.73 | 48.89 | 53.56 | 57.85 | 61.85 | 65.60 | 69.15 | 75.75 | 81.82 | 87.46 | 92.77 | 97.79 |
| 1¼:1 | 80 | 40.88 | 47.20 | 52.77 | 57.81 | 62.44 | 66.75 | 70.80 | 74.63 | 81.76 | 88.31 | 94.41 | 100.13 | 105.55 |
| | 85 | 43.78 | 50.55 | 56.51 | 61.91 | 66.87 | 71.48 | 75.82 | 79.92 | 87.55 | 94.57 | 101.09 | 107.23 | 113.03 |
| | 90 | 46.55 | 53.76 | 60.10 | 65.84 | 71.11 | 76.02 | 80.63 | 84.99 | 93.11 | 100.57 | 107.51 | 114.03 | 120.20 |
| | 95 | 49.21 | 56.82 | 63.53 | 69.59 | 75.17 | 80.36 | 85.23 | 89.84 | 98.42 | 106.30 | 113.64 | 120.54 | 127.06 |
| 1:1 | 100 | 51.74 | 59.74 | 66.79 | 73.17 | 79.03 | 84.49 | 89.61 | 94.46 | 103.48 | 111.77 | 119.48 | 126.73 | 133.59 |

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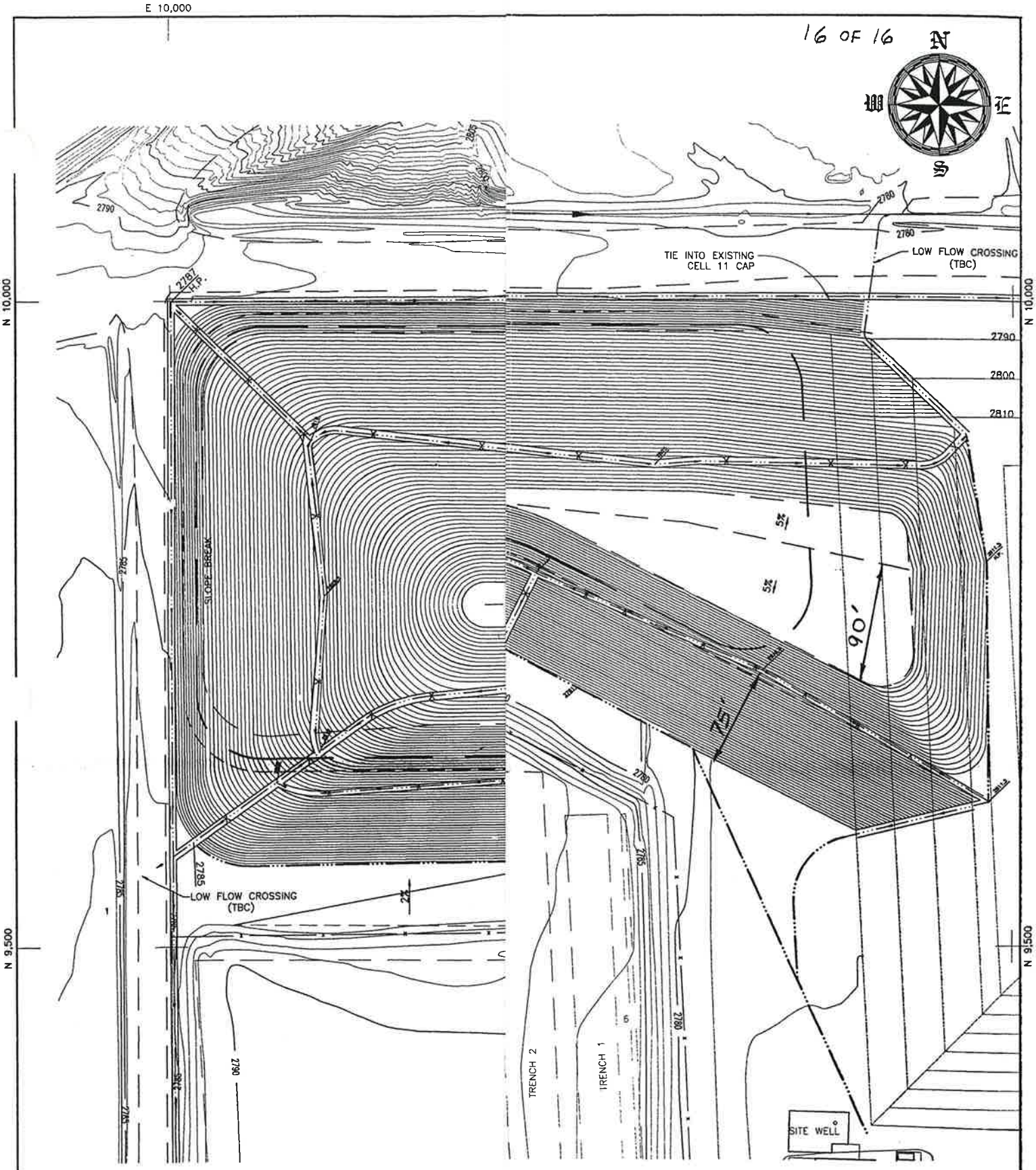



## LEGEND

- DRAINAGE DITCH AND DIRECTION OF FLOW
- DRAINAGE SWALE AND DIRECTION OF FLOW
- PERMIT LINE
- FENCE LINE
- INDICATED DIRECTION OF STORWATER FLOW
- 90' MAX. SLOPE LENGTH

E 2

| | |
|---|---|
| HAZARDOUS WASTE MANAGEMENT FACILITY<br>U.S. ECOLOGY<br>BEATTY NEVADA | |
| SURFACE COVER EROSION MAP<br>CELL 12 | |
| ENVIRONMENTAL SOLUTIONS, INC. | |
| JOB. NO. 95284<br>SCALE AS NOTED | D-95284-BL003 |

DESCRIPTION | DRAWN BY | CHK'D BY | APPROVED BY

EXHIBIT D

MTR EQUIVALENCY DEMONSTRATION

# EXHIBIT D

## D.1.0 INTRODUCTION AND PURPOSE

1. The liner design for Cell 12 at the US Ecology, Inc. facility located outside of Beatty, Nevada includes a lower component of the bottom composite liner which differs from that of the standard Minimum Technology Requirements (MTR) as presented in regulation. However, the regulations allow for flexibility in liner designs, provided that such designs are demonstrated to be equivalent to the MTR design. This paper demonstrates the MTR equivalency of the proposed liner system for Cell 12 at the Beatty landfill.

2. The remainder of this paper is organized as follows:
   - Section D.2.0 - MTR and Cell 12 Liner Cross Sections
   - Section D.3.0 - Regulatory Approval of Alternative Designs
   - Section D.4.0 - Equivalency Demonstration
   - Section D.5.0 - Summary and Conclusions
   - Section D.6.0 - References
   - Appendix D.1 - Calculations

## D.2.0 MTR AND CELL 12 LINER CROSS SECTIONS

1. The MTR liner system components are specified in regulation at Title 40 of the Code of Federal Regulations (CFR) in Section 264.301(c), which states in part:

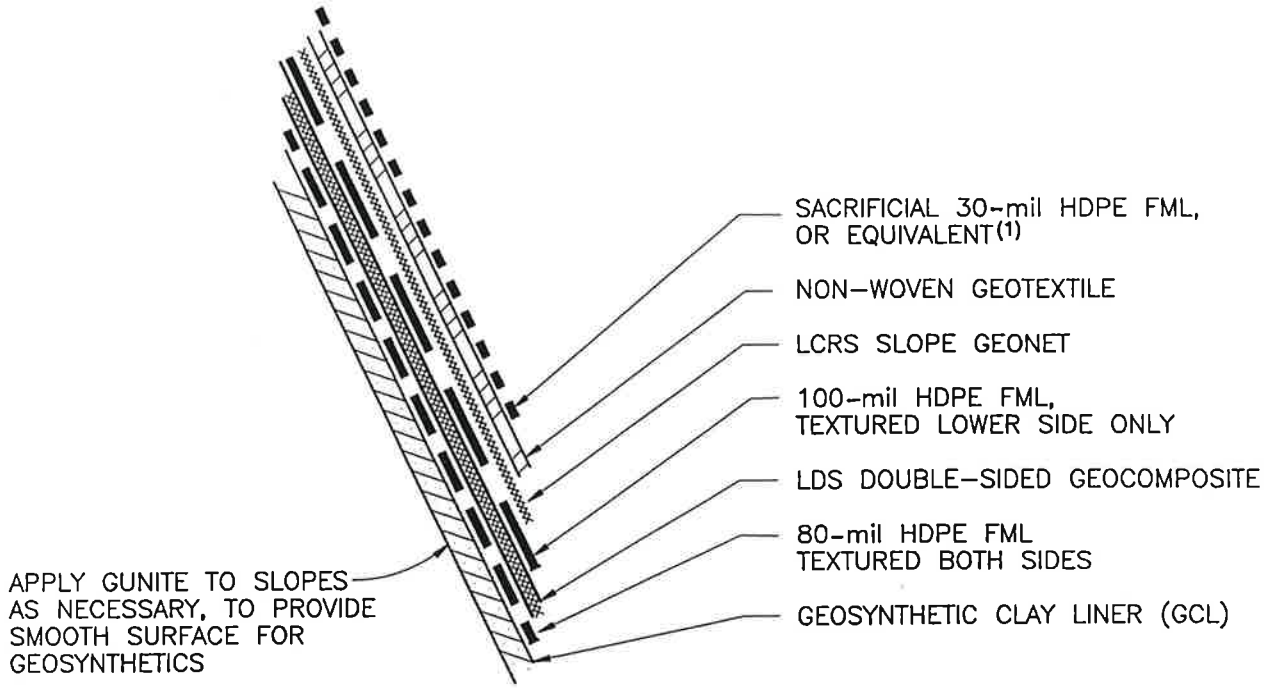
   "(1)(i) The liner system must include:

   "(A) A top liner designed and constructed of materials (e.g., a geomembrane) to prevent the migration of hazardous constituents into such liner during the active life and post-closure care period; and

   "(B) A composite bottom liner, consisting of at least two components. The upper component must be designed and constructed of materials (e.g., a geomembrane) to prevent the migration of hazardous constituents into this component during the active life and post-closure care period. The lower component must be designed and constructed of materials to minimize the migration of hazardous constituents if a breach in the upper component were to occur. The lower component must be constructed of at least 3 feet (91 cm) of compacted soil material with a hydraulic conductivity of no more than $1 \times 10^{-7}$ cm/sec."

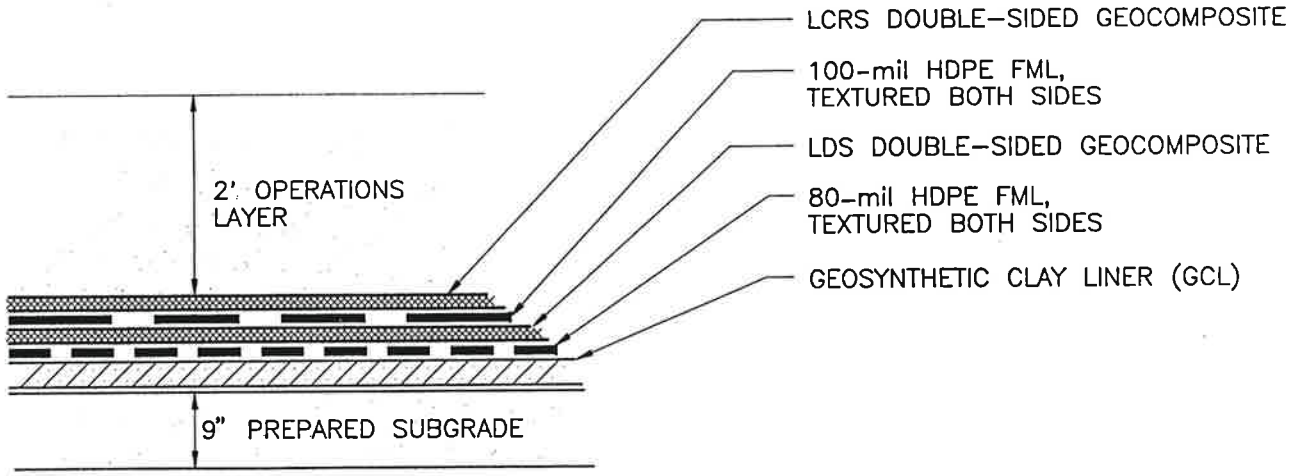
2. Figure D.1-1 shows cross sections of the Cell 12 base and sideslope liner systems. The top (primary) liner and the upper component of the bottom (secondary) composite liner for both the base and sideslope systems include 100-mil and 80-mil high density polyethylene (HDPE), respectively, which conform to MTR (and are thicker than recommended by EPA

95-284 MTR REV. 3/8/96




## TYPICAL SLOPE LINER SYSTEM

(GEOSYNTHETICS SHOWN EXAGGERATED FOR CLARITY)




## TYPICAL BASE LINER SYSTEM

(GEOSYNTHETICS SHOWN EXAGGERATED FOR CLARITY)

NOTE: (1) SACRIFICIAL HDPE LINER ON SLOPES TO BE REPLACED WITH 2-FOOT OPERATIONS LAYER AS WASTE FILLING PROGRESSES.

FIGURE D.1

**TYPICAL LINER SYSTEMS**

US ECOLOGY, INC.
BEATTY, NEVADA

TRC ENVIRONMENTAL SOLUTIONS, INC.

draft guidance [EPA, 1987]) and are therefore not addressed in this exhibit. The lower component of the bottom composite liner varies by location on the cell base and cell sideslopes, both of which differ from the MTR standard of 3 feet of compacted soil with a hydraulic conductivity of $\leq 1 \times 10^{-7}$ cm/sec. As shown in Figure D.1-1, the lower component of the Cell 12 bottom composite liner system includes the following:

| Location | Lower Component of Bottom Composite Liner System |
|---|---|
| Cell Base | Geosynthetic clay liner (GCL), underlain by 9 inches of compacted fine-grained soil.(1) |
| Cell Sideslopes | GCL, with subgrade treatment to prevent penetration potential, where appropriate. |

3. The purpose of the compacted soil layer below the GCL in cell base locations is twofold: (1) to provide a smooth load bearing surface for the overlying liner components to increase puncture resistance; and (2) to provide an additional hydraulic and chemical adsorption barrier against potential leachate migration. The compacted soil layer will have a silty sand consistency, to provide a relatively low permeability (e.g., on the order of $10^{-5}$ cm/sec), but without the magnitude of clay content which would have a high potential for substantial desiccation to occur. As discussed below, this layer will be beneficial for retarding both fluid flux and chemical constituent migration.

4. Cell sideslope locations have no potential to experience fluid head buildup, and therefore do not require the compacted soil layer. For example, calculations provided in *Geosynthetic Composite Liner System for Subtitle D* (Snow et al., 1994) show that the leachate head for a particular landfill located in the Los Angeles Basin that has slopes which vary from 1H:1V to 1.5H:1V would only be $1.5 \times 10^{-5}$ meters (about 0.00005 feet)) in the drainage layer above an FML. This head, which is conservative for the steeper slope and lower rainfall conditions at Cell 12, is used for evaluating sideslope liner system performance in the following discussions.

5. The remainder of this paper is focused on the lower component of the Cell 12 bottom composite liner systems (at both base and sideslope locations), and the equivalency of a GCL to 3 feet of clay with a hydraulic conductivity of $\leq 1 \times 10^{-7}$ cm/sec.

(1) The prepared subgrade soil will be screened from the excavated materials, moisture conditioned and compacted to 90 percent of its Modified Proctor density.

## D.3.0 REGULATORY APPROVAL OF ALTERNATIVE DESIGNS

1. The regulations at 40 CFR 264.301(d) provide flexibility for the consideration of liner designs other than the MTR prescriptive configuration. The subject regulations state in part:

> "(d) The Regional Administrator may approve alternative design or operating practices to those specified in paragraph (c) of this section if the owner or operator demonstrates to the Regional Administrator that such design and operating practices, together with location characteristics:
>
> "(1) Will prevent the migration of any hazardous constituent into the ground water or surface water at least as effectively as the liners and leachate collection and removal systems specified in paragraph (c) of this section."

2. The regulations have a second equivalency demonstration requirement in the ensuing paragraph at Section 264.301(d)(2) which relates to the detection of leaks through the top liner. However, that second equivalency demonstration is not required since the design of the Cell 12 leachate detection system conforms to MTR.

## D.4.0 EQUIVALENCY DEMONSTRATION

### D.4.1 GENERAL

1. This section presents an MTR equivalency demonstration for the lower component of the Cell 12 bottom composite liner systems. Section D.4.2 addresses the methodology with which to evaluate MTR equivalency. The ensuing sections present the equivalency demonstration for criteria identified to be most important in Section D.4.2.

### D.4.2 METHODOLOGY

1. Neither regulation nor regulatory guidance provide a methodology or evaluation criteria with which to make an MTR equivalency demonstration for an alternative liner design. However, the fact that: (1) the MTR standard for the lower component of a bottom composite liner is specified in terms of a prescriptive thickness and hydraulic conductivity; and (2) the alternative liner design must be demonstrated to be equivalent in terms of preventing migration, both support the application of steady-state fluid flux as the key regulatory evaluation criterion. Additional equivalency assessment evaluation criteria (for consideration of other aspects of a GCL) are discussed in the paragraphs below.

2. Published literature includes at least two evaluations on the equivalency of a GCL to a compacted clay liner (CCL). These evaluations are found within the following workshop and conference proceedings:

   - *Report of Workshop on Geosynthetic Clay Liners* (1993). The workshop was sponsored by EPA. Chapter 7 of this report is authored by D.E. Daniel and is entitled "Equivalency."
   - *GRI-7 Conference on Geosynthetic Liner Systems* (1994). The conference was sponsored by the Geosynthetics Research Institute (GRI). Pages 255 through 275 from these proceedings are authored by R. M. Koerner and D. E. Daniel and are entitled "Technical Equivalency Assessment of GCLs to CCLs."

3. The latter of the two is similar to and is an apparent revision to the former. The Nevada Division of Environmental Protection (NDEP) has recommended that the former be used as the basis to evaluate the equivalency of a GCL to a CCL. On this issue, the *Report of Workshop on Geosynthetic Clay Liners* states in the report Abstract on Page v:

   > "A number of criteria might be applied, but only a few seem truly rational. Steady-state water flux and solute flux are obvious and clear criteria that should usually be part of an equivalency analysis. Other criteria can be applied but most are much less meaningful in terms of addressing regulatory compliance."

   The former report thus corroborates the selection of steady-state fluid flux as the key evaluation criterion.

4. Chapter 7 of the *Report of Workshop on Geosynthetic Clay Liners* assesses GCL-to-CCL equivalency for 20 evaluation criteria (see Table D.4-1) and categorizes the results for each criteria as either: (1) Probably Equivalent; (2) Probably Not Equivalent; and (3) Product or Site-Specific. A review of the Probably Equivalent and Product or Site-Specific categories indicates, for the application planned at the Beatty facility, that a GCL is superior or equivalent, to a CCL for the subject criteria (see rationale given in Table D.4-1).
   The Probably Not Equivalent category contains the three criteria of adsorption capacity, solute breakthrough time and puncture resistance. Accordingly, these three criteria are evaluated herein as part of this Cell 12 equivalency assessment. The discussions for adsorption capacity and solute breakthrough are combined since both relate to the potential for constituents in leachate to migrate through the liner systems.

5. The *GRI-7 Conference on Geosynthetic Liner Systems* also identifies the criterion of horizontal fluid flow in seams or lifts as an issue to be considered. This criterion is therefore also included as part of this Cell 12 equivalency assessment.

# TABLE D.4-1
# SUMMARY OF EQUIVALENCY ASSESSMENT
# US ECOLOGY, INC.

## BEATTY NEVADA FACILITY, CELL 12 DESIGN

| SUMMARY OF NON-SPECIAL ASSESSMENT IN EPA GUIDANCE[1] | | | | | RATIONALE SPECIFIC TO APPLICATION AT CELL 12 |
|---|---|---|---|---|---|
| Category | Criterion For Evaluation | Equivalency of GCL to CCL | | | |
| | | Probably Equivalent | Probably Not Equivalent | Product or Site-Specific | |
| Hydraulic Issues | Steady Flux of Water[2] | | | ● | GCL equivalent. GCL will be specified with maximum permeability of 5 x $10^{-9}$ cm/sec (Claymax Shear Pro 500). |
| | Steady Solute Flux[2] | | | ● | GCL equivalent. Sodium bentonite material will be used to assure material integrity. |
| | Adsorption Capacity | | ● | | GCL equivalent. See discussion of adsorption capacity (Section D.4.4). |
| | Breakthrough Time:<br>• Water<br>• Solute | | | ● | GCL equivalent. GCL will be specified with maximum permeability of 5 x $10^{-9}$ cm/sec and a minimum thickness of 7 mm (Claymax Shear Pro 500). |
| | | | ● | | GCL equivalent. See discussion of solute breakthrough time (Section D.4.4). |
| | Consolidation Water | ● | | | GCL superior since material will not be wetted. |
| Physical/ Mechanical Issues | Freeze-Thaw | ● | | | GCL superior since level of saturation of the GCL is low. |
| | Wet-Dry | ● | | | GCL superior because of arid conditions. CCL would likely desiccate. |
| | Total Settlement | ● | | | GCL equivalent.[3] |
| | Differential Settlement | ● | | | GCL equivalent.[3] |
| | Slope Stability | | | ● | GCL superior because Claymax Shear Pro 500 material has high internal shear strength. |
| | Erosion | | | ● | GCL superior since material exposed to rainfall during construction will be replaced. |
| | Bearing Capacity | | | ● | GCL equivalent since Claymax Shear Pro 500 material is sewn on 4-inch centers which confines the bentonite material. |
| Construction Issues | Puncture Resistance | | ● | | GCL equivalent. See discussion of puncture resistance (Section D.4.5). |
| | Subgrade Condition | | | ● | GCL equivalent. See special subgrade preparation requirements (Section D.4.5). |
| | Ease of Placement | ● | | | GCL superior due to 0.5:1 (H:V) slopes and soil construction constraints in small base areas. |
| | Speed of Construction | ● | | | GCL superior since placement and trimming of CCL would be very difficult and time consuming. |
| | Availability of Materials | ● | | | GCL superior since a CCL material source is not available near the site. A bentonite admixture with site soils would likely be required, resulting in a high potential for desiccation to occur. |
| | Weather Constraints | ● | | | GCL superior since the short installation period for a GCL can be scheduled for dry periods only, whereas the CCL would require several months of continuous construction to install. |
| | Quality Assurance | ● | | | GCL superior since manufacturing quality control of the GCL is readily monitored. Field installation monitoring of a GCL is also less complex. |

95-284 (3/8/96/bd)

(1) Modified after EPA, 1993.
(2) Considered to be a primary criterion for demonstration of regulatory equivalency.
(3) Foundation settlement is negligible. Deformation of ultimate waste configuration does not result in unacceptable strains. See discussion in Chapter 3.0 of Cell 12 Design Report.

