ANCHOR TRENCH STRESSES

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CALCULATION SUMMARY SHEET

PROJECT NUMBER: 073113

PROJECT NAME: USEN – Trench 12 Design, Supplemental Calculations

DATE: August 15, 2007

CALCULATION NUMBER: C.6 Revision: Update per 2007 Design

CALCULATION TITLE: Anchor Trench

DESCRIPTION OF CALCULATION:

Calculation to determine if 1996 trench design dimensions are sufficient for 2007 proposed liner members

REFERENCES USED: Number of Reference Pages Attached:______

1. Previous calc: 1996 calculation titled Anchor Trench"
2. Selected pages from 2007 Calculation - Liner Stability on Trench 12 Side-Slopes dated August 13, 2007

REVIEW COMMENTS:

CALCULATION MADE BY: CAB DATE: 8/22/2007

CALCULATION CHECKED BY: SLW DATE: 8/22/2007

CALCULATION REVISED BY: CAB DATE: 8/27/07

CALCULATION REVIEWED BY: SLW DATE: 8/22/2007

Purpose of Calculation

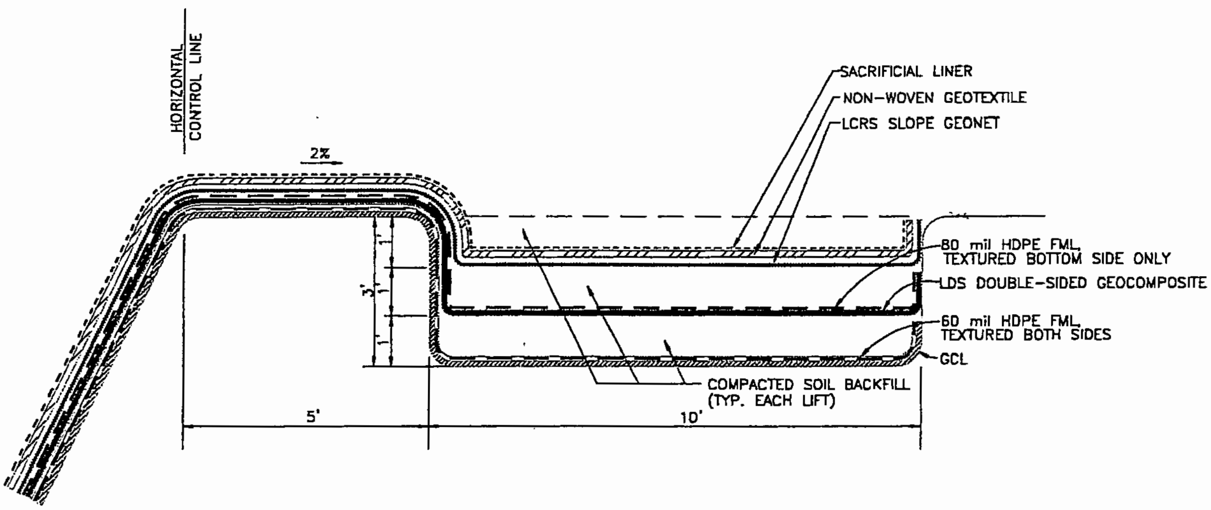
Verify that the 1996 anchor trench calculation to determine trench dimensions is applicable for liner members proposed in the 2007 design.

Method

- Compare total liner stresses (tensile loads) of 2007 liner members and 1996 liner members.
- Compare yield strengths of 2007 liner members and 1996 liner members.

Approach

The anchor trench is designed to withstand the total stresses of the liner side slope members. The anchor trench has the following configuration.



Total induced liner stresses due to gravity load, thermal expansion/contraction, seismic deformation, and settlement of waste fill in the 2007 design and detailed in the Liner Stresses Calculation done by AAT, August 13, 2007 was compared to stresses evaluated in the 1996 design as shown in the table below. These are the stresses considered normal during construction and operation of the landfill trench. The members are combined in zones. Zone 1 includes the upper geotextile and LCRS geonet, Zone 2 includes the primary geomembrane and LDS geocomposite, and Zone 3 includes the secondary geomembrane and GCL. A safety factor of 4 is used for the minimum required anchor capacity as was done in the 1996 calculation.