TRC Environmental Solutions Inc.

6. Note that several of the criteria from the two papers, such as slope stability, bearing capacity and stresses in materials on the steep sideslopes, although not part of this equivalency assessment are design factors which are addressed within the Cell 12 Design Report.

7. In summary, the Cell 12 MTR equivalency assessment criteria which are evaluated in additional detail are the following (as presented in Sections D.4.4 through D.4.6):

   - Adsorption Capacity and Solute Breakthrough Time
   - Puncture Resistance
   - Horizontal Flow in Seams or Lifts

   A discussion of the steady flux of water for the MTR and Cell 12 systems is initially provided in Section D.4.3 to evaluate the basic hydraulic properties of GCL materials which are specified. Those properties are then used for the other evaluations. The steady flux evaluation also illustrates the superiority of the Cell 12 systems for this key criteria.

### D.4.3 STEADY FLUX OF WATER

1. Calculations presented in Appendix 1 evaluate the steady-state flux of water through the lower component of the Cell 12 bottom composite liner for both the base and sideslope locations. The calculations compare those values to flux rates that would be expected for the MTR of 3 feet of CCL with a hydraulic conductivity of $\leq 1 \times 10^{-7}$ cm/sec. The comparisons provided in the calculations do not consider the high probability that the CCL will desiccate fairly rapidly[(2)] under very dry conditions at the Cell 12 site. Such desiccation would potentially result in large cracks in the CCL which would allow the flow of much larger quantities of water.

2. Key calculation conditions include the following: (1) the thickness of the hydrated GCL is 7 millimeters; (2) the hydraulic conductivity of the GCL is $5 \times 10^{-9}$ cm/sec in sideslope locations, and $5 \times 10^{-10}$ cm/sec in base locations (which experience overburden pressures); and (3) the constant fluid head on the lower component of the bottom composite liner is 0.1 feet for base areas and 0.00005 feet for sideslope areas. These assumptions are based on material properties obtained from various GCL manufacturers, and conservative leachate heads within the Cell 12 secondary leachate collection and removal system.

---

(2) In a recent personal communication with David E. Daniel, Ph.D., of the University of Texas at Austin, Dr. Daniel stated that there is a "good possibility to a high probability" of CCL desiccation within 10 years in an arid environment.

3. Calculation results are summarized below:

| Application | Component Type | Flux ($cm^3/cm^2$-sec) |
|---|---|---|
| MTR (on base) | 3 feet CCL (without desiccation) | $1.0 \times 10^{-7}$ |
| Cell 12 Base | 7 mm GCL plus 9 inches soil | $2.7 \times 10^{-9}$ |
| MTR (on sideslopes) | 3 feet CCL (without desiccation) | $1.0 \times 10^{-7}$ |
| Cell 12 Sideslopes | 7 mm GCL | $5 \times 10^{-9}$ |

4. As expected, the lower component of the Cell 12 bottom composite liner, in both cell bottom and sideslope locations, has a lower flux and is thus considered to be equivalent (or superior) to MTR. The results also illustrate the very low potential for leakage to occur at the steep sideslope areas.

5. Because solute flux is equal to the product of solute concentration and water flux, the above estimates of water flux are proportional to the rate at which solute flux would occur, if a source of liquid were to exist for a period sufficient for long-term steady-state conditions to develop. This comparison is made at two locations on Page 263 of the *GRI-7 Conference on Geosynthetic Liner Systems* paper:

> "Thus, the ratio of [advective] solute flux is the same as the ratio of water flux. Therefore, if one has demonstrated equivalency in terms of steady water flux, one has necessarily also demonstrated equivalency in terms of steady mass flux of solute."
>
> "Solutes can also migrate through soil liners by advection plus diffusion. However, since advective and diffusive mass fluxes are additive, and since the advective mass flux dominates, demonstration of equivalency in terms of water flux will generally ensure equivalency in terms of total mass flux."

The discussion in Section D.4.4 shows that the length of time for solute breakthrough to occur is so great that such steady-state conditions will never occur.

## D.4.4 ADSORPTION CAPACITY AND SOLUTE BREAKTHROUGH TIME

1. Adsorption capacity is addressed in the *Report of Workshop on Geosynthetic Clay Liners.* As stated on Page 87 of that report:

> "Adsorption is only relevant in the short term. When steady state mass transport is reached, adsorption capacity is exhausted. Equivalency in terms of adsorption, if evaluated at all, should be evaluated in terms of a specified performance period. For example, suppose the performance period being considered is 30 years. If the adsorption capacity of neither the CCL nor the GCL is exhausted after 30 years, both types of liner have 'reserve' adsorption capacity and may be considered equivalent for the

performance period. Alternatively, if either or both is exceeded, breakthrough of solute will occur and other issues, e.g., steady state solute flux, will require consideration."

Accordingly, chemical adsorption capacity equivalency must first be evaluated relative to the potential for solute breakthrough to occur within a specified performance period, as discussed below.

2. Calculations presented in Appendix 1 evaluate solute breakthrough time for the MTR and Cell 12 designs for the lower component of a bottom composite liner for both the cell base and sideslope configurations.

3. The solute composition used for the breakthrough calculations is based on chemical analyses of leachate from operational Cell 11, as provided in Attachment A to Appendix 1 of this exhibit. That data shows that zinc and chloroform are the dominant inorganic and organic constituents, respectively.

4. Solute breakthrough time is similar to adsorption capacity as an evaluation criteria in that breakthrough time is only relevant in the short term. The *Report of Workshop on Geosynthetic Clay Liners* addresses this issue on Page 88 with the following statement:

   "One must carefully consider whether the breakthrough time for solutes is relevant. In the long term, breakthrough time is irrelevant -- breakthrough will eventually occur in all liner systems with an outward gradient. The important long-term issue is solute flux."

5. The quote in Item 1 supposes a performance period of 30 years. For Cell 12, a performance period of 10 years (the estimated operating life) might be more appropriate. After that period, the cover will be installed and the potential for the secondary leachate collection and removal system (i.e., that which overlies the bottom composite liner) to be exposed to leachate would essentially be eliminated. The use of an additional 30 years to also include the postclosure care and maintenance period would be conservative. If solute breakthrough would not occur during a performance period of 40 years for either the Cell 12 or MRL liner configuration, the systems should be considered to be equivalent.

6. Solute breakthrough time calculations provided in Appendix 1 have been developed using two different chemical retardation methods. Method A is based on cation exchange of inorganic constituents. As the leachate moves into the liner, inorganic constituents like zinc occupy cation exchange sites and hence are removed from the leachate. As the exchange sites become

occupied, zinc and other ionic constituents remaining in the leachate at some distance back from the leading edge can move forward, but at a velocity slower than the leachate. Method B is based on the concept of retardation of a constituent by reversible adsorption. Instead of a molecule being permanently removed from the leachate by cation exchange, the constituent molecules are temporarily adsorbed and released. The result is that the initial full constituent concentration front in the leachate becomes retarded. Instead, a gradually increasing concentration develops with distance back from the leading edge. The resulting time for breakthrough of the 50 percent concentration is calculated. Calculation results are summarized below:

| CELL LOCATION | LINER TYPE | SOLUTE CONSTITUENT | BREAKTHROUGH TIME (years) | |
|---|---|---|---|---|
| | | | Method A | Method B |
| Base | Cell 12 | Zinc | 5,000 to 11,000 | 300,000 |
| | | Chloroform | Not Applicable | 44,000 |
| | MTR (assuming undesiccated CCL) | Zinc | 100 to 1,000 | 36,000 |
| | | Chloroform | Not Applicable | 5,300 |
| Sideslopes | Cell 12 | Zinc | 510 to 1,400 | 6,700 |
| | | Chloroform | Not Applicable | 1,000 |
| | MTR (assuming undesiccated CCL) | Zinc | 230 to 2,300 | 83,000 |
| | | Chloroform | Not Applicable | 12,000 |

If the anticipated formation of desiccation cracks were to be considered for the CCL, the breakthrough times for the MTR systems would be much less than those shown in the table. For example, if a crack were to form through the clay in the 10-year period indicated by Dr. Daniel (see Section D.4.3), a direct pathway would exist. For that case, the potential breakthrough time would be only 10 years.

7. The more important data shown in the above table are the very long periods required for breakthrough to occur for the Cell 12 systems. Since these periods are much greater than the performance period, breakthrough would not occur. For this condition, the liners would have substantial reserve adsorption (or breakthrough time) capacity and therefore equivalence is demonstrated.

## D.4.5 PUNCTURE RESISTANCE

1. For the lower component of a bottom composite liner, GCL puncture resistance is an issue for the following conditions and activities:

   - Subgrade Conditions (Protection from Below)
   - Construction Activities (Protection during Installation)
   - Operations Activities (Protection from Above)

   These conditions and activities are discussed in the following sections.

### D.4.5.1 SUBGRADE CONDITIONS

1. There is the possibility of GCL puncture due to compression (from overburden pressures) onto an inadequately prepared subgrade. The design for Cell 12 includes a specification for subgrade preparation which is established to assure that large and/or sharp materials are not allowed to puncture the overlying GCL. That specification is provided in Exhibit B of the Cell 12 Design Report, and includes the following:

   - In cell base locations, a 9-inch layer of compacted, fine-grained soil will be placed below the GCL. This layer will have a smooth, compacted surface and will protect the GCL from sharp gravel projections which may exist at the base of the cell excavation.
   - In cell sideslope locations, which will experience relatively low forces oriented normal to the 0.5H:1V slopes, exposed bands or layers of sharp or otherwise inappropriate subgrade materials will be treated with protective materials (as necessary). This protection will consist of a gunnite coating, or its equivalent (e.g., geotextile, sprayed elastomer).

2. These elements of the Cell 12 design will sufficiently protect the GCL against penetrations occurring as the result of subsurface materials.

### D.4.5.2 CONSTRUCTION ACTIVITIES

1. On the issue of GCL puncture due to construction activities, as stated on Page 92 of the *Report of Workshop on Geosynthetic Clay Liners*, although GCLs will not have equivalent puncture resistance to CCLs:

   > "[Q]uality assurance and quality control procedures can be established and implemented to make the probability of a puncture during construction extremely low. Ultimately, site-specific conditions and quality assurance procedures will determine whether puncture is a relevant issue that deserves serious consideration."

2. As required in regulation at 40 CFR 264.19, Cell 12 will be constructed in accordance with a Construction Quality Assurance (CQA) Program designed to assure proper installation of the GCL and other components of the liner system. The CQA Program is one element of the landfill design package submitted for approval (see Exhibit B of the Cell 12 Design Report). In any such approval of Cell 12, NDEP is required to review and approve the Cell 12 CQA Program, and will thereby have the opportunity to assure that the program is adequate. In addition, NDEP will also have the opportunity to review implementation of the program during construction of the cell.

3. The Cell 12 CQA procedures are provided in Exhibit B and include the following special requirements which are designed to protect the GCL from puncture during installation:
   - The prepared subgrade will be visually inspected prior to GCL installation to check for the absence of potential sources of puncture. Identified areas of concern will be removed and/or repaired as described in Section D.4.5.1.
   - The handling of the GCL during unloading and installation will be observed to assure that those activities are properly protective of the material.
   - The GCL will be monitored during and upon installation to confirm proper installation procedures and check for apparent punctures.
   - Prior to being covered, the entire GCL surface will be carefully inspected. Identified holes and imperfections will be marked and repaired.

4. The Cell 12 CQA Program has been developed to assure that the installed GCL does not have unrepaired punctures. Since these careful procedures will be followed, the installed GCL will be equivalent to an MTR clay layer.

D.4.5.3 OPERATIONS ACTIVITIES

1. The following procedures will be implemented to protect against the possibility of GCL puncture due to operations activities:
   - A 2-foot operations layer of select waste will be placed above the liner on the cell base and sideslopes. The select waste will typically be soils and other similar materials (such as most filtercakes and incinerator ashes) which do not contain large, sharp or angular elements. The operations layer will be inspected as it is placed to check for conformity to specified properties. If select waste is not available, stockpiled material from site excavations will be used provided it is free of sharp rocks. Particle sizes of up to 6 inches in the largest dimension will be allowed provided the particle remains in a soil matrix.

- Drums, debris (such as concrete and rebar), transformers and other similar large and angular waste will not be placed within 5 feet of the liner.
- An operations "buffer zone" will be established adjacent to cell sideslopes, within which operations will be restrained and additionally monitored, to assure that operating equipment does not damage the sideslope liner.

2. The above procedures have been developed to protect the Cell 12 GCL as well as both of the overlying HDPE layers against the possibility of puncture due to operations activities. These operating practices will result in the lower component of the Cell 12 bottom composite liner being equivalent to the MTR standard design with respect to puncture potential from operations activities.

### D.4.6 HORIZONTAL FLOW IN SEAMS OR LIFTS

1. Horizontal liquid flow in seams or lifts is addressed in *GRI-7 Conference on Geosynthetic Liner Systems*. As stated on Page 265 of that paper:

> "Concerns are raised as to horizontal [liquid] flow which might be more rapid [than vertical] and tend to increase the water or solute flux over a large area. For GCLs, the concern is clearly in the overlap seam area. Yet, large scale experiments tend to substantiate manufacturers recommendations that the overlap areas either self-seal or, by adding bentonite, commingle with the abutting geotextiles to form an adequate seal, LaGatta, (1992). For CCLs, the concern is between individual lifts with inadequate bonding from one surface to the next, Rogowski (1990). This issue, as with the GCLs, is clearly related to CQC/CQA monitoring which will be discussed later. If properly constructed, neither material should be a major concern with respect to horizontal liquid flow."

2. Specific testing on this issue is reported in *Hydraulic Conductivity of Three Geosynthetic Clay Liners* (Estornell and Daniel, 1992), which describes results of bench-scale tank tests performed on: (1) individual sheets of GCL; (2) overlapped pieces of GCL; and (3) composite liners comprised of a punctured geomembrane overlying a GCL. On the issue of flow in GCL seams (i.e., overlapping GCL sheets), the report states that:

> "The hydraulic conductivities of overlapped GCLs were about the same as those of the control samples with no overlap; an effective hydraulic seal developed along the overlaps in all of the materials tested."

3. Accordingly, a properly installed GCL is equivalent to an MTR CCL for the criterion of horizontal flow of liquid in seams or lifts. On the issue of proper installation, note that (as required in regulation at 40 CFR 264.19) Cell 12 will be constructed in accordance with a CQA Program designed to assure proper installation of the GCL.

## D.5.0 SUMMARY AND CONCLUSIONS

1. The Cell 12 liner design will provide equivalent or superior performance to the MTR standard liner cross section. The gauge stipulated in regulation for the determination of MTR equivalency is whether the alternative design will prevent the migration of constituents at least as effectively as the MTR standard design. The lower component of the Cell 12 bottom liner has been demonstrated to be at least as effective as the MTR standard in preventing constituent migration, and is thus deemed to be equivalent. Considering the real potential of desiccation of the CCL portion of the MTR system, the Cell 12 design is actually superior for site-specific conditions.

## D.6.0 REFERENCES


Daniel, D. E. University of Texas at Austin. Personal communication regarding GCLs and their equivalency to CCLs for landfill liner systems. Environmental Solutions, Inc. December 19, 1995.

Estornell, P. and D. E. Daniel. *Hydraulic Conductivity of Three Geosynthetic Clay Liners.* Journal of Geotechnical Engineering, Volume 118, Number 10. October 1992.

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Snow, M. S., R. Bonaparte and E. Kavazanjian, Jr. *Geosynthetic Composite Liner System for Subtitle D.* WasteTech, 1994.

U.S. Environmental Protection Agency. *Draft Minimum Technology Guidance on Double Liner Systems for Landfills and Surface Impoundments -- Design, Construction, and Operation.* EPA/530-SW-85-014. April 1987.

U.S. Environmental Protection Agency. *Report of Workshop on Geosynthetic Clay Liners,* EPA/600/R-93/171. Daniel, D. E. and B. T. Boardman. August 1993.

U.S. Environmental Protection Agency. *40 CFR Parts 260 to 299.* Revised as of July 1, 1995.

APPENDIX 1

COMPARISON OF GEOSYNTHETIC CLAY LINER (GCL)
WITH AND WITHOUT A COMPACTED SOIL LINER (CSL)
VERSUS A COMPACTED CLAY LINER (CCL)

# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 16 Jan 96 Subject COMPARISON OF GCL + CSL AND CCL Sheet No. 1 of 32
Chkd. By TLP Date 1/31/96 Proj. No. 95-234

PURPOSE

Calculate measures of leachate migration that test the expected equivalence to a regulatory-specified compacted clay liner (CCL) of two engineered alternative liners, one made from a geosynthetic clay liner (GCL) for side-slopes, and the second made from a GCL over a compacted soil liner (CSL) for the bottom.

APPROACH

Based on the comparative cross-sections shown in Figures 1A and 1B, the migration of one representative inorganic and one representative organic constituent downward through each liner will be simulated. One measure of leachate migration is the flux of water which will be calculated through each liner to compare the flux through each of the two engineered alternatives with the flux through the regulatory-specified liner. Solute breakthrough time will be calculated by simulating retardation of the representative constituents.



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# ENVIRONMENTAL SOLUTIONS, INC.

By BMI Date 8 Dec 95 Subject Comparison of GCL + CSL Sheet No. 2 of 32
Chkd. By TLP Date 1/31/96 AND CCL Proj. No. 95-239

FIGURE 1A

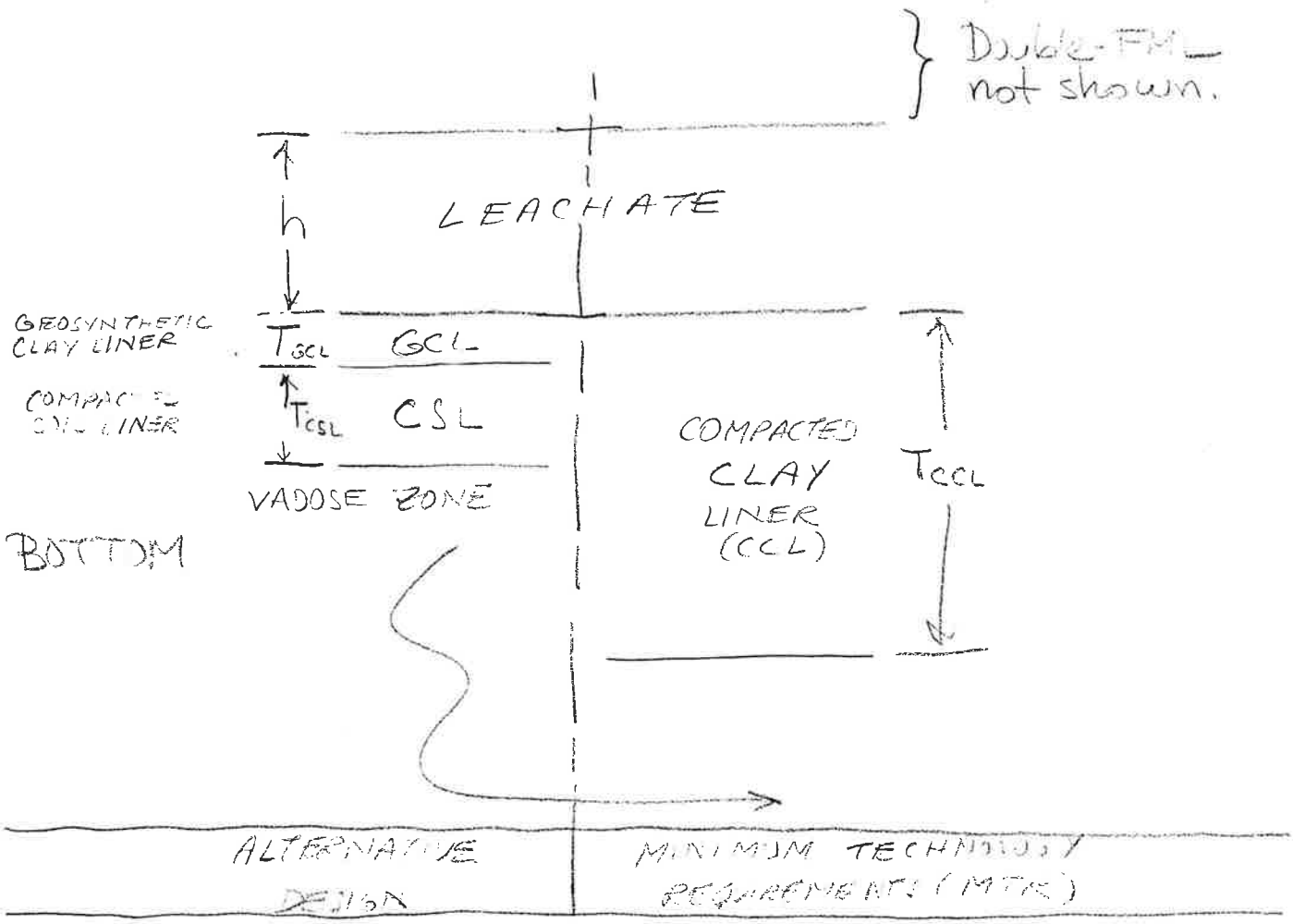
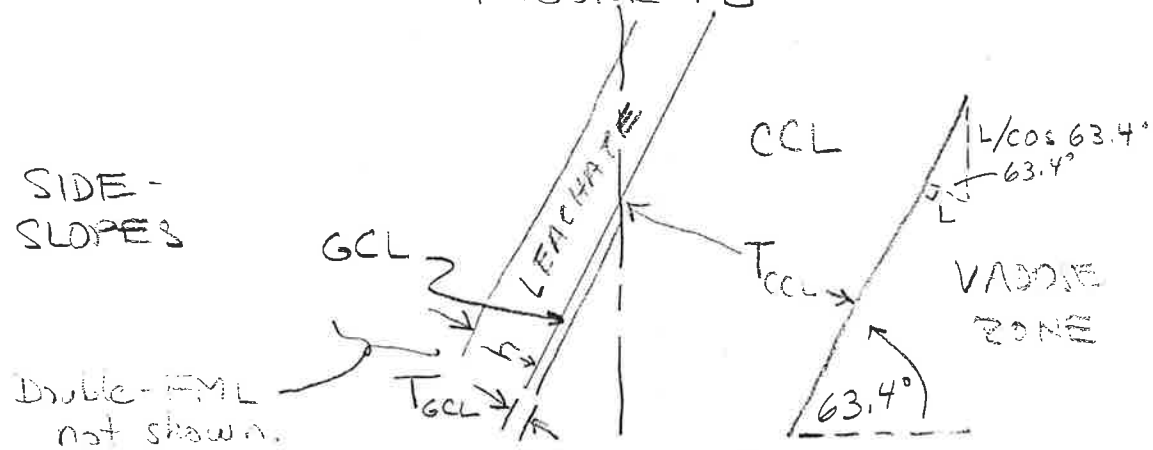



FIGURE 1B



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# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 16 Jan 96 Subject COMPARISON OF GCL ± CSL AND CCL Sheet No. 3 of 32
Chkd. By [illegible] Date 1/31/96 Proj. No. 95-284

ASSUMPTIONS

- Water flux and solute breakthrough time are measures of leachate migration requiring proof of equivalence by Nevada DEP/EPA Protection.
- Cell 12 of Beatty (NW) Facility is the location for which the lining systems will be compared.
- Double flexible membrane liners (FML's), below the clay liners to provide primary protection against leachate migration, are assumed to allow leachate to reach the top of the clay liner and build a head.
- The leachate heads on the bottom and sideslope liners are 0.1 ft and $4.9 \times 10^{-5}$ ft, respectively. The bottom value is a conservative choice, while the sideslope value was calculated by Snow et al (1994). Both heads are low because Beatty is arid, the waste does not contain much water, and the drainage layer on top of the first FML (LCRS) and between the two FMLs would remove most of the head created by infrequent rain.
- Representative constituents for migration calculation are zinc (inorganic) and chloroform (organic).
- Concentration of zinc in leachate = 1,100 $\mu g\ l^{-1}$ (maximum), based on analyses by TRI/Environmental, Inc. (1994) (See Attachment A).
- Concentration of chloroform in leachate = 240 $\mu g\ l^{-1}$ (max.) based on same analyses (See Attachment A).



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By [illegible] Date 16 Jan 96 Subject COMPARISON OF GCL + CSL AND CCL Sheet No. 4 of 32
Chkd. By TLS Date 1/22/96 Proj. No. 95-284

ASSUMPTIONS (continued)

- Thickness of GCL = 7 mm (conservative choice of 7-10 mm for hydrated GCLs stated in Daniel and Boardman (1993) in Attachment E).
- Thickness of CSL = 9 in. (= 0.75 ft) by design for bottom.
- Thickness of CCL = 3 ft. by regulatory requirement (40 CFR 264.301(c)).
- Density (wet) of clay in GCL = 67 lb/ft³, based on manufacturer's specifications and calculation in Attachment B.
- Density (wet) of clay in CCL = 127 lb/ft³, according to measurements and calculation in Attachment B.
- Density (wet) of CSL = 108 lb/ft³ according to The Mark Group (1989) (see Attachment B).
- Hydraulic conductivity of GCL on side-slopes = $5 \times 10^{-9}$ cm/s (Attachment B).
- Hydraulic conductivity of GCL = $5 \times 10^{-10}$ cm/s or less at the bottom of the cell where the effective stress is assumed to be at least 30 psi (= 4320 psf). These specifications agree with the Claymax data in Attachment B and the recommendation by Daniel (1993) that hydraulic conductivity decreases by a factor of 10 under the load in the cell



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# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 16 Jan 96 Subject COMPARISON OF GCL + CSL AND CCL Sheet No. 5 of 32
Chkd. By TLP Date 1/31/96 Proj. No. 95-284

ASSUMPTIONS (continued)

- Hydraulic conductivity of CSL lies in the wide range of $5 \times 10^{-4}$ to $3 \times 10^{-8}$ cm/s according to falling head permeability tests conducted by The Mark Group (1989) on undisturbed bulk soil samples extracted from test borings (Attachment B).

  The local (Beatty, NV) soil is assumed to have a hydraulic conductivity no higher than $10^{-5}$ cm/s, and is likely to be lower.

- Hydraulic conductivity of CCL = $10^{-7}$ cm/s, as required by 40 CFR 264.301(c).

- Loss of inorganic ionic constituent such as zinc ($Zn^{+2}$) is caused by cation exchange in which only one constituent molecule is held for each 100 exchange sites ($f_s$ = 0.01 in calculation). It is assumed that other competing constituent molecules occupy the other 99 sites.



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# ENVIRONMENTAL SOLUTIONS, INC.

By BGW Date 8Dec95 Subject COMPARISON OF GCL+CSL AND CCL Sheet No. 6 of 32
Chkd. By TLP Date 1/31/96 Proj. No. 95-284

- Cation exchange capacity of natural clay for CCL = 3-30 meq/100g (Daniel and Boardman, 1993) (Attachment E)

- Cation exchange capacity of CSL = 16-35 meq/100g according to measurements by The Mark Group (1989).

- Clay in GCL and CCL has no cracks that could allow mass transport of leachate to the top of the vadose zone.

- The process of diffusion of constituents does not significantly alter the results of these calculations. The basis for this assumption is provided in Attachment K.

- Zinc (Zn) is divalent, hence 0.5 g mole = 1 equivalent.

- Cation exchange capacity of montmorillonite (or bentonite) in GCL = 80-150 meq/100g (Mitchell, 1976) (Attachment H), while Peck et al (1953) (Attachment I) give

$$\frac{360 \text{ to } 600 \times 10^{20} \text{ charges}/100\,g}{6.023 \times 10^{23} \text{ charges/equivalent}}$$

= 60 to 100 meq/100g. Cation exchange capacity of bentonite = 100-150 meq/100g (Daniel and Boardman, 1993) (Attachment E).



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# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 11Dec95 Subject COMPARISON OF GCL ± CSL Sheet No. 7 of 32
Chkd. By TLP Date 1/31/96 AND CCL Proj. No. 95-234

ASSUMPTIONS (continued)

- Distribution coefficient ($K_d$) for zinc $= \log_{10}^{-1} 2.8 \ \frac{cm^3}{g}$ (Dragun, 1988)(Attachment F) $= 631 \ cm^3/g$

- Octanol-water partition coefficient for chloroform (Attachment D)
  $K_{ow\,CHCl_3} = \log_{10}^{-1} 1.97 = 93$

- Density of water ($\rho_w$) = 62.4 lb/ft$^3$

- Precipitation of inorganic constituents is neglected, even though it sometimes exceeds cation exchange retardation.

- Total porosities of GCL, CSL and CCL are taken as 0.75, 0.4 and 0.437 vol/vol according to HELP Model Version 3.04 Soil Types 17, 10 and 25 (Attachment C), reduced to 0.3, 0.2 and 0.2 vol/vol effective porosities on the recommendation of Daniel (1995) (Attachment J).

- Because of the lack of information on the quantitative partition of an organic constituent (e.g., chloroform) between adsorption on the surface of clay and remaining in solution in the leachate, the octanol-water partition coefficient is used as an indirect surrogate.



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# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 8DEC95 Subject COMPARISON OF GCL + CSL AND CCL Sheet No. 8 of 32
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METHODOLOGY, GENERAL

The leachate will move downward through either clay liner as described by Darcy's Law:

$$Q = K A I \qquad (1)$$

Where Q = liquid volume flow rate $[L^3 T^{-1}]$ $(cm^3 s^{-1})$
K = hydraulic conductivity of liner $[L T^{-1}]$ $(cm s^{-1})$
A = area through which flow is occurring $[L^2]$ $(cm^2)$
I = gradient of head $[L L^{-1}]$ (-)

The gradient of head is given by the expression:

$$I = \frac{dh}{dz} = \frac{h+T}{T} \qquad (2)$$

where h = head of liquid [L] (cm)
T = thickness of porous medium experiencing head [L] (cm)
Z = vertical coordinate [L] (cm)

In terms of volume flow rate per unit area or volume flux, Equations 1 and 2 can be combined as follows:

---

Brackets [ ] give dimensions of a variable in which L = length, T = time, and M = mass.
Parentheses ( ) give example units of a variable (e.g., cgs system).



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METHODOLOGY (continued)

$$F_v = \frac{Q}{A} = K\frac{dh}{dz} = v \quad \text{(bulk velocity through medium)} \qquad (3)$$

where $F_v$ = volume flux or equivalent vertical velocity $[L^3T^{-1}L^{-2} = LT^{-1}]$ $(cm^3 s^{-1} cm^{-2} = \frac{cm}{s})$

$v$ = velocity of liquid flow $[LT^{-1}]$ $(cm\ s^{-1})$

If no interactions took place between a constituent and the porous medium (e.g., clay), the constituent would have at least the same vertical downward velocity. The time it would then take for both the liquid carrier (water), and non-interacting constituents, are described by the following expression:

$$t_w = \frac{T\phi}{v} \quad (4A) \qquad \frac{v}{\phi} = v_p \quad (4B)$$

where $\phi$ = effective porosity of medium [-] (-), $v_p$ = linear velocity in pores $[LT^{-1}]$ $(cm\ s^{-1})$

Combining Equations 3 and 4A, we get:

$$t_w = \frac{T\phi}{K\frac{dh}{dz}} \qquad (5)$$

where $t_w$ = time for leachate (water) to move through liner material $[T]$ (sec)




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METHODOLOGY (continued)

The flux of water is calculated with the following expression that combines Eqs. 2 and 3.

$$F = K\left(\frac{h+T}{T}\right) \qquad (5A)$$

For the combination of two layers (GCL and CSL), the flux-constraining layer is the GCL on top because its hydraulic conductivity is only 1/20,000 that of the underlying CSL. Hence, the time required for leachate to penetrate the CSL is given by the following combination of Equations 3, 4B and 5A:

$$t_{CSL} = \frac{T_{CSL}\,\phi_{CSL}}{F_{v\,GCL}} \qquad (5B)$$

$$t_{CSL} = \frac{T_{CSL}\,\phi_{CSL}}{K_{GCL}\left(\frac{h+T_{GCL}}{T_{GCL}}\right)} \qquad (5C)$$



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METHODOLOGY (continued)

Two Methods, A and B, will now be explored to develop expressions describing the migration, loss and retardation of constituents in leachate. Method A is based on the loss of inorganic constituent molecules to cation exchange sites. Such loss results in the front of the constituent molecules appearing to move through the clay liner more slowly.

Method B is based on temporary retardation of constituent molecules at adsorption sites, also resulting in a slower movement of the constituent through the clay liner.



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METHOD A (FOR INORGANIC MIGRATION)

If interactions between the constituent molecules and porous medium (clay) involve loss of the constituent, then the constituent "front" will appear to move more slowly down through the liner. The rate of loss is proportional to the cation exchange capacity of the medium. Cation exchange capacity of a clay is a common measure of inorganic adsorptive capacity. Cation exchange capacity is usually measured in milli equivalents of constituent per 100 grams of medium. One equivalent equals Avogadro's number of molecules in a gram-mole of constituent (monovalent)

As the leachate migrates down into the clay liner at a volume flux of $F_v$, it is carrying with it the following amount of constituent:

$$\frac{dm_i}{dt} = F_v C_{iM} \qquad (6)$$



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METHOD A (continued)

where $n_i$ = number of gram-moles of constituent i moving down into liner per unit area [mole $L^{-2}$] (g moles $cm^{-2}$)

$C_{iM}$ = concentration of constituent i in leachate in terms of moles per unit volume of leachate [mole $L^{-3}$] (g mole $cm^{-3}$)

As the leachate mass downward through the clay at velocity $v_i$ the amount of constituent i that can be removed by adsorption is equal to the following:

$$\left.\frac{dn_i}{dt}\right|_L = v_i \rho_c \, CAC f_s \qquad (7)$$

where $\rho_c$ = mass density of the clay [M $L^{-3}$] (g $cm^{-3}$)

CAC = chemical adsorptive capacity [moles $M^{-1}$] (g mole $g^{-1}$) in moles of constituent per unit mass of medium

$f_s$ = fraction of sites absorbing constituent i [-] (-)

$v_i$ = effective velocity of constituent i [$LT^{-1}$] (cm $s^{-1}$)

At the front edge of the adsorbed constituent shown in Figure 2, the arriving molar flux equals the flux of adsorption.

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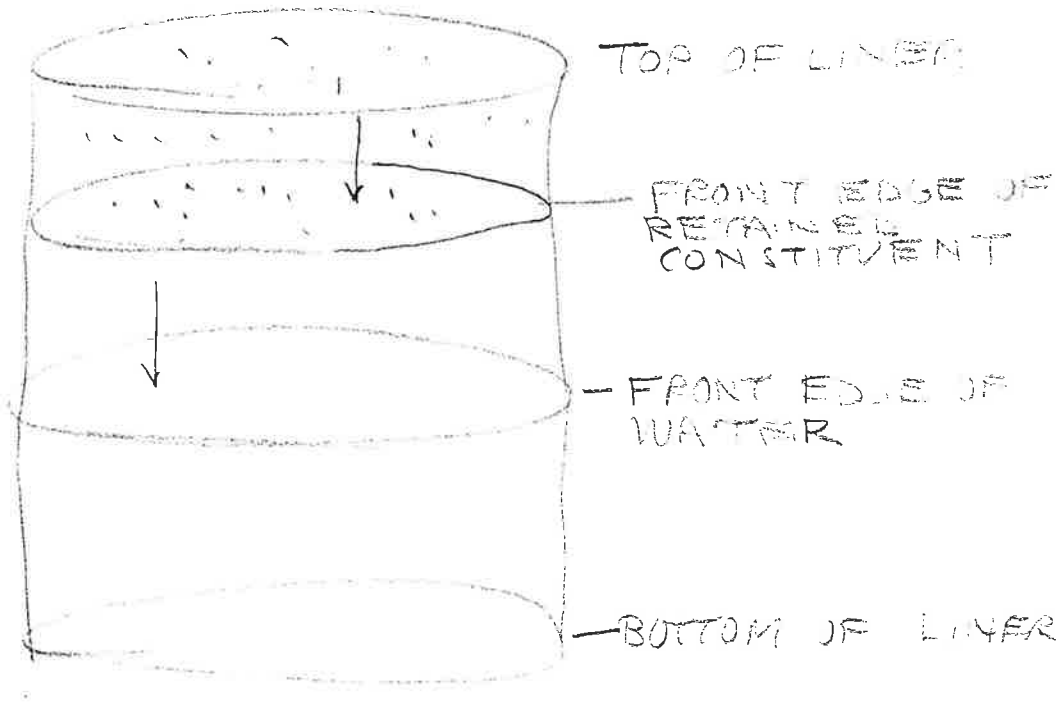
METHOD A (continued)

FIGURE 2



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METHOD A (continued)

The effective velocity at which constituent i can move down through the adsorbing medium is given by:

$$\frac{dn_i}{dt} = \left.\frac{dn_i}{dt}\right|_L \qquad (8)$$

$$F_v C_{iM} = v_i \rho_c CAC f_s \qquad (9)$$

$$v_i = \frac{F_v C_{iM}}{\rho_c CAC f_s} \qquad (10)$$

The time it would take for constituent i to progress through the liner at its adsorbing front is given by:

$$t_i = \frac{T\phi}{v_i} \qquad (11)$$

Combining Eqs 10 and 11, we get:

$$t_i = \frac{T \rho_c CAC f_s \phi}{F_v C_{iM}} \qquad (12)$$

Combining Eqs. 2, 3, and 12, we get

$$t_i = \frac{T \rho_c CAC\, m_i f_s \phi}{K I C_{im}} \qquad (13)$$



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METHOD A (continued)

where $C_{im}$ = concentration of constituent i in terms of mass of i per unit volume of liquid [$M L^{-3}$] (g $cm^{-3}$ or mg $l^{-1}$)

$\mathcal{M}_i$ = molecular weight of constituent i [$M\ mole^{-1}$] (g $gmole^{-1}$)

In terms of inorganic cation exchange capacity, Eq. 13 can be rewritten as follows:

$$t_i = \frac{T \rho_c\ CEC\ a_i \mathcal{M}_i f_s \phi}{K I C_{im}} \qquad (14)$$

where CEC = cation exchange capacity of the porous medium [equivalents $M^{-1}$] (equivalents/100 g)

$a_i$ = number of gram moles of constituent i per equivalent of constituent [mole equivalent$^{-1}$] (g-mole/equivalent)



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METHOD 2

Following the discussion of adsorptive retardation in Freeze and Cherry (1979), the relationship between the average linear (pore) velocities of water (conservative) and retarded constituent i at 50% initial concentration is:

$$\frac{v_p}{v_{pi}} = 1 + \frac{\rho_c}{\phi} K_{di} \qquad (15)$$

where $v_{pi}$ = retarded velocity of constituent i through the pores (linear velocity) $[L T^{-1}]$ $(cm\ s^{-1})$

$K_{di}$ = distribution coefficient of constituent i $[L^3 M^{-1}]$ $(cm^3\ g^{-1})$

The distribution coefficient is the mass of solute (or sorbate) sorbed per unit mass of sorbent divided by the mass of sorbate in the leachate per unit volume of leachate.

$$K_{di} = \frac{C_{is}'}{C_{im}} \qquad (16)$$

where $C_{is}'$ = concentration of constituent i sorbed in terms of mass of i per unit mass of sorbent $[M M^{-1}]$ $(g\ g^{-1})$



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METHOD B (continued)

A closely related "distribution" coefficient is the octanol-water partition coefficient, defined as:

$$K_{ow\,i} = \frac{C'_{io}}{C'_{iw}} \qquad (17)$$

where $C'_{io}$ = concentration of constituent i in octanol in terms of mass of i per unit mass of octanol [M M$^{-1}$] (g g$^{-1}$)

$C'_{iw}$ = concentration of constituent i in water in terms of mass i per unit mass of water [M M$^{-1}$] (g g$^{-1}$)

Dragun (1988) (see Attachment G) discusses the important adsorption of constituents on mineral surfaces, depending only in part on the organic content of the soil. Because of the lack of information on this partition, we assume that the distributed concentration of constituent i in octanol indirectly represents the sorbed concentration on the surface of clay, hence

$$C'_{is} = C'_{io} \qquad (18)$$

The concentrations of constituent i in water are related as follows:

$$C_{im} = C'_{iw}\rho_w \quad \left(= \frac{M_i}{M_w}\,\frac{M_w}{V_w}\right) \qquad (19)$$



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METHOD B (continued)

Combining Eqs. 15, 16, 17, 18, and 19, we get:

$$\frac{V_P}{V_{Pi}} = 1 + \frac{\rho_c}{\phi}\frac{C_{is}'}{C_{im}}$$

$$= 1 + \frac{\rho_c}{\phi}\frac{C_{io}'}{C_{iw}'\rho_w}$$

$$= 1 + \frac{\rho_c}{\phi}\frac{C_{iw}' K_{owi}}{C_{iw}'\rho_w}$$

$$\frac{V_P}{V_{Pi}} = 1 + \frac{\rho_c}{\phi}\frac{K_{owi}}{\rho_w} \qquad (20)$$

Combining Eqs. 3, 4B, 11 and 28, we get:

$$t_i = \frac{T\phi}{V_i} = \frac{T}{V_{Pi}}$$

$$= \frac{T\phi}{V}\left(1 + \frac{\rho_c}{\rho_w}\frac{K_{owi}}{\phi}\right)$$

$$t_i = \frac{T\phi}{KI}\left(1 + \frac{\rho_c}{\rho_w}\frac{K_{owi}}{\phi}\right) = \frac{T}{KI}\left(\phi + \frac{\rho_c}{\rho_w}K_{owi}\right) \qquad (21)$$

For inorganic constituents,

$$t_i = \frac{T\phi}{KI}\cdot\left(1 + \frac{\rho_c}{\phi}K_d\right) = \frac{T}{KI}\left(\phi + \rho_c K_d\right) \qquad (22)$$




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CALCULATIONS

To calculate the flux of water through each bottom liner we use Eq. 3A, noting that $F_{v\,CSL} = F_{v\,GCL}$

$$F_{v\,CSL} = 5\times10^{-10}\,\frac{cm}{s}\left(\frac{0.1\,ft\;304.8\,\frac{mm}{ft} + 7\,mm}{7\,mm}\right)$$

$$F_{v\,GCL} = 2.68\times10^{-9}\,\frac{cm^3}{cm^2\text{-}s}$$

Hence,

$$F_{v\,CSL} = 2.68\times10^{-9}\,\frac{cm^3}{cm^2\,sec}$$

The GCL is controlling the flux because $K_{CSL}$ is in the range of $10^{-5}\,\frac{cm}{s} < K_{CSL} < 10^{-4}\,\frac{cm}{s}$.

$$F_{v\,CCL} = 10^{-7}\,\frac{cm}{s}\left(\frac{0.1\,ft + 3\,ft}{3\,ft}\right)$$

$$F_{v\,CCL} = 1.03\times10^{-7}\,\frac{cm^3}{cm^2\,sec}$$

Hence, the flux of water through the GCL/CSL is less than the flux through the CCL. On the side-slopes, we use Eq. 5A:

$$F_{v\,GCL} = \left(\frac{4.9\times10^{-5}\,ft\;304.8\,\frac{mm}{ft} + \frac{7\,mm}{\cos 63.4^\circ}}{\frac{7\,mm}{\cos 63.4^\circ}}\right) \qquad 5\times10^{-9}\,\frac{cm}{s}$$

$$F_{v\,GCL} = 5.0\times10^{-9}\,cm^3/cm^2\,sec$$

$$F_{v\,CCL} = 10^{-7}\,cm/s\left(\frac{4.9\times10^{-5}\,ft + 3\,ft/\cos 63.4^\circ}{3\,ft/\cos 63.4^\circ}\right)$$

$$= 1.0\times10^{-7}\,cm^3/cm^2\,sec$$

Hence, $F_{v\,GCL} < F_{v\,CCL}$ on side-slopes.

# ENVIRONMENTAL SOLUTIONS, INC.

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CALCULATIONS USING METHOD A (FOR MIGRATION OF INORGANIC CONSTITUENTS)

BOTTOM LINER:

For GCL and inorganic constituent $i$ = zinc, using Eq. 14:

$$t_{Zn/GCL} = \frac{7\,mm \cdot \frac{1\,cm}{10\,mm}\; 67\,\frac{lb}{ft^3}\; 453.59\,\frac{g}{lb}\; \frac{1\,ft^3}{30.48^3\,cm^3}}{5\times10^{-10}\,\frac{cm}{s} \times \left(\frac{0.1\,ft\;\frac{304.8\,mm}{ft} + 7\,mm}{7\,mm}\right) 1100\times10^{-6}\,\frac{g\,Zn}{l}\,\frac{1\,l}{10^3\,cm^3}}$$

$$\cdot\; 60\times10^{-3}\,\frac{\text{equivalents of Zn}}{100\,g.\ clay}\;\frac{0.5\,gmole\ Zn}{1\ \text{equivalent Zn}}$$

$$\cdot\; \frac{65.37\,g\ Zn}{gmole\ Zn} \cdot 0.01\times0.3 = 1.5\cdot10^{10}\,sec$$

$\doteq$ 476 to 1190 years (for 60 to 150 $\frac{meq}{100g}$, respectively

$$t_{Zn/CSL} = \frac{9\,in\; 2.54\,\frac{cm}{in}\; 108\,\frac{lb\ CSL}{ft^3}\; 453.59\,\frac{g}{lb}\; \frac{1\,ft^3}{30.48^3\,cm^3}}{5\times10^{-10}\,\frac{cm}{s}\left(\frac{0.1\,ft\;304.8\,\frac{mm}{ft} + 7\,mm}{7\,mm}\right)}$$

$$\times\; \frac{16\times10^{-3}\,\text{equiv. Zn}}{100\,g.\ CSL} \times \frac{0.5\,g\,mole\ Zn}{1\ \text{equiv. Zn}} \times 65.37\,\frac{g\ Zn}{g\,mole\ Zn} \times 0.01\times0.2$$

$$1100\times10^{-6}\,\frac{g\ Zn}{l}\,\frac{1\,l}{10^3\,cm^3}$$

$t_{Zn/CSL} = 1.40\times10^{11}\,s$

= 4450 yr, which is large compared to 476 yr. to GCL, to 9,740 yrs for upper limit of CEC of 35 meq/100 g.