Zone	Material	Total Induced Liner Stresses (lbs/in) 1996 Design	Zone Stresses (lbs/in) 1996 Design	Zone Stress with x4 FS (lbs/in) 1996 Design	Total Induced Liner Stresses (lbs/in) 2007 Design	Zone Stresses (lbs/in) 2007 Design	Zone Stress with x4 FS (lbs/in) 1996 Design
1	Geotextile	12	23.3	93.2	12.1	22.3	89.2
	Geonet	11.3			10.2		
2	100-mil HDPE/80-mil HDPE	10.6	10.6	42.4	8.7	10.0	40.0
	Geocomposite	0			1.3		
3	80-mil HDPE/60-mil HDPE	9	11.4	45.6	6.8	10.3	41.2
	GCL	2.4			3.5		

Since all of the zone stresses in the 2007 design are less than the 1996 design, the anchor trench design as presented in the 1996 design prevents pull-out.

The anchor trench capacity also is designed to exceed the yield strength of the stronger material in each zone such that the liner members will yield before being pulled out in the event of unforeseen incident. Localized liner member failure is preferred over pull-out at trenches. Localized liner member failures should be more easily repairable than an anchor trench pull-out.

Yield strengths were compared in the table below.

Zone	Material (reference for product data)	Yield Strength (lb/in) 1996 Design	Strongest Yield per Zone (lbs/in) 1996 Design	Yield Strength (lb/in) 2007 Design	Strongest Yield per Zone (lbs/in) 1996 Design
1	Geotextile	150	150	140	140
	Geonet	50		45	
2	100-mil HDPE/80-mil HDPE	240	240	168	168
	Geocomposite	Not Reported		78[1]	
3	80-mil HDPE/60-mil HDPE	192	192	126	126
	GCL	37.5		50	

Comparison in each zone shows that all yield strengths are highest in the 1996 design; therefore, using the 1996 anchor trench design is sufficiently conservative relative to anchor trench pull-out.

Results

Total liner stresses (by zone) for 2007 liner members are all less than those considered for 1996 liner members; therefore, the 1996 anchor trench design is sufficiently conservative for use in the 2007 design update.

Yield strengths (by zone) for 2007 liner members are all less than 1996 liner members; therefore, liner members should yield prior to being pulled out of the anchor trench. The 1996 anchor trench design is sufficiently conservative for use in the 2007 design update.

REFERENCE

2007 LINER STRESSES

APPLICABLE PAGES

Table 2: Liner Properties Considered in 2007 Design

Liner Material (reference for product data)	Unit Weight (lbs/ft^2)	Self Weight (lbs/ft)	Accumulated Weight (lbs/ft)
Non-woven Geotextile (GSE NW10; GEO 1008002)	0.069	5.8	5.8
Geonet (GSE HyperNet XL4000N004)	0.17	14	20
80-mil HDPE (GSE HST 080G000 - textured on one side)	0.37	31	51
Double-sided Geocomposite (GSE Fabrinet with double sided 6 ounce/yd^2 geotextile)	0.25	21	72
60-mil HDPE (GSE HD 060G000 –textured both sides)	0.28	24	96
GCL (CETCO Bentomat DN)	1.22	103	198

The 2007 design Geonet and weight was obtained from a phone conversation with the manufacturer's technical representative. Geonets are not commonly measured for unit weight.

Table2: Material Strengths

Material (reference for product data)	Yield Strength (lb/in) 1998 Design	Yield Strength (lb/in) 2007 Design
Geotextile	150	140[1]
Geonet	50	45
100-mil HDPE/80-mil HDPE	240	168
Geocomposite	Not Reported	78[1]
80-mil HDPE/60-mil HDPE	192	126
GCL	37.5	50

1 Yield strength obtained in phone conversation with manufacturer's representative

This calculation analyzes strength for liner materials proposed for use in Trench 12 at US Ecology's Beatty, Nevada facility. Stresses and strains on liner materials are estimated from the self weight of the liners, thermal expansion, seismic deformation, and settlement of the waste fill. The total induced stress from these factors is compared to estimated allowable stresses on the liner materials from estimated factors of safety that are assumed to be conservative.

The liner materials will be placed on a 0.5:1 slope (an angle of 63.4° below horizontal). The trench will be 75 feet deep resulting in a slope length of 84 feet.

Self Weight Forces on Liner Members and Tensile Loads

Assuming stable slopes, the load due to gravity (tensile load) on the liners is due to self weight (shown in the tables above). The weight on the liner members accumulates for underlying liner members.

Self Weight = unit weight * slope length

Table 4: Stains and Total Tensile Loads on each Liner Member.