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# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 12 Jan 95 Subject COMPARISON OF GCL + CSL AND CCL Sheet No. 22 of 32

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CALCULATIONS (continued)

BOTTOM LINER (continued)

For the CCL and inorganic constituent (= zinc):

$$t_{Zn/CCL} = \frac{3\,ft \times 30.48\,\frac{cm}{ft} \times 127\,\frac{lb\,CCL}{ft^3}\,453.59\,\frac{g}{lb}\,\frac{1\,ft^3}{30.48^3\,cm^3}}{10^{-7}\,\frac{cm}{s} \times \left(\frac{0.1\,ft + 3\,ft}{3\,ft}\right)}$$

$$\times \frac{\frac{3\times10^{-3}\,equiv.\,Zn}{100\,g.\,CCL} \times \frac{0.5\,g\,mole\,Zn}{1\,equiv.\,Zn} \times 65.37\,\frac{g\,Zn}{g\,mole\,Zn} \times 0.01 \times 0.2}{1100\times10^{-6}\,\frac{g\,Zn}{l}\,\frac{1\,l}{10^3\,cm^3}}$$

$$= 3.21\times10^{9}\ sec.$$

$t_{Zn/CCL} \doteq$ 100 to 1000 yrs for cation exchange capacity ranging from 3 to 30 meq/100 g., respectively

SIDE-SLOPE LINER

$$t_{Zn/GCL} = \frac{\frac{7\,mm}{\cos 63.4^\circ} \times \frac{1\,cm}{10\,mm}\,67\,\frac{lb\,clay}{ft^3}\,453.59\,\frac{g}{lb}\,\frac{1\,ft^3}{30.48^3\,cm^3}}{5\times10^{-9}\,\frac{cm}{s}\left(\frac{4.9\times10^{-3}\,ft\,304.8\,\frac{mm}{ft} + \frac{7\,mm}{\cos 63.4^\circ}}{7\,mm/\cos 63.4^\circ}\right)1100\times10^{-6}\,\frac{g\,Zn}{l}\,\frac{1\,l}{10^3\,cm^3}}$$

$$\times \frac{60\times10^{-3}\,eq\,Zn}{100\,g.\,clay} \times \frac{0.5\,g\,mole\,Zn}{1\,eq\,Zn} \times 65.37\,\frac{g\,Zn}{g.\,mole\,Zn} \times 0.01 \times 0.3$$

$= 1.79\times10^{10}$ sec $=$ 568 yrs. to 1420 yrs for CEC = 150 meq/100 g.



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# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 8 Dec 95 Subject COMPARISON OF GCL + CSL AND CCL Sheet No. 23 of 32
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SIDE-SLOPE LINER (continued)

$$t_{Zn/CCL} = \frac{\frac{3\,ft}{\cos 63.4^\circ} \times \frac{30.48\,cm}{1\,ft} \times 127\,\frac{lb\ CCL}{ft^3}\ 453.59\,\frac{g}{lb} \cdot \frac{1\,ft^2}{30.48^2\,cm^3}}{10^{-7}\,\frac{cm}{s}\left(\frac{4.9\times10^{-5}\,ft + 3\,ft/\cos 63.4^\circ}{3\,ft/\cos 63.4^\circ}\right) 1100\times10^{-6}\,\frac{g\ Zn}{l}\ \frac{1\,l}{10^3\,cm^3}}$$

$$\times \frac{3\times10^{-3}\,eq.\,Zn}{100\,g\ CCL} \times \frac{0.5\ g\text{-}mole\ Zn}{1\ eq.\,Zn} \times 65.37\,\frac{g\ Zn}{g\text{-}mole\ Zn} \times 0.01 \times 0.2$$

$= 7.41 \times 10^9$ sec = 235 yrs to 2349 yrs for CEC = 30 meq/100 g.

CALCULATIONS FOR ZINC USING METHOD B

BOTTOM LINER:

Use Eq. 22. For GCL,

$$t_{Zn/GCL} = \frac{7\,mm\,\frac{1\,cm}{10\,mm} \times \left\{0.3 + \frac{67\,\frac{lb}{ft^3}\,453.59\,\frac{g}{lb}}{30.48^3\,\frac{cm^3}{ft^3}}\left[\left(10^{-1.8}\right)\frac{cm^2}{g}\right]\right\}}{5\times10^{-10}\,\frac{cm}{s}\left(\frac{0.1\,ft\ 304.8\,\frac{mm}{ft} + 7\,mm}{7\,mm}\right)}$$

$= 9.77 \times 10^{11}$ s $\doteq 5619$ yr $\simeq 5600$ yr

$$t_{Zn/CCL} = \frac{3\,ft \times 30.48\,\frac{cm}{ft}\left\{0.2 + \frac{127\,\frac{lb}{ft^3}\,453.59\,\frac{g}{lb}}{30.48^3\,\frac{cm^3}{ft^3}}\left[\left(10^{-1.8}\right)\frac{cm^2}{g}\right]\right\}}{10^{-7}\,\frac{cm}{s}\left(\frac{0.1\,ft + 3\,ft}{3\,ft}\right)}$$

$\doteq 1.14\times10^{12}$ sec = 36,000 yr.

$$t_{Zn/CSL} = \frac{9\,in \cdot 2.54\,\frac{cm}{in}\left\{0.2 + \frac{108\,\frac{lb}{ft^3}\,453.59\,\frac{g}{lb}}{30.48^3\,\frac{cm^3}{ft^3}}\left[\left(10^{-1.8}\right)\frac{cm^2}{g}\right]\right\}}{5\times10^{-10}\,\frac{cm}{s}\left(\frac{0.1\,ft\ 304.8\,\frac{mm}{ft} + 7\,mm}{7\,mm}\right)}$$

$= 9.32 \cdot 10^{12}$ s = 296,000 yrs, which is large compared to 5600 yrs.



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# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 3Dec95 Subject COMPARISON OF GCL + CSL AND CCL Sheet No. 24 of 32
Chkd. By RLP Date 1/31/96 Proj. No. 95-284

CALCULATIONS USING METHOD 2 (FOR RETARDATION OF ORGANIC CONSTITUENTS)

BOTTOM LINER:

For GCL and organic constituent chloroform, use

Eq. 21

$$t_{CHCl_3/GCL} = \frac{7\,mm \frac{1\,cm}{10\,mm} \cdot \left(0.3 + \frac{67\,\frac{lb}{ft^3}}{62.4\,\frac{lb}{ft^3}} \times 93\right)}{5\times10^{-10}\,\frac{cm}{s}\left(\frac{0.1\,ft^3/ft^2 \cdot 304.8\,\frac{mm}{ft} + 7\,mm}{7\,mm}\right)}$$

$$= 2.62\times10^{10}\ sec = 830\,yr$$

For CCL and organic constituent chloroform:

$$t_{CHCl_3/CCL} = \frac{3\,ft \cdot 30.48\,\frac{cm}{ft}}{10^{-7}\,\frac{cm}{s}\left(\frac{0.1\,ft^3/ft^2 + 3\,ft}{3\,ft}\right)}\left(0.2 + \frac{127\,\frac{lb}{ft^3}}{62.4\,\frac{lb}{ft^3}} \times 93\right)$$

$$= 1.68\times10^{11}\ sec.$$

$$= 5300\ yrs.$$

For CSL and $CHCl_3$:

$$t_{CHCl_3/CSL} = \frac{9\,in \times 2.54\,\frac{cm}{in}\left(0.2 + \frac{108\,lb/ft^3}{62.4\,lb/ft^3} \times 93\right)}{5\times10^{-10}\,\frac{cm}{s}\left(\frac{0.1\,ft^3/ft^2 \cdot 304.8\,\frac{mm}{ft} + 7\,mm}{7\,mm}\right)}$$

$$= 1.38\times10^{12}\ sec$$

$= 43{,}600$ yrs, which is long compared to 830 yrs for the GCL



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# ENVIRONMENTAL SOLUTIONS, INC.

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CALCULATIONS (continued)

METHOD B FOR SIDE-SLOPES

$$t_{zn/GCL} = \frac{\frac{7\,mm}{\cos 63.4^\circ} \cdot \frac{1\,cm}{10\,mm} \cdot \left\{0.3 + \frac{67\,\frac{lb}{ft^3}\,453.59\,\frac{g}{lb}}{30.48^3\,\frac{cm^3}{ft^3}}\left[\log^{-1} 2.8\,\frac{cm^3}{g}\right]\right\}}{5\times10^{-9}\,\frac{cm}{s}\left(\frac{4.9\times10^{-5}\,ft\,\frac{304.8\,mm}{1\,ft} + \frac{7\,mm}{\cos 63.4^\circ}}{7\,mm/\cos 63.4^\circ}\right)}$$

$t_{zn/GCL} = 2.12\times10^{11}$ sec = 6709 yrs ≃ 6700 yrs

For CCL on side slopes:

$$t_{zn/CCL} = \frac{\frac{3\,ft}{\cos 63.4^\circ} \cdot \frac{30.48\,cm}{1\,ft}\left\{0.2 + \frac{127\,\frac{lb}{ft^3}\,453.59\,\frac{g}{lb}}{30.48^3\,\frac{cm^3}{ft^3}}\left[\log^{-1}(2.3)\,\frac{cm^3}{g}\right]\right\}}{10^{-7}\,\frac{cm}{s}\left(\frac{4.9\times10^{-5}\,ft + 3\,ft/\cos 63.4^\circ}{3\,ft/\cos 63.4^\circ}\right)}$$

$t_{zn/CCL} = 2.62\times10^{12}$ sec = 83,000 yrs

For GCL and $CHCl_3$ on side slopes:

$$t_{CHCl_3/GCL} = \frac{\frac{7\,mm}{\cos 63.4^\circ} \cdot \frac{1\,cm}{10\,mm}\left(0.3 + \frac{67\,\frac{lb}{ft^3}}{62.4\,\frac{lb}{ft^3}} \cdot 13\right)}{5\times10^{-9}\,\frac{cm}{s}\left(\frac{4.9\times10^{-5}\,ft\,\frac{304.8\,mm}{1\,ft} + \frac{7\,mm}{\cos 63.4^\circ}}{7\,mm/\cos 63.4^\circ}\right)}$$

$t_{CHCl_3/GCL} = 3.12\times10^{10}$ sec = 990 yrs ≃ 1000 yrs



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By [illegible] Date 18 Jan 96 Subject COMPARISON OF GCL & CCL Sheet No. 26 of 32
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CALCULATION (continued)

METHOD B FOR SIDE-SLOPES (continued)

For CCL and $CHCl_3$ on side-slopes:

$$t_{CHCl_3/CCL} = \frac{\dfrac{3\,ft}{\cos 63.4^\circ} \times \dfrac{30.48\,cm}{1\,ft}\left(0.2 + \dfrac{127\,\frac{lb}{ft^3}}{62.4\,\frac{lb}{ft^3}} \times 93\right)}{10^{-7}\,\frac{cm}{s}\left(\dfrac{4.9\times10^{-5}\,ft + 3\,ft/\cos 63.4^\circ}{3\,ft/\cos 63.4^\circ}\right)}$$

$t_{CHCl_3/CCL} = 3.87 \times 10^{11}$ sec $= 12{,}270$ yr $\simeq 12{,}000$ yr.



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# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 18 Jan 96 Subject COMPARISON OF GCL + CSL AND CCL Sheet No. 27 of 32
Chkd. By [illegible] Date 1/31/96 Proj. No. 95-284

FINDINGS

The flux of water through each liner on the bottom and side-slopes is tabulated as follows:

| POSITION | LINER | WATER FLUX ($cm^3/cm^2$-sec) |
|---|---|---|
| BOTTOM | GCL+CSL | $2.7 \times 10^{-9}$ |
| | CCL (nondesiccated) | $1.0 \times 10^{-7}$ |
| SIDE-SLOPES | GCL | $5.0 \times 10^{-9}$ |
| | CCL (nondesiccated) | $1.0 \times 10^{-7}$ |

The breakthrough time for each solute (zinc and chloroform) for each liner in each position (bottom and side-slopes) is presented for Methods A and B in the following table (Note: Some values are rounded):



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By [illegible] Date 18 Jan 96 Subject COMPARISON OF GCL + CSL AND CCL Sheet No. 28 of 32

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FINDINGS (continued)

| POSITION | LINER | CONSTITUENT | BREAKTHROUGH TIME (YRS) STEP FUNCTION METHOD A | BREAKTHROUGH TIME (YRS) DISTRIBUTION COEFFICIENT METHOD B |
|---|---|---|---|---|
| BOTTOM | GCL + CSL | Zinc | 5,000 - 11,000 | 300,000 |
| | | Chloroform | NA | 44,000 |
| | CCL (nondesiccated) | Zinc | 100 - 1,000 | 36,000 |
| | | Chloroform | NA | 5,300 |
| SIDE-SLOPES | GCL | Zinc | 570 - 1,400 | 6,700 |
| | | Chloroform | NA | 1,000 |
| | CCL (nondesiccated) | Zinc | 230 - 2,300 | 83,000 |
| | | Chloroform | NA | 12,000 |

NA = Not applicable.

Note: Method A gives the break through time of the first constituent molecules, while Method B gives the later time when the concentration breaking through is half of the original concentration of the constituent in the leachate.



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CONCLUSIONS

The water flux through the engineered alternative liners on both the bottom and side-slopes of Cell 12 would be less than that through the regulatory-specified (CCL) liner. Hence, the alternative liners are equivalent in performance relative to water flux.

The times for representative inorganic and organic constituents (solutes) to break through the engineered alternative liners on both the bottom and side-slopes are much longer than commonly accepted periods of performance for such waste containment cells (e.g., 30 years). Because the breakthrough times for the solutes are much longer than the performance period of 30 years, "reserve" adsorptive capacity exists for each liner at the end of the performance period, and hence, the alternative liners are equivalent to the regulatory-specified liner.



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CONCLUSIONS (continued)

The calculations of water flux and breakthrough times for the CCL assume that the 3-foot thick layer is perfect, meaning that it has not desiccated and cracked. In reality, the arid environment of the Beatty facility creates a soil suction on moisture of tens to hundreds of atmospheres, leading to a "good possibility to a high probability" that a CCL will desiccate and crack (Daniel, 1995) (Attachment J). The network of desiccation cracks would increase the water flux and decrease the breakthrough times, probably by several orders of magnitude.

Method A gives shorter breakthrough times because it is based on the appearance of the first constituent molecules, while Method B gives the later breakthrough time when the concentration breaking through is half of the original concentration in the leachate.



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# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 8 Dec 95 Subject COMPARISON OF GCL + CSL AND CCL Sheet No. 31 of 32
Chkd. By RP Date 1/31/96 Proj. No. 95-284

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# ENVIRONMENTAL SOLUTIONS, INC.

By EGW Date 13Dec95 Subject COMPARISON OF GCL + CSL AND CCL Sheet No. 32 of 32
Chkd. By TLP Date 1/31/96 Proj. No. 95-284

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ATTACHMENT A

# LEACHATE ANALYSIS REPORT

ıerican Ecology Corporation
ls. Joe Gayle Simmons
33 Westheimer, Suite 1000
ıuston, TX 77056-5407

TRI Log No.: 3012-56-01
Date Reported: 07/15/94

## METALS

| TOTAL REC. METALS | UNITS | DL | DILUTION | RESULT | % RECOVERY | DATE |
|---|---|---|---|---|---|---|
| Antimony | ug/L | 10 | 5.000 | < 10 | 57 | 06/24/94 |
| Arsenic | ug/L | 10 | 5.000 | < 10 | 97 | 06/30/94 |
| Barium | mg/L | 0.005 | 1.000 | 0.398 | 44 | 06/27/94 |
| Beryllium | mg/L | 0.010 | 1.000 | < 0.010 | 43 | 06/27/94 |
| Cadmium | mg/L | 0.002 | 1.000 | 0.003 | 40 | 06/27/94 |
| Chromium | mg/L | 0.005 | 1.000 | < 0.005 | 34 | 06/27/94 |
| Cobalt | mg/L | 0.02 | 1.000 | < 0.02 | 37 | 06/27/94 |
| Copper | mg/L | 0.025 | 1.000 | < 0.025 | 54 | 06/27/94 |
| Lead | ug/L | 20.0 | 10.000 | < 20.0 | 124 | 06/27/94 |
| Mercury | ug/L | 0.2 | 1.000 | < 0.2 | 96 | 06/16/94 |
| Nickel | mg/L | 0.040 | 1.000 | 0.070 | 41 | 06/27/94 |
| Selenium | ug/L | 40 | 20.000 | < 40 | 61 | 06/22/94 |
| Silver | mg/L | 0.015 | 1.000 | < 0.015 | 80 | 06/28/94 |
| Thallium | ug/L | 40 | 20.000 | < 40 | 49 | 06/24/94 |
| Tin | mg/L | 0.10 | 2.000 | < 0.10 | 95 | 06/28/94 |
| Vanadium | mg/L | 0.010 | 1.000 | < 0.010 | 37 | 06/27/94 |
| Zinc | mg/L | 0.030 | 1.000 | 1.100 | 40 | 06/27/94 |

SRA
Quality Review

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ATTACHMENT A (continued)

# LEACHATE ANALYSIS REPORT

nerican Ecology Corporation
Ms. Joe Gayle Simmons
933 Westheimer, Suite 1000
ouston, TX 77056-5407

TRI Log No.: 3012-56-01
Date Reported: 07/15/94

## VOLATILE COMPOUNDS

Concentration: LOW
Date Extracted/Prepared:
Date Analyzed: 06/23/94
Dil Factor: 2.00 pH
Percent Moisture: (Not Decanted)

| Number | | UG/L | |
|---|---|---|---|
| 1-87-3 | Chloromethane | 20 | U |
| 1-83-9 | Bromomethane | 20 | U |
| 01-4 | Vinyl Chloride | 20 | U |
| 5-00-3 | Chloroethane | 20 | U |
| 9-2 | Methylene Chloride | 22 | |
| 9-4 | Trichlorofluoromethane | 20 | U |
| 64-1 | Acetone | 46 | |
| 5-15-0 | Carbon Disulfide | 10 | U |
| 35-4 | 1,1-Dichloroethene | 10 | U |
| 34-3 | 1,1-Dichloroethane | 32 | |
| 56-60-5 | trans-1,2-Dichloroethene | 10 | U |
| 66-3 | Chloroform | 240 | |
| -06-2 | 1,2-Dichloroethane | 10 | U |
| 8-93-3 | 2-Butanone | 20 | U |
| 1-55-6 | 1,1,1-Trichloroethane | 18 | |
| 23-5 | Carbon Tetrachloride | 10 | U |
| 3-05-4 | Vinyl Acetate | 20 | U |
| 5-27-4 | Bromodichloromethane | 10 | U |
| 87-5 | 1,2-Dichloropropane | 10 | U |
| 061-02-6 | trans-1,3-Dichloropropene | 10 | U |
| 9-01-6 | Trichloroethene | 10 | U |
| 24-48-1 | Dibromochloromethane | 10 | U |
| -00-5 | 1,1,2-Trichloroethane | 10 | U |
| 1-43-2 | Benzene | 10 | U |
| 10061-01-5 | cis-1,3-Dichloropropene | 10 | U |
| -25-2 | Bromoform | 10 | U |
| 3-10-1 | 4-Methyl-2-Pentanone | 20 | U |
| 591-78-6 | 2-Hexanone | 20 | U |

| CAS Number | | UG/L | |
|---|---|---|---|
| 127-18-4 | Tetrachloroethene | 10 | |
| 79-34-5 | 1,1,2,2-Tetrachloroethane | 20 | U |
| 108-88-3 | Toluene | 10 | U |
| 108-90-7 | Chlorobenzene | 10 | U |
| 100-41-4 | Ethylbenzene | 10 | U |
| 100-42-5 | Styrene | 10 | U |
| 75-71-88 | Dichlorodifluoromethane | 10 | U |
| | meta+para Xylenes | 10 | U |
| 95-47-6 | ortho-Xylene | 10 | U |
| 107-05-1 | Allyl Chloride | 20 | U |
| | Chloroprene | 10 | U |
| 74-88-4 | Methyl iodide | 10 | U |
| 80-62-6 | Methyl methacrylate | 10 | U |
| 126-98-7 | methacrylonitrile | 200 | U |
| 107-12-0 | Propionitrile | 200 | U |
| 123-91-1 | 1,4-dioxane | 200 | U |
| 75-05-8 | Acetonitrile | 200 | U |
| 78-83-1 | Isobutanol | 200 | U |
| 107-02-8 | Acrolein | 200 | U |
| 107-13-1 | Acrylonitrile | 200 | U |
| 106-93-4 | 1,2-Dibromoethane | 20 | U |
| 110-57-6 | trans-1,4-dichloro-2-butene | 20 | U |
| | Ethyl Methacrylate | 10 | U |
| 630-20-6 | 1,1,1,2-tetrachloroethane | 10 | U |
| 96-18-4 | 1,2,3-trichloropropane | 10 | U |
| 75-95-3 | Dibromomethane | 10 | U |
| 96-12-8 | 1,2-dibromo-3-chloropropane | 10 | U |

U - Compound analyzed for but not detected. The reported value is the minimum attainable practical quantitatio quantitation limit (PQL) for the samples. A minimum reporting level (MRL) of 1.0 ppb has been established for all the analytes, except meta+para-xylene, for which the MRL is 2.0 ppb.

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ATTACHMENT B

# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 12 Jan 96 Subject DENSITY OF LINER MATERIALS Sheet No. 1 of 5

Chkd. By TLP Date 1/31/96 Proj. No. 95-284

PURPOSE

Calculate the wet density of geosynthetic clay liner (GCL) and compacted clay liner (CCL).

APPROACH

Develop appropriate expression for density and use manufacturer specifications in calculation.

THEORY

$$\rho_{GCL} = \frac{M}{V}$$

$$= \frac{M}{AT}$$

where $\rho$ = density, wet $[ML^{-3}]$ (lb/ft$^3$)

M = mass of material $[M]$ (lb)

A = area, unit $[L^2]$ (ft$^2$)

T = thickness of material $[L]$ (ft)

Brackets [ ] give dimensions, where M = mass, L = length, and T = time.

Parentheses ( ) give example units.



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# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 12Jan95 Subject DENSITY OF LINER MATERIALS Sheet No. 2 of 5
Chkd. By RLP Date 1/31/96 Proj. No. 95-284

Using Claymax specifications on next sheet, we get:

$$\rho_{GCL} = \frac{0.95\ lb}{1\ ft^2} \frac{12\ in}{0.17\ in} \frac{}{1\ ft}$$

$$\rho_{GCL} = 67\ lb/ft^3$$

For CCL, attached data on compacted clay from El Sobrante Landfill give

- dry density $\rho_d = 110.2\ lb/ft^3$
- geotechnical moisture content $\theta_{mG} = 0.156$

Wet density $\rho = \rho_d (1 + \theta_{mG})$

$$= 110.2 \frac{lb}{ft^3} (1 + 0.156)$$

$$\rho_{CCL} = 127\ lb/ft^3$$



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By: [illegible] 12 Jan 96
Ckd by: TLP 1/31/96



## CLAYMAX GEOSYNTHETIC CLAY LINER MATERIAL SPECIFICATIONS

| | PROPERTY | TEST METHOD * | UNITS | CLAYMAX STYLE 200R | CLAYMAX STYLE 500 SP |
|---|---|---|---|---|---|
| BENTONITE PROPERTIES (1) | SODIUM MONTMORILLONITE CONTENT | X-RAY DIFFRACTION | % | 90 (TYP) | 90 (TYP) |
| | FREE SWELL | USP-NF-XVII | ml | 27 (MARV) | 27 (MARV) |
| | FLUID LOSS | API 13 B | ml | 12 (MAX. A.R.V.) | 12 (MAX. A.R.V.) |
| | MOISTURE CONTENT (3) | ASTM D4643 | % | 20 (TYP) | 20 (TYP) |
| ADHESIVE | ADHESION | VISUAL | | CONTINUOUS ADHESION TO BACKING MATERIAL | CONTINUOUS ADHESION TO BACKING MATERIAL |
| PHYSICAL PROPERTIES | THICKNESS (excluding fabrics) | ASTM D1777 | inches | 0.17 (MARV) | 0.17 (MARV) |
| | COMPOSITE THICKNESS | ASTM D1777 | inches | 0.20 (MARV) | 0.20 (MARV) |
| | WIDE WIDTH TENSILE | ASTM D4595 | PPI | 60 (TYP) (2) | 100 (TYP) |
| | GRAB TENSILE | ASTM D4632 | LB | 90 (MARV) (2) | 90 (MARV) (2) |
| | BENTONITE CONTENT (3) @ 20% moisture | WEIGH 12" X ROLL WIDTH | LB/SF | 0.95 (MARV) | 0.95 (MARV) |
| | SHEAR RESISTANCE | ASTM D35.01.81.07 (DRAFT) | | | |
| | HYDRATED | | DEG | >10 | >40 |
| | DRY | | DEG | >35 | >40 |
| HYDRAULIC PROPERTIES | PERMEABILITY | | | | |
| | A) 2 psi EFFECTIVE STRESS | ASTM D5084 | CM/S | $5 \times 10^{-9}$ (MAX.A.R.V.) | $5 \times 10^{-9}$ (MAX.A.R.V.) |
| | B) 30 psi EFFECTIVE STRESS | ASTM D5084 | CM/S | $< 5 \times 10^{-10}$ (TYP) | $< 5 \times 10^{-10}$ (TYP) |
| | PERMEABILITY (2 psi effective stress) | | | | |
| | C) 2" OVERLAPPED CLAYMAX (without the use of granular bentonite between the seams) | ASTM D5084 | CM/S | $< 5 \times 10^{-9}$ (TYP) | $< 5 \times 10^{-9}$ (TYP) |
| | D) DAMAGED CLAYMAX (3 each, 1" holes) | ASTM D5084 | CM/S | $< 5 \times 10^{-9}$ (TYP) | N/A (4) |
| | CLAYMAX underneath damaged HDPE geomembrane (1" hole) | ASTM D5084 | CM/S | $< 5 \times 10^{-9}$ (TYP) | N/A (4) |
| | F) AFTER 3 WET/DRY CYCLES | ASTM D5084 | CM/S | $< 5 \times 10^{-9}$ (TYP) | N/A (4) |
| | G) AFTER 5 FRZ/THAW CYCLES | ASTM D5084 | CM/S | $< 5 \times 10^{-9}$ (TYP) | N/A (4) |

** See opposite page for footnotes.

By: [illegible] 12 Jan 96
Ckd by: TL 1/31/96



# ENVIRONMENTAL SOLUTIONS, INC.
# COMPACTION CHARACTERISTICS DETERMINATION

| | | | |
|---|---|---|---|
| PROJECT NAME | EL SOBRANTE LANDFILL | TESTED BY | WIN AUNG |
| PROJECT NUMBER | 94-116 | DATE | 9/14/95 |
| SAMPLE NUMBER | CLSP-4 (CHECK POINT) | CHECKED BY | |
| SOIL DESCRIPTION | LEAN CLAY WITH SAND(CL),ASTM D2487. | DATE | |

| | | | | | |
|---|---|---|---|---|---|
| TEST STANDARD | ASTM D1557 | PROCEDURE | A | SIZE OF MOLD | 4 in. |

## WET DENSITY / DRY DENSITY DETERMINATION

| TEST NUMBER | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| WT. OF WET SOIL+MOLD (g) | 6116 | | | | |
| WT. OF MOLD (g) | 4267 | | | | |
| WT. OF WET SOIL (g) | 1849 | | | | |
| FACTOR | 0.06614 | | | | |
| WET DENSITY (lbf/cu.ft) | 122.3 | | | | |
| DRY DENSITY (lbf/cu.ft) | 109.0 | | | | |

## MOISTURE DETERMINATION

| | | | | | |
|---|---|---|---|---|---|
| TARE NUMBER | G | | | | |
| WT. OF WET SOIL+TARE (g) | 239.4 | | | | |
| WT. OF DRY SOIL+TARE (g) | 225.5 | | | | |
| WT. OF TARE (g) | 111.9 | | | | |
| WT. OF WATER (g) | 13.9 | | | | |
| WT. OF DRY SOIL (g) | 113.6 | | | | |
| MOISTURE CONTENT (%) | 12.2 | | | | |

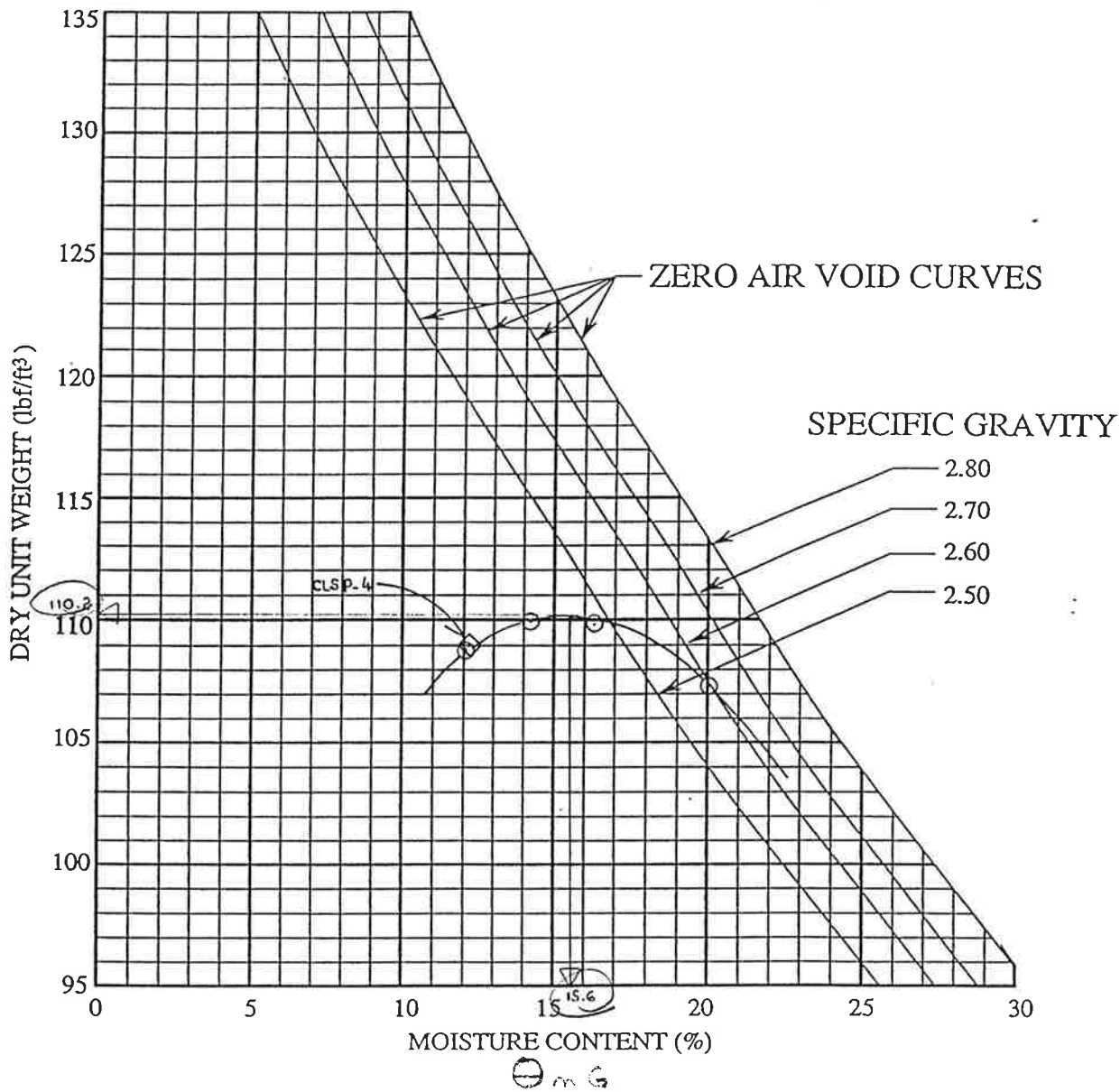
## RESULT FROM COMPACTION CURVE

| | | |
|---|---|---|
| MAXIMUM DRY DENSITY (lbf/cu.ft) | $\rho_d$ | 110.2 |
| OPTIMUM MOISTURE CONTENT (%) | $\theta_{m\,G}$ | 15.6 |

BY: [illegible] 12 Jan 96
Ckd by: TZP 1/31/96



CLSP-1




| MAXIMUM DRY DENSITY (lbf/ft³) | OPTIMUM MOISTURE CONTENT (%) |
|---|---|
| 110.2 | 15.6 |

Exploratory Boring and
Monitoring Well Installation
Program at the US Ecology RCRA
Facility in Beatty, Nevada
April 7, 1989
88-02107.18

THE MARK GROUP
ENGINEERS & GEOLOGISTS, INC.

April 7, 1989

Mr. Hal O. Hackett
US Ecology
P.O. Box 7246
9200 Shelbyville Road, Suite 300
Louisville, Kentucky 40207

88-2107.1

Subject: Transmittal of Final Report, Beatty, Nevada RCRA Facility

Dear Hal:

Enclosed are ten copies of our final report titled "Exploratory Boring and Monitoring Well Installation Program at the US Ecology RCRA Facility in Beatty, Nevada." This project was conducted in accordance with your Purchase Order number 21205.

The final report includes all information presented in our Progress Report of September 23, 1988 in addition to procedures followed, data collected, and results through project completion for the installation of the final six monitoring wells.

We trust the report provides the needed information for the Trench 11 and Trench 10 U.S. EPA Part B permit requirements. If there are any questions, please do not hesitate to contact us.

Thank you for the opportunity to provide this service for you.

Sincerely,

The MARK Group
Engineers & Geologists, Inc.

Steve J. Nacht
Director of Waste Technology

Reviewed and Approved:

Robert F. Kaufmann
Principal

SJN:tg
Enclosure

2300 Paseo Del Prado, Suite D-108 • Las Vegas, NV 89102 • (702) 364-1066

with varying degrees of cementation were found from near the static water level to a depth of approximately 390 feet. Below this depth, sandy and cobbly gravels were predominant.

### 5.1.2 Moisture Content

All moisture content results were obtained using ASTM Method 2216-80. On-site laboratory analysis (performed by a geotechnical laboratory contractor or MARK personnel) was deemed the most accurate because bulk and undisturbed samples that were sent to an off-site laboratory may not have been analyzed for up to two weeks. In general, moisture content above the water table fluctuated between 6 and 10 percent by weight. Water content tended to be lower in the coarser-grained samples from Borings 402C and higher in the finer-grained samples from Boring 403. Several samples below the first water were analyzed on-site because they appeared to have low moisture content; however, the moisture content was unexpectedly high (40-55 percent).

### 5.1.3 Cation Exchange Capacity

Cation exchange capacity was determined for that portion of a sample less than 2 millimeters in size. Results generally ranged between either 16 and 21 meq/100 grams or greater than 35 meq/100 grams. As expected, the lower range was found in sand and gravel samples. The higher range was found in samples of high clay content, which was usually below the water table but was also found above the water table in some thin zones of clayey gravelly sands and in the finer-grained materials of Boring 403.

### 5.1.4 Bulk Density

Bulk density results show an average wet density of 108 pounds per cubic foot (pcf) and an average dry density of 91 pcf. In general, wet density results appear fairly consistent in samples taken above and below the first water-bearing zone, and dry density results decrease below the first water-bearing zone. Both the wet and dry bulk density of sample B-401-34-374, which was taken from the sandy cobbly gravels of the lower unit, were 30 to 40 percent higher than the average. This indicates that the bulk density may be higher in that unit; however, no other samples could be retrieved because of sample refusal.

### 5.1.5 Falling Head Permeability

Falling head permeability test results show permeabilities ranging between $4.65 \times 10^{-4}$ and $2.54 \times 10^{-8}$ cm/sec.

### 5.1.6 Mineralogy

Mineralogy was analyzed by an independent off-site laboratory using petrographic microscopy and X-ray Diffraction-Energy Dispersive Analysis (XRD-EDS). Representative portions of the samples were screened over a 200 mesh sieve. Samples of the +200 material were prepared as thin sections and examined with a petrographic microscope to determine the composition of the rock and minerals present.

XRD is used to confirm the presence of the minerals that were identified using the petrographic microscope for the less than 200 mesh fraction of the sample. XRD will not

TABLE 3-2: EXPLORATORY BORING SAMPLING PLAN

UPPER VADOSE ZONE

| | Sample Type | | Sample Analysis | | | | | |
|---|---|---|---|---|---|---|---|---|
| Depth (ft) | Bulk | Undisturbed | Grain Size | Moisture Content | Cation Exchange Capacity | Moisture Tension Curves | Mineralogy | Bulk Density |
| 20 | X | | X | X | | | | |
| 40 | X | X | X | X | | X | | X |
| 60 | X | | X | X | X | | | |
| 80 | X | X | X | X | X | X | | X |
| 100 | X | | X | X | X | | | |
| 120 | X | X | X | X | X | X | X | X |
| 140 | X | | X | X | | | | |
| 160 | X | X | X | X | X | X | | X |
| 180 | X | | X | X | | | | |
| 200 | X | X | X | X | X | X | X | X |
| 220 | X | | X | X | | | | |
| 240 | X | X | X | X | X | X | | X |

Continue this sampling schedule in vadose zone until major lithologic changes are encountered (e.g. extensive playa deposition previously encountered at 240 feet).

X - Sample

TABLE 3-2 (C .TINUED)

LOWER VADOSE ZONE

| | Sample Type | | Sample Analysis | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Depth (ft) | Bulk | Undisturbed | Grain Size | Moisture Content | Cation Exchange Capacity | Moisture Tension Curves | Mineralogy | Bulk Density | Falling Head Permeability* |
| 250 | X | X | X | X | | | X | X | |
| 260 | X | X | X | X | X | X | | X | X |
| 270 | X | X | X | X | | | | X | |
| 280 | X | X | X | X | X | X | | X | X |
| 290 | X | X | X | X | | | | X | |
| 300 | X | X | X | X | X | X | X | X | X |

Continue this sampling schedule until the upper surface of the saturated zone is identified

X - sample

*Falling head permeability testing if appropriate materials are encountered

ATTACHMENT C

EPA/600/R-94/168b
September 1994

# THE HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE (HELP) MODEL

## *ENGINEERING DOCUMENTATION FOR VERSION 3*

by

Paul R. Schroeder, Tamsen S. Dozier, Paul A. Zappi,
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Interagency Agreement No. DW21931425

Project Officer

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OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

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TABLE 1. DEFAULT LOW DENSITY SOIL CHARACTERISTICS

| Soil Texture Class | | | Total Porosity vol/vol | Field Capacity vol/vol | Wilting Point vol/vol | Saturated Hydraulic Conductivity cm/sec |
|---|---|---|---|---|---|---|
| HELP | USDA | USCS | | | | |
| 1 | CoS | SP | 0.417 | 0.045 | 0.018 | $1.0 \times 10^{-2}$ |
| 2 | S | SW | 0.437 | 0.062 | 0.024 | $5.8 \times 10^{-3}$ |
| 3 | FS | SW | 0.457 | 0.083 | 0.033 | $3.1 \times 10^{-3}$ |
| 4 | LS | SM | 0.437 | 0.105 | 0.047 | $1.7 \times 10^{-3}$ |
| 5 | LFS | SM | 0.457 | 0.131 | 0.058 | $1.0 \times 10^{-3}$ |
| 6 | SL | SM | 0.453 | 0.190 | 0.085 | $7.2 \times 10^{-4}$ |
| 7 | FSL | SM | 0.473 | 0.222 | 0.104 | $5.2 \times 10^{-4}$ |
| 8 | L | ML | 0.463 | 0.232 | 0.116 | $3.7 \times 10^{-4}$ |
| 9 | SiL | ML | 0.501 | 0.284 | 0.135 | $1.9 \times 10^{-4}$ |
| 10 | SCL | SC | 0.398 | 0.244 | 0.136 | $1.2 \times 10^{-4}$ |
| 11 | CL | CL | 0.464 | 0.310 | 0.187 | $6.4 \times 10^{-5}$ |
| 12 | SiCL | CL | 0.471 | 0.342 | 0.210 | $4.2 \times 10^{-5}$ |
| 13 | SC | SC | 0.430 | 0.321 | 0.221 | $3.3 \times 10^{-5}$ |
| 14 | SiC | CH | 0.479 | 0.371 | 0.251 | $2.5 \times 10^{-5}$ |
| 15 | C | CH | 0.475 | 0.378 | 0.251 | $2.5 \times 10^{-5}$ |
| 21 | G | GP | 0.397 | 0.032 | 0.013 | $3.0 \times 10^{-1}$ |

$a$ = constant representing the effects of various fluid constants and gravity, 21 $cm^3/sec$

$\phi$ = total porosity, vol/vol

$\theta_r$ = residual volumetric water content, vol/vol

$\psi_b$ = bubbling pressure, cm

$\lambda$ = pore-size distribution index, dimensionless

A more detailed explanation of Equation 11 can be found in Appendix A of the HELP program Version 3 User's Guide and the cited references.

ATTACHMENT C

TABLE 2. MODERATE AND HIGH DENSITY DEFAULT SOILS

| Soil Texture Class | | | Total Porosity vol/vol | Field Capacity vol/vol | Wilting Point vol/vol | Saturated Hydraulic Conductivity cm/sec |
|---|---|---|---|---|---|---|
| HELP | USDA | USCS | | | | |
| 22 | L (Moderate) | ML | 0.419 | 0.307 | 0.180 | $1.9 \times 10^{-5}$ |
| 23 | SiL (Moderate) | ML | 0.461 | 0.360 | 0.203 | $9.0 \times 10^{-6}$ |
| 24 | SCL (Moderate) | SC | 0.365 | 0.305 | 0.202 | $2.7 \times 10^{-6}$ |
| 25 | CL (Moderate) | CL | 0.437 | 0.373 | 0.266 | $3.6 \times 10^{-6}$ |
| 26 | SiCL (Moderate) | CL | 0.445 | 0.393 | 0.277 | $1.9 \times 10^{-6}$ |
| 27 | SC (Moderate) | SC | 0.400 | 0.366 | 0.288 | $7.8 \times 10^{-7}$ |
| 28 | SiC (Moderate) | CH | 0.452 | 0.411 | 0.311 | $1.2 \times 10^{-6}$ |
| 29 | C (Moderate) | CH | 0.451 | 0.419 | 0.332 | $6.8 \times 10^{-7}$ |
| 16 | Liner Soil (High) | | 0.427 | 0.418 | 0.367 | $1.0 \times 10^{-7}$ |
| 17 | Bentonite (High) | | 0.750 | 0.747 | 0.400 | $3.0 \times 10^{-9}$ |

### 3.5.2 Default Waste Characteristics

Table 4 provides a summary of default moisture retention values for various waste layers. Municipal waste properties provided in Tchobanoglous et al. (1977) and Equations 6 and 7 were used to determine the total porosity, field capacity, and wilting point of a well compacted municipal waste. The field capacity and wilting point were calculated using Tchobanoglous et al.'s high and low water content values, respectively. Oweis et al. (1990) provided information on the in-situ saturated hydraulic conductivity of municipal waste. Zeiss and Major (1993) described the moisture flow through

ATTACHMENT D

Volume II
Solvents

# Handbook of Environmental FATE and EXPOSURE DATA For Organic Chemicals

Editor
Philip H. Howard

Associate Editors
Gloria W. Sage
William F. Jarvis
D. Anthony Gray

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The
Corporatio
authors of
Substances
chapters in
volume, is
responsible

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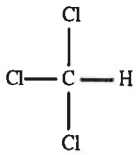
ATTACHMENT D (continued)

# Chloroform

## SUBSTANCE IDENTIFICATION

**Synonyms:** Trichloromethane

**Structure:**




**CAS Registry Number:** 67-66-3

**Molecular Formula:** $CHCl_3$

**Wiswesser Line Notation:** GYGG

## CHEMICAL AND PHYSICAL PROPERTIES

**Boiling Point:** 61.7 °C at 760 mm Hg

**Melting Point:** -63.5 °C

**Molecular Weight:** 119.39

**Dissociation Constants:**

**Log Octanol/Water Partition Coefficient:** 1.97 [25]

**Water Solubility:** 7950 mg/L [44]

**Vapor Pressure:** 246 mm Hg at 25 °C [22]

**Henry's Law Constant:** $4.35 \times 10^{-3}$ atm-m$^3$/mole at 25 °C [30]

## ENVIRONMENTAL FATE/EXPOSURE POTENTIAL

**Summary:** Chloroform is likely to enter the environment associated with its use as an industrial solvent, extractant, and chemical

ATTACHMENT E

**TECHNICAL REPORT DATA**
*(Please read Instructions on the reverse before completi*

| | | |
|---|---|---|
| 1. REPORT NO. EPA/600/R-93/171 | 2. | 3. PB94-114691 |
| 4. TITLE AND SUBTITLE Report of Workshop on Geosynthetic Clay Liners | | 5. REPORT DATE August 1993 |
| | | 6. PERFORMING ORGANIZATION CODE |
| 7. AUTHOR(S) David E. Daniel and B. Tom Boardman | | 8. PERFORMING ORGANIZATION REPORT NO. |
| 9. PERFORMING ORGANIZATION NAME AND ADDRESS Department of Civil Engineering, University of Texas at Austin, Austin, TX 78712 | | 10. PROGRAM ELEMENT NO. |
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| 12. SPONSORING AGENCY NAME AND ADDRESS Risk Reduction Engineering Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, Cincinnati, OH 45268 | | 13. TYPE OF REPORT AND PERIOD COVERED project report |
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15. SUPPLEMENTARY NOTES

Robert Landreth 513/569-7871

16. ABSTRACT

A workshop was held in Cincinnati, Ohio, to discuss the most recent information on the use of Geosynthetic Clay Liners (GCL's). Manufacturers discussed technical developments, recent research results, quality control and compared GCL's to compacted clay liners (CCL's). There are no standard test methods and no standard method was agreed upon. A discussion of performance indicated that GCL's provided low hydraulic conductivity, but more information is needed to quantify geomembrane and GCL composite behavior, and to confirm recent research at the University of Texas and at Drexel University, which indicated that most GCL's tested maintained low hydraulic conductivity even when subjected to large differential settlement.