Liner Material (reference for product data)	T, Tensile Load Self Weight (lbs/in)	Strain due to Self Weight (%)	Strain due to Thermal Expansion (%)	Strain due to Seismic Deformation (%)	Strain due to Waste Settlement (%)	Total Strain (%)	2005 Total Tensile Load (lbs/in)	1996 Total Tensile Load (lbs/in)
Non-woven Geotextile (GSE NW10; GEO 1008002)	0.36	0.13	0.0	0.2	4.0	4.3	12.1	12
Geonet (GSE HyperNet XL4000N004)	1.03	0.53	0.5	0.2	4.0	5.2	10.2	11.3
80-mil HDPE (GSE HST 080G000 - textured on one side)	1.69	0.12	0.5	0.0	0.0	0.62	8.7	10.6
Double-sided Geocomposite (GSE Fabrinet with double sided 6 ounce/yd² geotextile)	1.26	0.81	0.0	0.0	0.0	0.81	1.3	Ø
60-mil HDPE (GSE HD 060G000 –textured both sides)	1.56	0.15	0.5	0.0	0.0	0.65	6.8	9
GCL (CETCO Bentomat DN)	3.49	1.05	0.0	0.0	0.0	1.05	3.5	2.4

REFERENCE

1996 ANCHOR TRENCH CALCULATION

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

US ECOLOGY

BEATTY, NV

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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

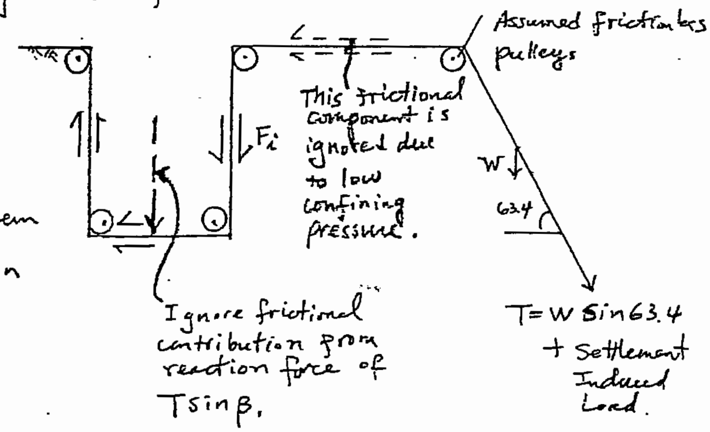
Table of Contents

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).



MODEL

- Downdrag Load
  T = Liner Stresses (Ref. 12)

- F_i = resistance along soil/liner system interfaces (see Fig. 1 for design geometry).

- 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 sacrificial HDPE membrane will be placed over the non-woven geotextile (see Figure 1) and will be anchored in Zone 1. However, this sacrificial 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 negligible load and embedment length of 10 feet, the layer will be adequately anchored.

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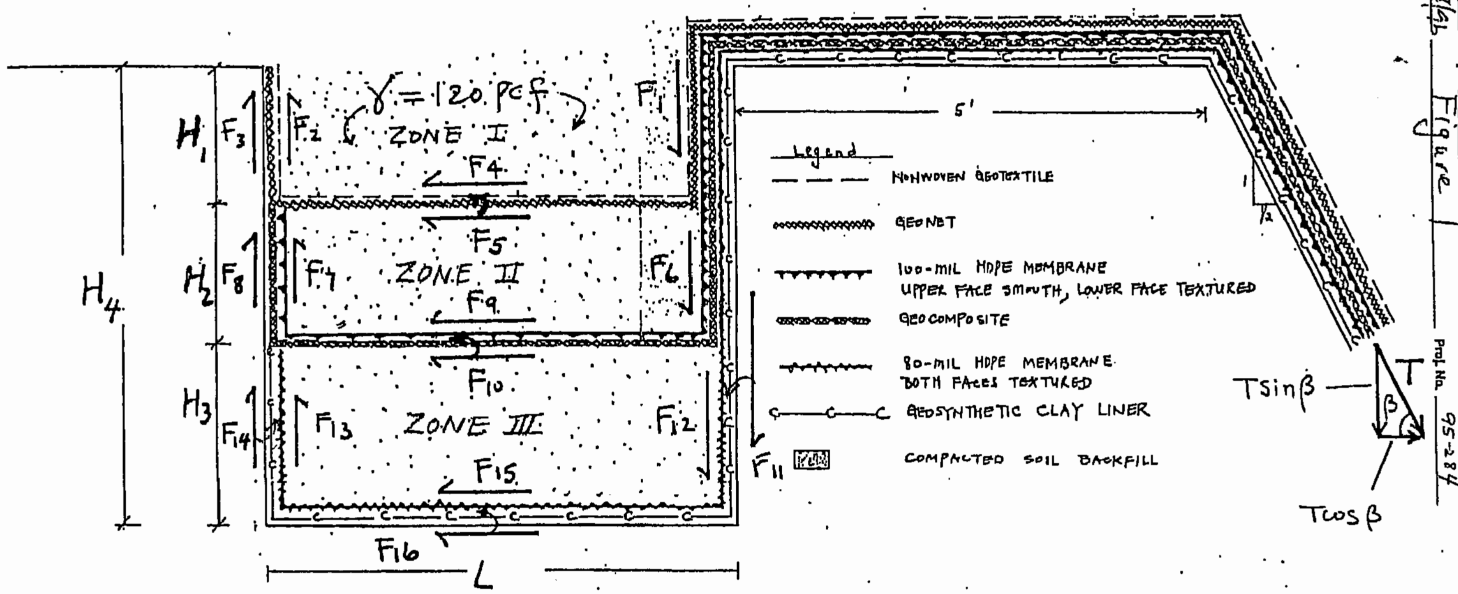
By JH Date 2/11/96 Subject ANCHOR TRENCH Sheet No. 3 of
Chkd. By WB Date 2/19/96 Figure Proj. No. 95-284