17. KEY WORDS AND DOCUMENT ANALYSIS

| a. DESCRIPTORS | b. IDENTIFIERS/OPEN ENDED TERMS | c. COSATI Field/Group |
|---|---|---|
| Lining<br>Bentonite<br>Clays<br>Tensile Strength<br>Sheer Strength | Geosynthetic<br>Clay Liners<br>Geotextile<br>Geomembrane | |

| | | |
|---|---|---|
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# CHAPTER 7
# EQUIVALENCY

## 7.1 Equivalency (By David Daniel, University of Texas)

### 7.1.1 Potential Applications

If one wants to substitute a geosynthetic clay liner (GCL) for a required compacted clay liner (CCL), one will generally have to demonstrate that the proposed GCL will provide equivalent or better performance to a CCL. Equivalency analyses may be required for:

- Final Cover Systems:
  - Single GCL Versus Single CCL Liner
  - Geomembrane/GCL Composite Liner Versus Geomembrane/CCL Composite Liner

- Single Liner Systems:
  - Single GCL Versus Single CCL Liner
  - Geomembrane/GCL Composite Liner Versus Geomembrane/CCL Composite Liner

- Double Liner Systems:
  - Geomembrane/GCL Composite Liner Versus Geomembrane/CCL Composite Liner in Primary Liner
  - Geomembrane/GCL Composite Liner Versus Geomembrane/CCL Composite Liner in Secondary Liner

The Solid Waste Disposal Facility Criteria found at 40 CFR Part 258 apply to municipal solid waste landfills and treat the area of "equivalency" differently for final covers and liners. Final cover systems are to be designed to minimize infiltration and erosion, therefore, designs other than the minimum requirements of §258.60 (a) could be approved using an "equivalency" demonstration. However, alternatives to the composite liner design in §258.40 (a) (2) are not approved based on "equivalency" demonstrations but must meet the performance standard at §258.40 (a) (1). Different standards apply to facilities

that receive hazardous waste regulated under Subtitle C of RCRA and to CERCLA clean-up sites.

The designs for Subtitle C and CERCLA are evaluated based on the site-specific design. Innovative use of modern materials is encouraged, providing they meet the requirements of the law.

### 7.1.2 Differences Between CCLs and GCLs

Some of the differences between compacted clay liners and geosynthetic clay liners are listed in Table 7.1. Some of the potentially important (depending upon specific application) relative advantages of CCLs and GCLs may be summarized as follows:

- Advantages of compacted clay liners (CCLs):
    - The large thickness of CCLs makes them virtually puncture proof
    - The large thickness of CCLs makes them relatively insensitive to small imperfections in any one lift
    - The large thickness of CCLs gives them substantial capacity for adsorption of leachate
    - The large thickness of CCLs delays the discharge of water and solutes from the base of liners
    - There is a long history of use of CCLs
    - Intimate hydraulic contact with a geomembrane is not an issue for CCLs
    - Many regulatory agencies require CCLs; use of another type of liner may require demonstration of equivalency to a CCL.
    - A CCL is a logical choice if suitable clay is available locally
    - Testing procedures are reasonably well established for CCLs.

- Advantages of geosynthetic clay liners (GCLs):
    - Small thickness of GCLs leads to low consumption of space
    - Construction of GCLs is rapid and simple
    - Heavy equipment is not needed to install a GCL, which is beneficial if the GCL is underlain by a geosynthetic material

Table 7.1 Differences Between GCLs and CCLs

| Characteristic | Geosynthetic Clay Liner | Compacted Clay Liner |
|---|---|---|
| Materials | Bentonite Clay, Adhesives, Geotextiles, and Geomembranes | Native Soils or Blend of Soil and Bentonite |
| Construction | Manufactured and Then Installed in the Field | Constructed in the Field |
| Thickness | Approximately 10 mm | Approximately 0.5 to 1.0 m |
| Hydraulic Conductivity of Clay | $10^{-10}$ to $10^{-8}$ cm/s (Typical) | $10^{-8}$ to $10^{-7}$ cm/s (Typical) |
| Speed and Ease Construction | Rapid, Simple Installation | Slow, Complicated Construction |
| Water Content at Time of Construction | Essentially Dry; Cannot Desiccate During Construction and Produces No Consolidation Water | Nearly Saturated; Can Desiccate and Can Produce Consolidation Water |
| Cost | $5 to $11 per Square Meter | Highly Variable (Estimated Range: $8 to $32 per Square Meter) |
| Experience Level | Limited Due to Newness | Has Been Used for Many Decades |

- Some inclement weather delays (e.g., freezing temperatures) that stop construction of CCLs are not experienced with GCLs
- Because a GCL is a manufactured material, a consistent and uniform material can be produced
- Because GCLs are manufactured materials many of the specialized performance properties can be determined and need not be repeatedly re-determined
- GCLs can accommodate large differential settlement
- Quality assurance is simpler for a GCL compared to a CCL
- GCLs are more easily repaired than CCLs
- GCLs can probably better withstand freeze/thaw and wet/dry cycles than CCLs
- Unlike CCLs, GCLs are not vulnerable to desiccation damage during construction

7.1.3 Criteria for Equivalency

Three issues should be addressed when one compares a GCL to a CCL and considers the equivalency of a GCL to a CCL:

1. Hydraulic issues
2. Physical/mechanical issues
3. Construction issues

The specific issues that might have to be addressed for a specific site are listed in Table 7.2.

7.1.4 Hydraulic Issues

Hydraulic issues are the easiest to quantify. The criteria are discussed separately.

7.1.4.1 Steady Flux of Water

Flux of water is usually assessed by comparing the long-term, steady state water flux for the CCL and GCL. The flux of water (v) through an individual layer of porous material is defined from Darcy's Law as:

Table 7.2 Potential Equivalency Issues

| Category | Criterion for Evaluation | Relevant for: Liners | Relevant for: Covers |
|---|---|---|---|
| Hydraulic Issues | Steady Flux of Water | X | X |
| | Steady Solute Flux | X | |
| | Adsorption Capacity | X | |
| | Breakout Time: | | |
| | -Water | X | X |
| | -Solute | X | |
| | Production of Consolidation Water | X | X |
| Physical/ Mechanical Issues | Freeze-Thaw | X* | X |
| | Wet-Dry | | X |
| | Total Settlement | X** | X |
| | Differential Settlement | X** | X |
| | Slope Stability | X*** | X |
| | Erosion | | X |
| | Bearing Capacity | X | |
| Construction Issues | Puncture Resistance | X | X |
| | Subgrade Condition | X | X |
| | Ease of Placement | X | X |
| | Speed of Construction | X | X |
| | Availability of Materials | X | X |
| | Weather Constraints | X | X |
| | Quality Assurance | X | X |

Notes:

* Relevant only until liner is covered sufficiently to prevent freezing
** Settlement of liners usually of concern only in certain circumstances, e.g., vertical expansions
*** Stability of liner may not be relevant after filling (except canyon landfills)

$$v = k \frac{H+L}{L} \tag{7.1}$$

where k is the hydraulic conductivity, H is the depth of liquid ponded on the liner, and L is the thickness of the liner. The water pressure on the base of the liner is assumed to be zero in Eq. 7.1.

For a GCL, Eq. 7.1 is applicable only for flow through the bentonite component; if the GCL contains a geomembrane, water flux will be controlled by water vapor diffusion through the geomembrane component. The geomembrane component, if present, should be considered in the equivalency analysis and in computation of water flux. Also, Eq. 7.1 applies to a CCL or GCL liner alone; composite action with a geomembrane is considered later.

The flux ratio for water, $F_W$, is defined as:

$$F_W = \frac{v_{GCL}}{v_{CCL}} \tag{7.2}$$

or:

$$F_W = \frac{k_{GCL} \dfrac{H+L_{GCL}}{L_{GCL}}}{k_{CCL} \dfrac{H+L_{CCL}}{L_{CCL}}} \tag{7.3}$$

For example, for a GCL without a geomembrane component, if:

$k_{GCL} = 1 \times 10^{-9}$ cm/s $= 1 \times 10^{-11}$ m/s
H = 0.3 m (1 ft)
$L_{GCL}$ = 7 mm = 0.007 m
$K_{CCL} = 1 \times 10^{-7}$ cm/s $= 1 \times 10^{-9}$ m/s
$L_{CCL}$ = 0.9 m (3 ft)

then $F_W$ from Eq. 7.3 equals 0.3. So long as $F_W \leq 1$, equivalency in terms of water flux is demonstrated, i.e., the rate of water flow through the GCL is less than or equal to that through the CCL. Most GCLs can be shown to be equivalent to a CCL that has a hydraulic conductivity of $1 \times 10^{-7}$ cm/s in terms of steady water

flux. If the GCL contains a geomembrane, the flux ratio will be even less than that computed from Eq. 7.3.

A composite liner consists of a geomembrane placed in contact with a low-permeability soil. A geomembrane/GCL composite may be considered as an alternate to a geomembrane/CCL composite. If so, flow through the composite should be analyzed. Flow through a flaw in a geomembrane in a composite liner depends on the hydraulic conductivity of the clay component, the hydraulic gradient across the clay component, the hydraulic contact between the geomembrane and the clay component, and the presence of a geomembrane within the GCL. No equations have been published for computing flow rates through a defect in a geomembrane component of a geomembrane/GCL composite liner. However, published information can be used to make comparative estimates. Equivalency evaluations would clearly be product and perhaps site specific.

7.1.4.2 Steady Solute Flux

The maximum flux is the steady-state flux. Long-term, steady solute flux, which is relevant only for liners, may be analyzed on the basis of advection alone, diffusion alone, or advection plus diffusion. As will be seen later, the assumptions necessary to analyze steady diffusion and steady advection-plus-diffusion are inconsistent with the processes themselves, and only the case of advection is relevant for steady-state conditions. Nevertheless, for completeness, the methods for analyzing steady diffusion and steady diffusion-plus-advection are presented so that these processes can be understood.

It is assumed that the concentration of a solute of concern in the leachate remains constant. The advective mass flux, $v_{m,A}$, is:

$$v_{m,A} = c_{leachate}\, k \frac{H + L}{L} \tag{7.4}$$

where $c_{leachate}$ is the concentration of the solute of interest in the leachate. The advective mass flux ratio, $F_{m,A}$ is defined as:

$$F_{m,A} = \frac{v_{m\,A(GCL)}}{v_{m\,A(CCL)}} \tag{7.5}$$

or:

$$F_{m,A} = \frac{c_{leachate}\, k_{GCL} \dfrac{H + L_{GCL}}{L_{GCL}}}{c_{leachate}\, k_{CCL} \dfrac{H + L_{CCL}}{L_{CCL}}} \tag{7.6}$$

or:

$$F_{m,A} = \frac{k_{GCL} \dfrac{H+L_{GCL}}{L_{GCL}}}{k_{CCL} \dfrac{H+L_{CCL}}{L_{CCL}}} = F_w \tag{7.7}$$

It is noted that $c_{leachate}$ cancels out of Eq. 7.6. Because $F_{m,A} = F_w$ (Eq. 7.7), if one has demonstrated equivalency of steady water flux, one has also demonstrated equivalency of steady mass flux of solute caused by advection.

Solutes in leachate can also migrate through clay liners by molecular diffusion. Steady diffusion of solutes is usually analyzed with Fick's first law, which states that:

$$v_D = D\ \theta\ \frac{\Delta c}{L} \tag{7.8}$$

where $v_D$ is the diffusive mass flux, D is the diffusion coefficient for the solute of interest, $\theta$ is the volumetric water content, $\Delta c$ is the difference in concentration of the solute between the top and bottom of the liner, and L is the thickness of the liner.

The diffusive mass flux ratio ($F_{m,D}$) is defined as

$$F_{m,D} = \frac{F_{m\,D(GCL)}}{F_{m\,D(CCL)}} \tag{7.9}$$

or:

$$F_{m,D} = \frac{D_{GCL}\, \theta_{GCL} \dfrac{\Delta c}{L_{GCL}}}{D_{CCL}\, \theta_{CCL} \dfrac{\Delta c}{L_{CCL}}} \tag{7.10}$$

or:

$$F_{m,D} = \frac{D_{GCL}}{D_{CCL}} \frac{\theta_{GCL}}{\theta_{CCL}} \frac{L_{CCL}}{L_{GCL}} \tag{7.11}$$

Limited data exist on diffusion coefficients in clay liners. Data developed for compacted kaolinite at the University of Texas indicate that $D \approx 6 \times 10^{-10}$ $m^2/s$ for the non-reactive solute chlorine. For one GCL tested, the diffusion coefficient for the bentonite in the GCL was approximately $2 \times 10^{-10}$ $m^2/s$. If, for example, one assumes:

$$\frac{D_{GCL}}{D_{CCL}} = \frac{2 \times 10^{-10}}{6 \times 10^{-10}} = 0.33$$

$$\frac{\theta_{GCL}}{\theta_{CCL}} = \frac{0.6}{0.4} = 1.5$$

$$\frac{L_{CCL}}{L_{GCL}} = \frac{0.9 \text{ m}}{0.007 \text{ m}} = 129$$

then one computes a diffusive mass flux ratio of:

$$F_{m,D} = (0.33)\ (1.5)\ (129) = 64$$

In this example, the GCL is not equivalent to the CCL since there would be more diffusive mass flux through the GCL than CCL. In general the calculated steady, diffusive mass flux through the bentonite within the GCL is always expected to be greater than the steady, diffusive mass flux through the CCL. However, for those GCLs that have a geomembrane component, the geomembrane, which has an extremely low diffusion coefficient for most solutes, should be considered and will tend to greatly reduce the steady, diffusive mass flux.

As mentioned earlier, the assumptions necessary for computing steady diffusive flux are inconsistent with the process itself. The problem is that a contradiction exists in the boundary conditions. Diffusion is driven by a concentration gradient, $\Delta c$. Over time, the solute of interest in the leachate will

diffuse to the base of the liner, and the concentration at the base of the liner will eventually equal the concentration on top of the liner, i.e., the concentration in the leachate (which is assumed to be constant). Thus, the diffusion-driving concentration gradient becomes zero and diffusive transport ceases. The only way that steady diffusion could develop through a liner would be for fresh water to continually flush the underside of the liner to maintain a concentration gradient across the liner. For nearly all sites, the case of steady diffusion will be irrelevant and need not be considered.

It may be argued that neither advection alone nor diffusion alone is important -- solutes will migrate through soil liners by advection plus diffusion. The total mass flux due to advection plus diffusion ($v_{m,A+D}$) is generally assumed to be:

$$v_{m,A+D} = v_{m,A} + v_{m,D} \tag{7.12}$$

and the ratio of advective plus diffusive mass flux, $F_{m,A+D}$, may be defined as:

$$F_{m,A+D} = \frac{v_{m\ A+D\ (GCL)}}{v_{m\ A+D\ (CCL)}} \tag{7.13}$$

Although Eq. 7.13 can be applied, it should not be applied because the assumed conditions are physically impossible for long-term, steady conditions. If advection carries solutes downward through the liner, then at steady state the base of the liner must necessarily be saturated with leachate. If the base is saturated with leachate, then $\Delta c = 0$ and $v_{m,D} = 0$. Thus, when one analyzes long-term, steady mass flux of a solute through a GCL or CCL, only advective transport need generally be considered.

In some cases, one may wish to analyze transient conditions that lead up to steady conditions, in which case both advective and diffusive transport should be considered. If the GCL contains a geomembrane, the presence of the geomembrane should be taken into account.

7.1.4.3 Adsorption Capacity

The adsorption capacity of a clay liner may be relevant only for liners (not covers). Regulations generally have no specific adsorption requirements.

Adsorption of organics tends to be different from adsorption by inorganics. Adsorption of inorganics is controlled by cation exchange reactions and geochemical processes such as precipitation. Adsorption of organic solutes is generally assumed to be controlled by the amount of organic carbon in the soil and a partition coefficient for the solute (which is characterized by the octanol-water partition coefficient or water solubility of the organic species).

For inorganics, the maximum adsorbed mass per unit cross-sectional area of liner (C) resulting from cation exchange processes may be defined as follows:

$$C = CEC\, \rho_d\, L \tag{7.14}$$

where CEC is the cation exchange capacity (maximum mass of solute sorbed per unit mass of dry soil), $\rho_d$ is the dry mass density of the soil, and L is the thickness of the liner. The ratio of thickness of a typical GCL to a CCL is small (on the order of 0.01). Thus, in order for a GCL to have equivalent cation adsorption capacity to a CCL, the adsorption coefficient of the GCL would have to be at least 100 times that of the CCL.

The cation exchange capacity of bentonite clay is typically on the order of 100 to 150 meq/100g. Natural soil materials used to construct CCLs have typical CECs in the range of 3 to 30 meq/100g. The ratio of cation adsorption capacities, denoted $F_{CEC}$, is:

$$F_{CEC} = \frac{C_{GCL}}{C_{CCL}} = \frac{CEC_{GCL}}{CEC_{CCL}} \frac{\rho_{d\,GCL}}{\rho_{d\,CCL}} \frac{L_{GCL}}{L_{CCL}} \tag{7.15}$$

For the typical range of values, $F_{CEC}$ would be expected to be in the range of 0.03 to 0.75. It appears unlikely that equivalency can be demonstrated for cation adsorption capacity using the expressions just presented. However, it must be understood that adsorption of inorganic species is a complex process. Cation exchange is just one of several processes that can affect adsorption. Precipitation of inorganic solutes can be a far more important mechanism than cation exchange, and pH is often a dominant variable controlling precipitation processes in many geochemical environments. Thus, site-specific factors, and not just simple comparisons of CECs and relative soil masses, will often need to be considered when relative adsorption capacities are compared.

Non-polar organic solutes are sorbed by carbon present in the soil. The carbon content of bentonite in GCLs is capable of estimation, but CCLs will be highly variable in their organic carbon content. Although site-specific assessments would be required (due to variability of CCLs), equivalency of a GCL to a CCL probably cannot be demonstrated in terms of capacity to adsorb non-polar constituents in leachate to the bentonite because the mass of bentonite present in a GCL is far less than the mass of soil present in a CCL.

Adsorption is only relevant in the short term. When steady state mass transport is reached, adsorption capacity is exhausted. Equivalency in terms of adsorption, if evaluated at all, should be evaluated in terms of a specified performance period. For example, suppose the performance period being considered is 30 years. If the adsorption capacity of neither the CCL nor the GCL is exhausted after 30 years, both types of liner have "reserve" adsorption capacity and may be considered equivalent for the performance period. Alternatively, if either or both is exceeded, breakthrough of solute will occur and other issues, e.g., steady state solute flux, will require consideration.

7.1.4.4 Time to Initiate Discharge of Water from Base of Liner

GCLs are initially unsaturated with water whereas CCLs are often very close to saturation. When liquid first enters the upper surface of the liner, no liquid initially discharges from the base of the liner. The GCL might be compared to the CCL in terms of time to achieve discharge of water from the bottom of the liner. Again, for those GCLs that contain a geomembrane, the presence of the geomembrane should be taken into account.

The time to achieve discharge of water from the base is difficult to describe in general terms. For CCLs, the time depends greatly upon the hydraulic conductivity, initial water content, and tendency to swell. For GCLs, the time is usually fairly short (a few weeks), although for GCLs that contain a geomembrane, the time may be much greater. A comparison of time to initiate discharge of water from the base of the liner would have to be performed on a site specific basis.

The time to initiate discharge of water from the base of a liner is not relevant in the long term and often will not be relevant even in the short term. Most designs assume that water will be discharged from the base of a liner and do not make any assumptions about how long this process will take.

7.1.4.5 Breakthrough Time for Solute

The breakthrough time for a solute, which is not relevant for covers, is the time required for a solute to travel from the top to the bottom of a liner. However, theoretically, the time required for an infinitely small concentration of solute to breakthrough to the base of a liner is zero for a thick or thin liner. Thus, breakthrough time is not a uniquely defined parameter -- the time depends upon the concentration of interest. In this section, it is assumed that the breakthrough time for a GCL is compared to that of a CCL for the same concentration at the base of the clay liner.

Because of the thinness of GCLs, diffusion will generally cause the breakthrough time of a thin layer of bentonite to be less than for a CCL. Even for GCLs that contain a geomembrane, diffusion of organic solutes across the geomembrane tends to occur quickly, but at a very low mass flux. However, diffusion of inorganics through the geomembrane would be nil. Equivalency depends on the GCL and the chemicals of concern.

One must carefully consider whether the breakthrough time for solutes is relevant. In the long term, breakthrough time is irrelevant -- breakthrough will eventually occur in all liner systems with an outward gradient. The important long-term issue is solute flux.

Steady-state flux represents a worst-case scenario, i.e., largest mass flux. Time-dependent flux, before steady state, also may need to be considered in certain situations.

7.1.4.6 Production of Consolidation Water

When clayey soils are loaded, water tends to be slowly squeezed out of the soil via a process known as "consolidation." The production of consolidation may or may not be of any concern, depending upon site-specific conditions. Examples of potential problems associated with the production of consolidation water include reduced stability at the geomembrane/clay liner interface (the consolidation water from the clay liner tends to reduce stability through increased water pressure at the interface) and collection of liquids in a leak detection layer for double composite liners.

Compacted clay liners are nearly saturated with water at the time of construction. When CCLs are loaded, substantial quantities of water are often squeezed out of the liner. For example, if a 1-m-thick liner is saturated with

water and compresses just 2 percent of its original thickness due to consolidation, the amount of water squeezed out of the liner would be approximately 200,000 liters of water per hectare (20,000 gallons per acre).

Geosynthetic clay liners are essentially dry when they are constructed and cannot produce consolidation water unless they are first soaked with water and then compressed. Normally, GCLs do not have an opportunity to become saturated before they are loaded. However, a saturated GCL will produce far less consolidation water than a saturated CCL. Thus, GCLs are superior to CCLs in terms of minimizing production of consolidation water.

### 7.1.5 Physical/Mechanical Issues

The physical/mechanical issues that might be considered in an equivalency analysis include freeze/thaw effects, wet/dry effects, response to total settlement, response to differential settlement, stability on slopes, and vulnerability to erosion. Some issues are relevant for liners but all are relevant for covers (Table 7.2).

#### 7.1.5.1 Freeze/Thaw Resistance

Compacted clay liners are known to be vulnerable to large increases in hydraulic conductivity from freeze/thaw. Limited laboratory data indicate that GCLs do not undergo increases in hydraulic conductivity as a result of freeze/thaw (Shan and Daniel, 1991). In addition, for those GCLs that contain a geomembrane, the geomembrane is unaffected by freeze/thaw. Thus, from the available data, GCLs appear to be superior to CCLs in terms of freeze/thaw resistance.

#### 7.1.5.2 Wet/Dry Effects

Wetting and drying of CCLs and GCLs can cause either type of clay liner to swell or shrink. The main concern with clay liners that are wet and then dry out, is that desiccation can lead to cracking and an increase in hydraulic conductivity.

Limited laboratory data indicate that when dry, cracked CCLs are rewetted, the clay swells and any cracks are partially closed, leading to partial recovery of the original, low hydraulic conductivity. In contrast, the available data show that the high swelling of bentonite results in full self healing and full

recovery of the original, low hydraulic conductivity when dried, cracked GCLs are rewetted. In addition, for those GCLs that contain a geomembrane, the geomembrane is insensitive to wet/dry effects. Thus, GCLs appear to be more than equivalent to CCLs in terms of ability to self-heal if the material is wetted, dried, and then rewetted.

7.1.5.3 Response to Total Settlement

Total settlement refers to block-like settlement without significant bending or distortion. It is believed that GCLs and CCLs would respond similarly to total settlement.

7.1.5.4 Response to Differential Settlement

Recent research by LaGatta (1992) indicates that some GCLs maintain their low hydraulic conductivity even when subjected to large differential settlements. In all probability, GCLs are more resistant to damage from differential settlement than CCLs (LaGatta, 1992). For example, for a depression with a diameter of 1 m that subsides 0.5 m, the liner will undergo approximately 10% tensile strain. While the data discussed in LaGatta (1992) suggests that GCLs can function under such conditions, it is known that CCLs cannot and will suffer tension cracks at 1% tensile strain or less.

7.1.5.5 Stability on Slopes

The shear strength of GCLs is very sensitive to the water content and type of GCL (Shan and Daniel, 1991; and Daniel and Shan, 1992). Water-saturated GCLs that have adhesive-bonded bentonite have angles of internal friction for consolidated-drained conditions of approximately 10 degrees. Dry materials are 2 to 3 times as strong as water-saturated GCLs. Also, needle-punched and stitch-bonded GCLs tend to have high strengths. The shear strength of CCLs varies widely, depending on materials, water content, and compaction conditions.

In stability analyses, one often must consider not only internal shear failure but interfacial shear with an adjacent layer, e.g., a geomembrane. Also, shear strength may be of short-or long-term concern, or both. No general statement can be made about probable equivalency of a GCL to a CCL because the assessment depends on specific materials, the degree to which the bentonite

can wet, consolidation water generated by CCLs, the slope angle, and other site-specific conditions.

7.1.5.6 Vulnerability to Erosion

Erosion resistance may be of concern in final covers if inadequate cover soil is present. Because of the presence of erosion-resistant geosynthetic materials in GCLs, GCLs can potentially be more resistant to erosion than CCLs. However, if the clay liner is exposed to erosive forces, the bentonite may be washed out of some GCL materials. Thus, equivalency depends upon the specific materials being considered. For many sites, erosion will not be of any concern, e.g., for a GCL underlying a geomembrane or a GCL containing a geomembrane that overlies the bentonite.

7.1.5.7 Bearing Capacity

A clay liner must have adequate bearing capacity to support loads, e.g., wheel loads from construction equipment. The clay liner must not thin or pump clay into adjacent layers under static or dynamic (e.g., traffic) loads. Hydrated bentonite is not as strong as most materials used in constructing CCLs. Equivalency is heavily dependent upon site-specific conditions.

7.1.6 Construction Issues

The construction issues that might be considered in an equivalency analysis include puncture resistance, effect of subgrade condition on constructability, ease of placement, speed of construction, availability of materials, weather constraints, and quality assurance requirements.

7.1.6.1 Puncture Resistance

Thick CCLs cannot be accidentally punctured during construction, but thin GCLs can. Some GCLs have the capability of self-sealing around certain types of puncture objects, e.g., penetration of the GCL with a sharp object such as a nail. The swelling capacity of bentonite gives GCLs this self-healing capability.

The more significant type of puncture for a GCL would be puncture caused by construction equipment. For example, if the corner of the blade of a bulldozer accidentally punctures the GCL during spreading of cover material, the GCL would probably not self seal at the puncture. A thick CCL could not be

punctured by a piece of construction equipment. Thus, GCLs will not have equivalent puncture resistance to CCLs. However, quality assurance and quality control procedures can be established and implemented to make the probability of a puncture during construction extremely low. Ultimately, site-specific conditions and quality assurance procedures will determine whether puncture is a relevant issue that deserves serious consideration.

7.1.6.2 Effect of Subgrade Condition

Compacted clay liners are constructed with heavy equipment. If the subgrade is compressible (e.g, solid waste), the GCL, which can be installed with light-weight equipment, will be easier to construct. On the other hand, stones and rocks can puncture a GCL but not a CCL; if the subgrade contains stones or rocks, the integrity of the GCL may be compromised. Thus, equivalency of a GCL to a CCL in terms of the effect of subgrade depends on the condition of the subgrade and will have to be evaluated on a site-specific basis.

7.1.6.3 Ease of Placement or Construction

A GCL will always be easier to place than a CCL, unless weather conditions are adverse (e.g., constant rain), in which case even a CCL will also be difficult to construct. In general, GCLs are equivalent to or better than CCLs in terms of ease of placement or construction.

7.1.6.4 Speed of Construction

Geosynthetic clay liners can be placed much more quickly than CCLs. Equivalency is obvious.

7.1.6.5 Availability of Materials

Suitable clays for construction of a CCL may or may not be available locally, depending on the site. Because GCLs are a manufactured material, they are readily available and can be shipped to a site quickly. The cost of shipment is not a large percentage of the total cost of a GCL. Thus, GCLs will often be superior to CCLs in terms of availability of materials.

7.1.6.6 Weather Constraints

Compacted clay liners are difficult to construct when soils are wet, heavy precipitation is occurring, the weather is extremely dry (clay desiccates), the soil is frozen, or the temperature is below freezing. Geosynthetic clay liners are difficult to construct during precipitation. Weather constraints generally favor GCLs.

Some GCLs must be covered before they get wet. If a geomembrane will be placed over the GCL, the GCL must be covered almost immediately with the geomembrane. Additional weather constraints, e.g., wind speed, may apply to the geomembrane and, indirectly, influence the GCL. The fact that many GCLs must be covered before they are hydrated is a significant weather constraint for GCLs that does not exist for CCLs. However, CCLs have weather constraints, too: CCLs must not be allowed to freeze or desiccate (GCLs cannot desiccate during construction because they are dry, and dry GCLs are unaffected by freezing temperatures).

Equivalency in terms of weather constraints must be considered on a site-specific basis, but weather constraints generally favor GCLs over CCLs.

7.1.6.7 Quality Assurance Requirements

Quality assurance (QA) requirements are less extensive for GCLs compared to CCLs, but no less critical. There is no reason to suspect that QA is more difficult for a GCL than a CCL. However, testing procedures and observational techniques are well established for CCLs but are not for GCLs. The GCL industry and the Geosynthetic Research Institute (GRI) are working hard through GRI and ASTM to established testing methods. While it would appear that GCLs and CCLs are equivalent in terms of QA requirements, more work needs to be done to establish standard test methods for GCLs.

7.1.7 Summary of Equivalency Issues

Table 7.3 summarizes the preceding discussion of equivalency. Equivalency can be demonstrated generically in many categories. However, in two categories, equivalency probably cannot be demonstrated: (1) GCLs do not have adsorption capacity equivalent to CCLs; and (2) GCLs do not have the puncture resistance of CCLs. The adsorption capacity has no relevancy to covers.

Table 7.3 Summary of Equivalency Assessment

| | | Equivalency of GCL to CCL | | |
|---|---|---|---|---|
| Category | Criterion for Evaluation | Probably Equivalent | Probably Not Equivalent | Product or Site Specific |
| Hydraulic Issues | Steady Flux of Water | | | X |
| | Steady Solute Flux | | | X |
| | Adsorption Capacity | | X | |
| | Breakthrough Time: | | | |
| | -Water | | | X |
| | -Solute | | X | |
| | Consolidation Water | X | | |
| Physical/ Mechanical Issues | Freeze-Thaw | X | | |
| | Wet-Dry | X | | |
| | Total Settlement | X | | |
| | Differential Settlement | X | | |
| | Slope Stability | | | X |
| | Erosion | | | X |
| | Bearing Capacity | | | X |
| Construction Issues | Puncture Resistance | | X | |
| | Subgrade Condition | | | X |
| | Ease of Placement | X | | |
| | Speed of Construction | X | | |
| | Availability of Materials | X | | |
| | Weather Constraints | X | | |
| | Quality Assurance | X | | |

For liners, the issue of adsorption capacity may or may not be relevant, depending on project-specific details. Although thin GCLs can be punctured during construction, the problem is of more concern for liners (due to higher stresses), and careful QA may be capable of addressing this potential problem for both liners and covers.

As suggested by Table 7.3, many equivalency issues depend on the GCL product and the particular conditions unique to a given site. Equivalency will clearly have to be evaluated on a case-by-case basis.

## 7.2 Discussion

A brief discussion took place after the presentation. Comments were made that there is a large need to establish criteria for analysis of equivalency. The potential criteria are numerous, but many may not apply. Also, differences in liners and covers were emphasized and the special problems in analyzing geomembrane/clay liner composites were briefly discussed.

ATTACHMENT F

# THE SOIL CHEMISTRY OF HAZARDOUS MATERIALS

James Dragun, Ph.D.

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ENVIRONMENTAL SOLUTIONS, INC.
c.2

Hazardous Materials Control Research Institute
Silver Spring, Maryland

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Ackn

Prefa

1: C

2: T

3: E

4: E

5: A

6: A

7: O

8: O

9: O

Index

**TABLE 4.2** Ranges for $K_d$s for Various Elements in Soils and Clays[5] (Reprinted with permission from the American Society of Agronomy, Inc., Crop Science Society of America, Inc., and the Soil Science Society of America, Inc.)

| *Element* | *Observed Range (ml/g)* | | *Mean*[a] | | *Standard Deviation*[b] |
|---|---|---|---|---|---|
| Ag | 10 - 1,000 | | 4.7 | | 1.3 |
| Am | 1.0 - 47,230 | | 6.7 | | 3.0 |
| As(III) | 1.0 - 8.3 | | 1.2 | | 0.6 |
| As(V) | 1.9 - 18 | | 1.9 | | 0.5 |
| Ca | 1.2 - 9.8 | | 1.4 | | 0.8 |
| Cd | 1.3 - 27 | | 1.9 | | 0.9 |
| Ce | 58 - 6,000 | | 7.0 | | 1.3 |
| Cm | 93 - 51,900 | | 8.1 | | 1.9 |
| Co | 0.2 - 3,800 | | 4.0 | | 2.3 |
| Cr(III) | 470 - 150,000 | | 7.7 | | 1.2 |
| Cr(VI) | 1.2 - 1,800 | | 3.6 | | 2.2 |
| Cs | 10 - 52,000 | | 7.0 | | 1.9 |
| Cu | 1.4 - 333 | | 3.1 | | 1.1 |
| Fe | 1.4 - 1,000 | | 4.0 | | 1.7 |
| K | 2.0 - 9.0 | | 1.7 | | 0.5 |
| Mg | 1.6 - 13.5 | | 1.7 | | 0.5 |
| Mn | 0.2 - 10,000 | | 5.0 | | 2.7 |
| Mo | 0.4 - 400 | | 3.0 | | 2.1 |
| Np | 0.2 - 929 | | 2.4 | | 2.3 |
| Pb | 4.5 - 7,640 | | 4.6 | | 1.7 |
| Po | 196 - 1,063 | | 6.3 | | 0.7 |
| Pu | 11 - 300,000 | | 7.5 | | 2.3 |
| Ru | 48 - 1,000 | | 6.4 | | 1.0 |
| Se(IV) | 1.2 - 8.6 | | 1.0 | | 0.7 |
| Sr | 0.2 - 3,300 | | 3.3 | | 2.0 |
| Tc | 0.003 - 0.28 | | 3.4 | | 1.1 |
| Th | 2,000 - 510,000 | | 11.0 | | 1.5 |
| U | 11 - 4,400 | MIN | 3.8 | MAX | 1.3 |
| Zn | 0.1 - 8,000 | −1 | 2.8 | 3.9 | 1.9 |

[a] Mean of the logarithms of the observed values.
[b] Standard deviation of the logarithms of the observed values.

$\log K_d(Zn) = 2.8$
$K_d(Zn) = \log^{-1} 2.8$
$= 631 \text{ cm g}^{-1}$

For divalent cations, the order is generally[6]:

$$Ba^{2+} > Sr^{2+} > Ca^{2+} > Mg^{2+}$$

In general, trivalent cations are preferentially adsorbed over divalent cations, which are preferentially adsorbed over monovalent cations.

A further analysis of the information presented in Table 4.2 will reveal that the $K_d$s measured for an individual element in soils can vary substantially; for example, the ranges of $K_d$s for Pu and Zn span four orders of magnitude. Because the magnitude of the $K_d$ is dependent upon the size and charge of the cation and upon those soil properties governing the ex-

ATTACHMENT G



The valid use of an equation requires that predictions be made for soils possessing organic carbon contents that fall within the range of those organic carbon contents used to develop the equation.

Fourth, these equations do not take into account the potential contributions of mineral surfaces to organic chemical adsorption. It is most important to note that one should not assume that the contribution of the inorganic soil mineral surfaces to the accumulation of an organic chemical is negligible, and that the organic carbon content is the only governing factor. An analysis of the information presented in Table 6.1 revealed earlier that inorganic mineral surfaces can make a substantial contribution to the accumulation of organic chemicals at soil surfaces. Table 6.5 lists suggested threshold values for significant mineral surface contributions to organic chemical adsorption. If these threshold values are exceeded, then the $K_{oc}$ equations listed in Table 6.4 should not be utilized.

Fifth, these equations do not take into account the fact that soil organic matter varies in chemical structure and composition among soils. It is important to remember that although soil organic matter is usually considered to be a "homogeneous mass," in reality it is a heterogeneous mixture of organic chemicals. And, organic chemicals will be preferentially accumulated by the different organic chemicals which comprise soil organic matter. One excellent example of this phenomenon involved a study utilizing Russell, Chalmers, and Kokomo silt loams[73]; benzidine and alpha-naphthylamine were preferentially and primarily accumulated by humic acid, whereas p-toluidine was primarily accumulated by fulvic acid. This selective accumulation occurs for several reasons. The humic to fulvic acid ratio varies significantly among soils[74]. The aromatic hydrocarbon content and the ratio of polypeptide to polysaccharide products varies significantly among soils. Even the macromolecular structure of the humic fraction of soil organic matter can change as pH, ionic strength, and the concentration of humic material changes[75]. Organic matter is oxidized to varying extents in soil[12]; it generally has a reactivity with respect to water which is similar to that of montmorillonite[76].

Sixth, these equations do not take into account the fact that serious errors may arise by assuming a linear adsorption isotherm occurs when high concentrations of organic chemicals exist in soil systems. Linear adsorption isotherms have usually been assumed to exist for low chemical concentrations because this assumption simplifies computer simulation modeling. However, linear adsorption isotherms may not exist at higher organic chemical concentrations[77]. It is important to recognize that $K_{oc}$ equations are developed primarily at low organic chemical concentrations, and these equations may overestimate the $K_{oc}$ of organic chemicals present at high concentrations.

Source: Dragun (1988).

ATTACHMENT H

# Fundamentals of Soil Behavior

*James K. Mitchell*

**University of California, Berkeley**

**1976**

*John Wiley & Sons, Inc.*

**New York London Sydney Toronto**

ATTACHMENT H

## Table 3.3 Summary of Clay Mineral Characteristics

Structural Units

1. Silica Tetrahedron: Si atom at center. Tetrahedron units form hexagonal network = $Si_4O_6(OH)_4$
2. Gibbsite Sheet: Aluminum in octahedral coordination. Two-thirds of possible positions filled. $Al_2(OH)_6$—O-O = 2.60 Å.
3. Brucite Sheet: Magnesium in octahedral coordination. All possible positions filled. $Mg_3(OH)_6$—O-O = 2.60 Å

All bases in same plane. O–O = 2.55 Å—Space for Si = 0.55 Å—Thicknesss 4.93 Å. C–C height = 2.1 Å.
OH–OH = 2.94 Å. Space for ion = 0.61 Å. Thickness of unit = 5.05 Å. Dioctahedral.
OH–OH = 2.94 Å. Space for ion = 0.61 Å. Thickness of unit = 5.05 Å. Trioctahedral.

| Type | Sub-Group and Schematic Structure | Mineral | Complete Formula/Unit Cell[a] | Octahedral Layer Cations | Tetrahedral Layer Cations | Structure—Isomorphous Substitution | Structure—Interlayer Bond | Structure—Crystal Structure | Structure—Basal Spacing | Shape | Size[b] | Cation Exchange Cap.(meq/100 gm) | Specific Gravity | Specific Surface m²/gm. | Occurrence in Soils of Engineering Interest |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Allophane | Allophanes | Amorphous | — | — | | | | | Irregular, somewhat rounded | .05–1 μ | | | | Common |
| 1:1 | Kaolinite | Kaolinite | $(OH)_8Si_4Al_4O_{10}$ | $Al_4$ | $Si_4$ | Little | O–OH Hydrogen Strong | Triclinic a = 5.14, b = 8.93, c = 7.37 α = 91.6°, β = 104.8°, γ = 89.9° | 7.2 Å | 6-sided flakes | 0.1–4 μ × .05–2 μ to 3000 × 4000(stacks) single | 3–15 | 2.60–2.68 | 10–20 | Very Common |
| | G S | Dickite | $(OH)_8Si_4Al_4O_{10}$ | $Al_4$ | $Si_4$ | Little | O–OH Hydrogen Strong | Monoclinic a = 5.15, b = 8.95, c = 14.42 β = 96°48′ | 14.4 Å | 6-sided flakes | 0.07–300 × 2.5–1000 μ | 1–30 | | | Rare |
| | G S | Nacrite | $(OH)_8Si_4Al_4O_{10}$ | $Al_4$ | $Si_4$ | Little | O–OH Hydrogen Strong | Almost Orthorhombic a = 5.15, b = 8.96, c = 43 β = 90°20′ | 43 Å | Rounded flakes | 1 μ × .02t–.15μ | | | | Rare |
| | | Halloysite (dehydrated) | $(OH)_8Si_4Al_4O_{10}$ | $Al_4$ | $Si_4$ | Little | O–OH Hydrogen Strong | a = 5.14 in O-Plane a = 5.06 in OH-Plane | 7.2 Å | Tubes | .07 μ O.D. .04 μ I.D. 1 μ long. | 5–10 | 2.55–2.56 | | Occasional |
| | | Halloysite (hydrated) | $(OH)_8Si_4Al_4O_{10}.4H_2O$ | $Al_4$ | $Si_4$ | Little | O–OH Hydrogen Strong | b = 8.93 in O-Plane b = 8.62 in OH-Plane ∴ layers curve | 10.1 Å | Tubes | | 5–40 | 2.0 –2.2 | 35–70 | Occasional |
| | Montmorillonite $(OH)_4Si_8Al_4O_{20}.NH_2O$ (Theoretical Unsubstituted) (Bentonite) | Montmorillonite | $(OH)_4Si_8(Al_{3.34}Mg_{.66})O_{20}.nH_2O$ ↓ $Na_{.66}$ | $Al_{3.34}Mg_{.66}$ | $Si_8$ | Mg for Al, Net charge always = 0.66-/unit cell | O–O Very weak expanding lattice | 9.6Å—Complete separation | | Dioctahedral | Flakes (Equidimensional) | >10 Å × up to 10 μ | 80–150 | 2.35–2.7 | 50–120 Primary 700–840 Secondary | Very Common |
| | S G S | Beidellite | $(OH)_4(Si_{7.34}Al_{.66})(Al_4)O_{20}.nH_2O$ ↓ $Na_{.66}$ | $Al_4$ | $Si_{7.34}Al_{.66}$ | Al for Si, Net charge always = 0.66-/for unit cell | O–O Very weak expanding lattice | 9.6 Å—Complete separation | | Dioctahedral | | | | | Rare |
| | | Nontronite | $(OH)_4(Si_{7.34}Al_{.66})Fe_4^{3+}O_{22}.nH_2O$ ↓ $Na_{.66}$ | $Fe_4$ | $Si_{7.34}Al_{.66}$ | Fe for Al, Al for Si, Net charge always = 0.66-/for unit cell | O–O Very weak expanding lattice | 9.6 Å—Complete separation | | Dioctahedral | Laths | Breadth = 1/5 length to several μ × unit cell | 110–150 | 2.2 –2.7 | | Rare |
| | S G S | | | | | | | | | | | | | | |
| 2:1 | Saponite | Hectorite | $(OH)_4Si_8(Mg_{5.34}Li_{.66})O_{20}.nH_2O$ ↓ $Na_{.66}$ | $Mg_{5.34}Li_{.66}$ | $Si_8$ | Mg, Li for Al, Net charge always = 0.66-/unit cell | O–O Very weak expanding lattice | 9.6 Å—Complete separation | | Trioctahedral | | To 1 μ × unit cell breadth = 0.02 – 0.1 μ | 17.5 | | | Rare |
| | B | Saponite | $(OH)_4(Si_{7.34}Al_{.66})Mg_6O_{20}.nH_2O$ ↓ $Na_{.66}$ | Mg, $Fe^{3+}$ | $Si_{7.34}Al_{.66}$ | Mg for Al, Al for Si, Net charge always = 0.66-/for unit cell | O–O Very weak expanding lattice | | | Trioctahedral | Similar to Mont. | Similar to Mont. | 70–90 | 2.24–2.30 | | Rare |
| | B | Sauconite | $(Si_{6.94}Al_{1.06})Al_{.44}Fe_{.34}Mg_{.26}Zn_{4.80}O_{20}(OH)_4.nH_2O$ ↓ $Na_{.66}$ | $Al_{.44}Fe_{.34}Mg_{.26}Zn_{4.80}$ | $Si_{6.94}Al_{1.06}$ | Zn for Al | O–O Very weak expanding lattice | | | Trioctahedral | Broad Laths | 50 Å Thick | | | | Rare |
| | Hydrous Mica (Illite) S G S (K) S G S | Illites | $(K,H_2O)_2(Si)_8(Al,Mg,Fe)_{4.6}O_{20}(OH)_4$ | $(Al,Mg,Fe)_{4-6}$ | $(Al,Si)_8$ | Some Si always replaced by Al. Balanced by K between layers. | K ions: strong | | 10 Å | Both Dioctahedral and Trioctahedral | Flakes | .003–.1 μ × up to 10 μ | 10–40 | 2.6 –3.0 | 65–100 | Very Common |
| | Vermiculite S S S S | Vermiculite | $(OH)_4(Mg.Ca)_x(Si_{8-x}.Al_x)(Mg.Fe)_6O_{20}.yH_2O$ x = 1 to 1.4, y = 8 | $(Mg,Fe)_6$ | $(Si,Al)_8$ | Al for Si net charge of 1 to 1.4/unit cell | Weak | a = 5.34, b = 9.20 c = 28.91, β = 93°15′ | 10.5–14 | Alternating Mica and double $H_2O$ layers | Similar to Illite | | 100–150 | | 40–80 Primary 870 Secondary | Fairly Common |
| 2:1:1 | Chlorite S S B S S | Chlorite (Several varieties known) | $(OH)_4(SiAl)_8(Mg.Fe)_6O_{20}$ (2:1 layer) $(MgAl)_6(OH)_{12}$ Interlayer | $(Mg,Fe)_6$ (2:1 layer) $(Mg,Al)_6$ Interlayer | $(Si,Al)_8$ | Al for Si in 2:1 layer Al for Mg in Interlayer | | Monoclinic (Mainly) a = 5.3, b = 9.3 c = 28.52, β = 97°8′ | 14 Å | | Similar to Illite | 1 μ | 10–40 | 2.6 –2.96 | | Common |
| | Chain Structure | Sepiolite | $Si_4O_{11}(Mg.H_2)_3H_2O\ 2(H_2O)$ | | | Fe or Al for Mg | | Monoclinic a = 2 × 11.6, b = 2 × 7.86 c = 5.33 | | Chain | Flakes or Fibers | | 20–30 | 2.08 | | Rare |
| | | Attapulgite | $(OH_2)_{4-}(OH)_2Mg_5Si_8O_{20}.4H_2O$ | | | Some for Al for Si | Weak = chains linked by 0 | $a_0 \sin\beta$ = 12.9 $b_0$ = 18 $c_0$ = 5.2 | | Double Silica Chains | Laths | Max, 4-5 μ × 50–100 Å Width = 2t | 20–30 | | | Occasional |

[a] Arrows indicate source of charge deficiency. Equivalent Na listed as balancing cation. Two formula units (Table 3.2) are required per unit cell.
[b] Electron Microscope Data.

*References*


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ATTACHMENT I

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# Foundation Engineering


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ATTACHMENT I (continued)



similar to the pages of a book. The arrangement and the chemical composition of these sheets determine the type of clay mineral. All three types have a flake-shaped structure due to the sheetlike arrangement of the crystal lattice. Depending upon the arrangement of the atoms in the crystal, the individual flakes or particles may separate in the presence of water or may even expand and allow the water molecules to arrange themselves between the sheets of the crystal lattice.

All clay minerals possess the ability to adsorb positive ions from an aqueous solution. When a salt such as NaCl is dissolved in water, it is dissociated into positive ions (cations) of $Na^+$ and negative ions (anions) of $Cl^-$. The unbalanced charges that exist on the surface of the clay mineral are mostly negative and thus attract the $Na^+$ cations. When the cation is attached to the clay mineral, it is said to be *adsorbed*. However, this type of adsorption is not permanent because if a portion of the clay mineral containing adsorbed $Na^+$ ions is placed in a solution of KCl (potassium chloride) most of the $Na^+$ ions are replaced by $K^+$ ions. This process of replacement by the excess ion is called *base exchange*.

**Table 1.6. Base-exchange Capacity of Clay Minerals**

| Mineral | Number of Positive Charges Adsorbed per 100 g |
|---|---|
| Montmorillonite | $360\text{–}600 \times 10^{20}$ |
| Illite | 120–240 |
| Kaolinite | 20– 90 |

The base-exchange capacity of the different types of clay minerals may be measured by washing a sample of each with a solution of a salt not commonly found in nature, such as ammonium chloride ($NH_4Cl$), and determining the amount of adsorbed $NH_4^+$ by measuring the difference between the original and final concentration of the washing solution. It is convenient to express the base-exchange capacity in terms of the total number of positive charges adsorbed per 100 g of clay mineral, since this factor is independent of the size and weight of each adsorbed ion or the number of charges associated with it. Table 1.6 shows the approximate range of values obtained for the three principal types of clay minerals. The ability to adsorb water on the surface of the clay mineral also varies in the same order.

Table 1.6 indicates that the montmorillonite minerals are about ten times as active as the kaolinite minerals. Hence, a much smaller amount of montmorillonite than of kaolinite would be required to impart typical colloidal properties to a mixed-grained soil. Few

ATTACHMENT J

# ENVIRONMENTAL SOLUTIONS, INC.

## MEMORANDUM

TO: File

December 27, 1995

FROM: Chris Hansen

Project No. 95-284

Record of Telephone Conference with
David Daniel, Ph.D. regarding GCLs
American Ecology Corporation
Beatty Facility, Nevada Cell 12 Design

This memorandum summarizes the discussions from a telephone conference held on December 19, 1995 at 9:00 a.m. Pacific Standard Time (PST) with Dr. David Daniel of the University of Texas at Austin. The purpose of the call was to discuss various issues related to geosynthetic clay liners (GCLs), including their equivalency to compacted clay liners (CCLs) in landfill liner systems.

The call was prompted by ongoing negotiations with the Nevada Division of Environmental Protection (NDEP) regarding the use of a GCL in lieu of a CCL as the lower component of the bottom composite liner for the design of Cell 12 at the American Ecology Corporation's (AEC) facility located near Beatty, Nevada.

Participants in the telephone conference were:

- David Daniel, Ph.D. - University of Texas at Austin
- Jo Gayle Simmons - American Ecology Corporation
- Dick Ellison, Ph.D. - Environmental Solutions, Inc.
- Eric Walther, Ph.D. - Environmental Solutions, Inc.
- Scott Brown - Environmental Solutions, Inc.
- Chris Hansen - Environmental Solutions, Inc.

Key issues discussed during the telephone conference are described as follows:

- Dr. Daniel summarized the activities and results of recent large-scale test plots (test plots were ±30 m long with 90 cm of cover soil) conducted in Cincinnati, Ohio:
  - The purpose of the test was to evaluate the internal shear strength of hydrated GCLs with a 1.5 factor of safety by evaluating their performance on 3H:1V and 2H:1V slopes. Five tests were run at 3H:1V and ten were run at 2H:1V. The GCLs were placed dry on a wet subgrade and were allowed to hydrate. The GCLs and overlying layers were cut and then observed for movement.
  - All five test plots at 3H:1V did not fail, and eight of the ten test plots at 2H:1V did not fail. Two of the 2H:1V test plots did fail, which occurred at the GCL/geomembrane interface: a Bentomat and a Claymax GCL.
  - One 2H:1V test plot produced somewhat erratic behavior. It was a Gundseal test plot, which partially hydrated in an irregular patchwork pattern.
  - Results show that internal friction angles of GCLs decline with increases in normal stresses. For example, at low normal stresses GCL has an internal friction angle of approximately 20 degrees, while at normal stresses approximating a loaded landfill cell (100 feet of overburden) GCL has an internal friction angle of approximately 8 degrees.
  - The test plot results are considered to confirm findings in literature. A workshop was held in August 1995 to discuss the test plot results and a written report will be released by the United States Environmental Protection Agency (EPA) likely in February 1996.

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- Dr. Daniel provided comments on a verbal description of draft calculations prepared by Dr. Walther regarding GCL and CCL chemical adsorption capacity:
  - Need to address diffusion (in addition to advection), which may be a significant component of chemical transport in thin GCLs.
  - The assumed total porosity numbers of 0.75 and 0.437 should be reduced to effective porosities in the range of 0.1 to 0.3.
  - The assumed permeability of the GCL with a 10,000 psf overburden pressure should be about an order of magnitude less than it is when the cell is not loaded (e.g., use $3x10^{-10}$ centimeters per second (cm/sec) loaded versus $3x10^{-9}$ cm/sec unloaded).
  - Prefers an approach based on cation exchange capacity for the purpose of comparing GCL and CCL chemical adsorption capacity, rather than one based on the time for chemical transport through the two media.
  - Is aware of landfill designs with an adsorptive layer placed above the GCL for the purpose of providing additional chemical adsorption capacity.
  - Need to tie the significance of liner chemical adsorption capacity to site-specific conditions, such as the nature of the vadose zone, ground water and potential contaminant receptors. For landfills in arid environments, contaminant fate and transport should be evaluated before looking at a liner's adsorption capacity.
- Dr. Daniel stated that a GCL is a "more sensible" choice (than a CCL) in an arid environment. A GCL is more "survivable" than a CCL. There is a "good possibility to a high probability" of desiccation of a CCL in an arid environment within 10 years. Soil suction in an arid environment may be on the order of tens to 100s of atmospheres (bars).
- Dr. Daniel had one of his Ph.D. students conduct a test in which a 4- by 8-foot sheet of GCL was hydrated in a tank, next drained and desiccated for 30 days with hot air blowers, and then flooded with water again. The results showed that the desiccated GCL's cracks healed within about three hours, and the rehydrated GCL returned to its original permeability.
- Dr. Daniel mentioned that Richard Anderson of the Texas Natural Resource Conservation Commission looks favorably on GCLs, and would likely be receptive to a telephone conversation on the issue.
- In a discussion of various media which may be used to prepare slope subgrades to mitigate the potential to puncture an overlying GCL, Dr. Daniel mentioned that asphalts are "creepy" (i.e., have a tendency to creep) materials which may lubricate the interface.
- On the issue of GCL puncture resistance, Dr. Daniel stated that owners should do all that is possible to mitigate the issue through design and Construction Quality Assurance (CQA), but there is still some risk such as through operator error. Owners may wish to consider adding a low-permeability soil layer below the GCL (to provide a thicker barrier against puncture).
- On the issue of the need for GCL field tests, Dr. Daniel stated that this was not brought up in recent conversations he has had with Cathe Pool (of NDEP). He said it would be hard to imagine a GCL field test with a useful objective.

Although not discussed in the telephone conference, attached to this memorandum is a copy of a SAGEOS Geosynthetics Analysis Service report on the Cincinnati workshop regarding the GCL test plots (see above).

CH/LP:pm
Attachment

cc: Dick Ellison
Miro Knezevic
Scott Brown
Eric Walther

ATTACHMENT K

# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 4Mar96 Subject DIFFUSION OF CONSTITUENTS IN LEACHATE Sheet No. 1 of 10

Chkd. By TLP Date 3/5/96 Proj. No. 95-284

## PURPOSE

Determine if diffusion of constituents would significantly affect the breakthrough times in the 16 Jan 96 calculation.

## APPROACH

Describe concentration patterns of examples constituents in leachate and gas-filled pore spaces in front of the infiltrating leachate, and relate diffusion to concentration.

## THEORY

Diffusion occurs where a concentration gradient moves molecules by intermolecular collisions from the location of high concentration towards the location of low concentration. The leachate is assumed to have a constant concentration of its constituents throughout. When the leachate reaches the top of each alternative liner, the



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THEORY (continued)

liner pore spaces are assumed to have no constituent molecules. The representative inorganic constituent, zinc, has a negligible vapor pressure and hence, will not escape from the leachate and diffuse through the pore spaces of the liner in advance of its previously calculated migration.

The representative organic constituent, chloroform, is a volatile liquid and will enter the pore spaces in front of the leachate. Chloroform molecules, whether they be carried in the leachate or diffuse through liner pore spaces in advance of the leachate, will be adsorbed or condense on sites on the surface of the liner medium. This retardation affects chloroform in the leachate and in gaseous form in pore spaces.

The concentration distribution of chloroform at the initial time when the leachate first reaches the top



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THEORY (continued)

of the liner is shown in Figure 1A. The chloroform concentration in the leachate is uniform and no chloroform nor leachate has migrated into the liner pore spaces.

At some later time $t_1$ the leachate has infiltrated distance $vt_1$ into the liner, where $v$ is the bulk velocity of the leachate. Some chloroform has volatilized out of the leachate into the pore spaces of the liner just in front of the leachate as shown in Figure 1B. Diffusion carries some of the volatilized constituent molecules forward with a decreasing concentration. The concentration of chloroform and other constituents in the leachate, volatile or not, is greatly reduced as shown by the long breakthrough times found in the 16Jan96 calculation, caused by adsorption on the surfaces of the particles comprising the liner porous medium. Hence, the equilibrium concentration of



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# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 5 Mar 96 Subject DIFFUSION OF CONSTITUENTS IN LEACHATE Sheet No. 4 of 10

Chkd. By TLP Date 3/5/96 Proj. No. 95-284

$C_{CHCl_3}$ mom (mole/mole)

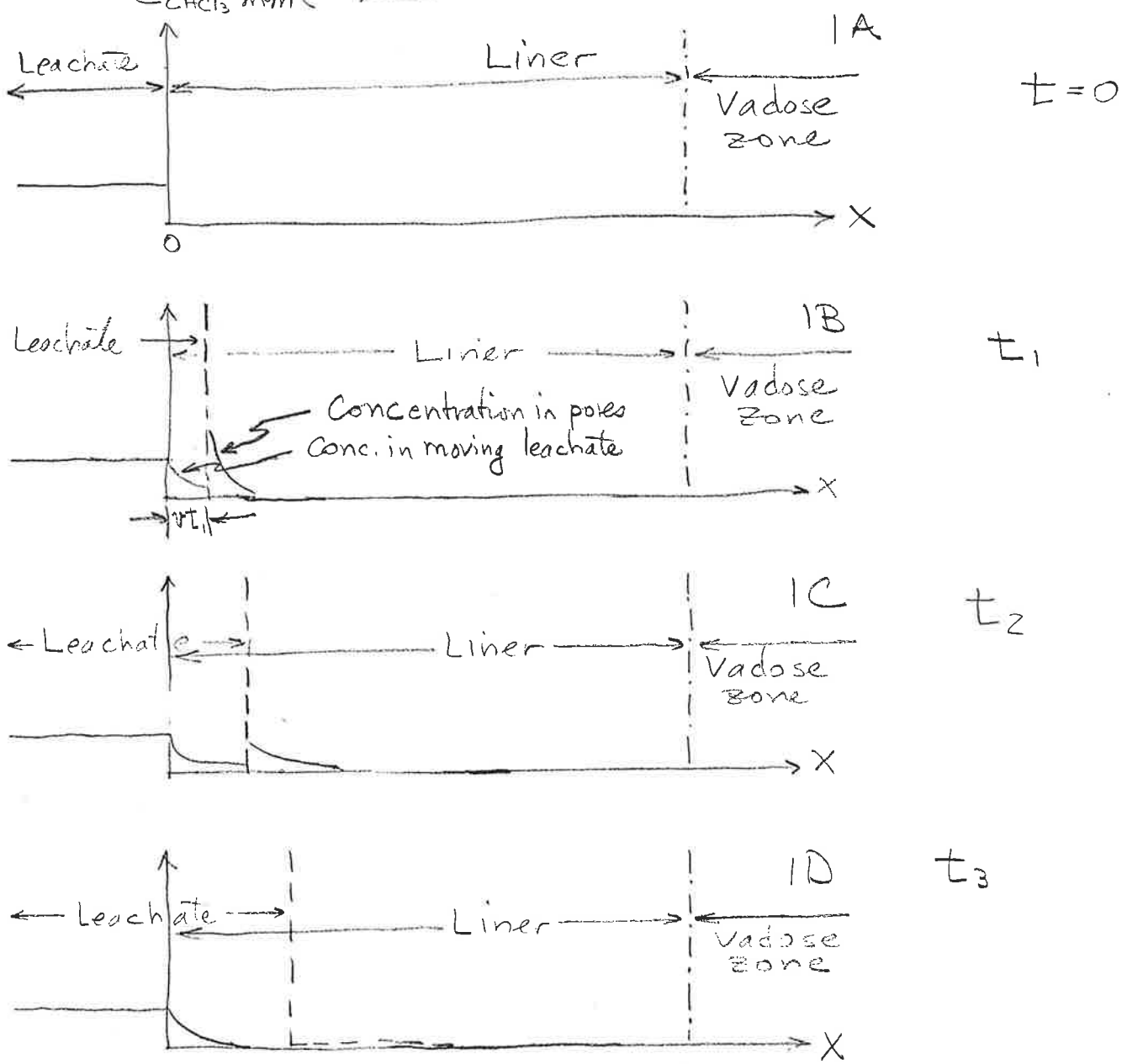


FIGURE 1

CHLOROFORM CONCENTRATION DISTRIBUTION



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THEORY (continued)

chloroform in the pore spaces adjacent to the leachate front is much reduced.

Because the leachate moves much faster (see Calculation section) than the chloroform in the leachate, the concentration of chloroform in the leachate near the front edge decreases to zero as shown in Figures 1C and 1D. Then , no chloroform is available to volatilize into the newly exposed pore space just in front of the advancing leachate. The diffusion of chloroform in the linen pore spaces loses its source and can no longer propagate a significant amount of chloroform in advance of the chloroform still left in the leachate at some distance back from the leading edge. To obtain an indirect measure of the magnitude of retardation of chloroform, we will calculate the breakthrough times for leachate as a comparison.

# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 5Mar96 Subject DIFFUSION OF CONSTITUENTS IN LEACHATE Sheet No. 6 of 10

Chkd. By TLP Date 3/5/96 Proj. No. 95-284

CALCULATIONS

The time for leachate to breakthrough the side slope CCL is found through the use of Equations 2 and 5 in the 16 Jan 96 calculation as follows:

$$t_w = \frac{T\phi}{K\left(\frac{h+T}{T}\right)}$$

Accounting for the 0.5 to 1 (63.4°) angle of the side slope,

$$t_w = \frac{\frac{3\,ft}{\cos 63.4°}\; 30.48\,\frac{cm}{ft}\; 0.2}{10^{-7}\,\frac{cm}{s}\left(\frac{0.1\,ft + 3\,ft/\cos 63.4°}{3\,ft/\cos 63.4°}\right)}$$



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# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 4Mar96 Subject DIFFUSION OF CONSTITUENTS IN LEACHATE Sheet No. 7 of 10
Chkd. By [illegible] Date 3/6/96 Proj. No. 95-284

CALCULATIONS (continued)

$t_w = 4.0 \times 10^8 \text{ sec} = 1.12 \times 10^5 \text{ hr} = 12.8 \text{ yr}$

The time for leachate to penetrate the bottom GCL is:

$$t_w = \frac{3\text{ ft } 30.48 \frac{\text{cm}}{\text{ft}} \; 0.2}{10^{-7} \frac{\text{cm}}{\text{s}} \left( \frac{0.1 \text{ ft} + 3 \text{ ft}}{3 \text{ ft}} \right)}$$

$t_w = 1.77 \times 10^8 \text{ sec} = 5.6 \text{ yr}$

For side-slope GCL, the time for leachate to penetrate is:

$$t_w = \frac{\frac{7 \text{ mm}}{\cos 63.4^\circ} \frac{1 \text{ cm}}{10 \text{ mm}} \; 0.3}{5 \times 10^{-9} \frac{\text{cm}}{\text{s}} \left( \frac{0.1 \text{ ft } 304.8 \frac{\text{mm}}{\text{ft}} + \frac{7 \text{ mm}}{\cos 63.4^\circ}}{7 \text{ mm}/\cos 63.4^\circ} \right)}$$

$t_w = 3.18 \times 10^7 \text{ sec} = 1.0 \text{ yr}$

For the bottom combined GCL + CSL, $t_w = t_{GCL} + t_{CSL}$

$$t_{GCL} = \frac{7 \text{ mm} \frac{1 \text{ cm}}{10 \text{ mm}} \; 0.3}{5 \times 10^{-10} \frac{\text{cm}}{\text{s}} \left( \frac{0.1 \text{ ft } 304.8 \frac{\text{mm}}{\text{ft}} + 7 \text{ mm}}{7 \text{ mm}} \right)}$$

$t_{GCL} = 7.84 \times 10^7 \text{ sec} = 2.5 \text{ yrs.}$ and the volume flux

$$F_v = K\left(\frac{h+T}{T}\right) = 2.68 \times 10^{-9} \frac{\text{cm}^3}{\text{cm}^2\text{-sec}}$$

CALCULATIONS (continued)

To calculate $t_{CSL}$, we need to recognize that the volume flux or velocity of leachate (see Eq. 3 in Theory section of 16 Jan 96 calculation) in the CSL is limited by the volume flux that occurs in the GCL. Hence,

$$t_{CSL} = \frac{T_{CSL}\,\phi_{CSL}}{K_{GCL}\,I_{GCL}} = \frac{T_{CSL}\,\phi_{CSL}}{F_{v\,GCL}} \qquad \text{(see Eqs. 2 \& 5B)}$$

$$t_{CSL} = \frac{T_{CSL}\,\phi_{CSL}}{K_{GCL}\left(\frac{h + T_{GCL}}{T_{GCL}}\right)} \qquad \text{(same as Eq. 5C)}$$

$$t_{CSL} = \frac{9\,\text{in}\;\; 2.54\,\frac{\text{cm}}{\text{in}}\;\; 0.2}{2.68\times10^{-9}\,\frac{\text{cm}^3}{\text{cm}^2\text{-sec}}}$$

$$t_{CSL} = 1.71\times10^{9}\,\text{sec} = 54\ \text{yrs.}$$

Hence, $t_w = t_{GCL} + t_{CSL}$
$= 2.5\ \text{yr} + 54\ \text{yr}$
$\doteq 56.5\ \text{yr}$

FINDINGS AND CONCLUSIONS

The leachate breakthrough times are listed along with the constituent breakthrough times on the next page (taken from p. 28 of 16 Jan 96 calculation). As can be seen, the breakthrough times of water are short compared to the times for chloroform, indicating that the chloroform molecules are adsorbed or otherwise retarded strongly. Such retardation means that the chloroform is not available to diffuse through the leachate nor stay at its front as the leachate migrates through the liner. Initially, some chloroform could volatilize from the front face of the leachate and diffuse through the pore spaces. The theoretical sequence of chloroform concentration curves indicated the potential change with time after initial arrival of the leachate at the top of the liner. The overall result is that leachate constituents breakthrough a liner much later, and early in the leachate migration constituents become unavailable at the leading edge to volatilize and diffuse through gas-filled pore spaces.

# ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 5 Mar 96 18 Jan 96 Subject COMPARISON OF GCL + CSL AND CCL Sheet No. ~~28~~ 10 of ~~32~~ 10

Chkd. By [illegible] Date 1/31/96 Proj. No. 95-284

FINDINGS ~~(continued)~~

| POSITION | LINER | CONSTITUENT | BREAKTHROUGH TIME (YRS) STEP-FUNCTION THEORY A | DISTRIBUTION COEFFICIENT THEORY B | LEAKAGE |
|---|---|---|---|---|---|
| BOTTOM | GCL + CSL | Zinc | 5,000 - 11,000 | 300,000 | 56 |
| | | Chloroform | NA | 44,000 | |
| | CCL (nondesiccated) | Zinc | 100 - 1,000 | 36,000 | 6 |
| | | Chloroform | NA | 5,300 | |
| SIDE-SLOPES | GCL | Zinc | 200-500 | 2,300 | 1 |
| | | Chloroform | NA | 340 | |
| | CCL (nondesiccated) | Zinc | 230 - 2,300 | 82,000 | 13 |
| | | Chloroform | NA | 12,000 | |

NA = Not applicable.

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