Figure 1



L = 10'

H_1 = 1'

H_2 = 1'

H_3 = 1'

H_4 = 3'

Vertical Frictional Component = $\frac{1}{2}\gamma H_i^2 K_o \tan\delta + a H_i$

Horizontal Frictional Component = $(\sigma_v \tan\delta + a) L$

δ = interface friction angle

a = interface adhesion

σ_v = vertical stress due to overburden.

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By JH Date 2/16/96 Subject Anchor Trench Sheet No. 4 of 10

Chkd. By WB Date 2/19/96 Proj. No. 75-284

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.

3.0 Calculations

Zone 1, Geotextile and Geonet Anchor Length

Min. Required anchor capacity is 93.2 lbs/"

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

$= \frac{1}{2} \gamma H_1^2 K_o \tan 21 + 25 H_1$ ($K_o = 0.5$, $\gamma = 120$ pcf) assumed

$= 11.5 H_1^2 + 25 H_1$

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/" = 1800 lbs/ft. < 1863 lbs/ft.
(oK.)

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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

$$42.4 \times 12 = 509 \text{ lbs/ft}$$

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.

$$= \tfrac{1}{2}\gamma\left(H_1 + \tfrac{H_2}{2}\right)^2 K_o \tan 18$$

$$= 9.75\left(1 + \tfrac{H_2}{2}\right)^2$$

$$F_8 = \tfrac{1}{2}\gamma\left(H_1 + \tfrac{H_2}{2}\right)^2 K_o \tan 21 + a H_2 \qquad \text{(Geocomposite / Sand)}$$

$$= 11.5\left(1 + \tfrac{H_2}{2}\right)^2 + 25 H_2$$

$$F_9 = \gamma (H_1 + H_2) \tan 18 \cdot L \qquad \text{(smooth HDPE / sand)}$$

$$= 39(1 + H_2) L$$

$$F_{10} = [\gamma (H_1 + H_2) \tan 7 + 280] \cdot L$$

$$= [14.7(1 + H_2) + 280] L$$

Let $H_2 = 1'$, $L = 10'$

$$\Rightarrow \Sigma F_i = 22 \times 2 + 51 + 780 + 3094 = 3969 \gg 509 \text{ lbs/ft, OK.}$$

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

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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/in

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

$$= \frac{1}{2}\gamma\left(\frac{2+H_3}{2}\right)^2 K_o \tan 5 + 550\cdot(2+H_3)$$
$$= 2.6\left(\frac{2+H_3}{2}\right)^2 + 550\,(2+H_3)$$

$$F_{12} = F_{13} = \frac{1}{2}\gamma\left(2+\frac{H_3}{2}\right)^2 K_o \tan 26$$ (textured HDPE/Sand)
$$= 14.6\left(2+\frac{H_3}{2}\right)^2$$

$$F_{14} = \frac{1}{2}\gamma\left(2+\frac{H_3}{2}\right)^2 K_o \tan 5 + 550\,H_3$$ (GCL Internal)
$$= 2.6\left(2+\frac{H_3}{2}\right)^2 + 550\,H_3$$

$$F_{15} = \gamma\,(2+H_3)\tan 26 \cdot L$$ (textured HDPE/Sand)
$$= 58.5\,(2+H_3)\,L$$

$$F_{16} = [\gamma\,(2+H_3)\tan 5 + 550]\,L$$ (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 >> 547 lbs/ft, O.K.

The yield strength of the 80-mil HDPE is 192 lbs/in (Ref. 4) which is $192 \times 12 = 2304$ lbs/ft. (< 9856 lbs/ft, O.K.)

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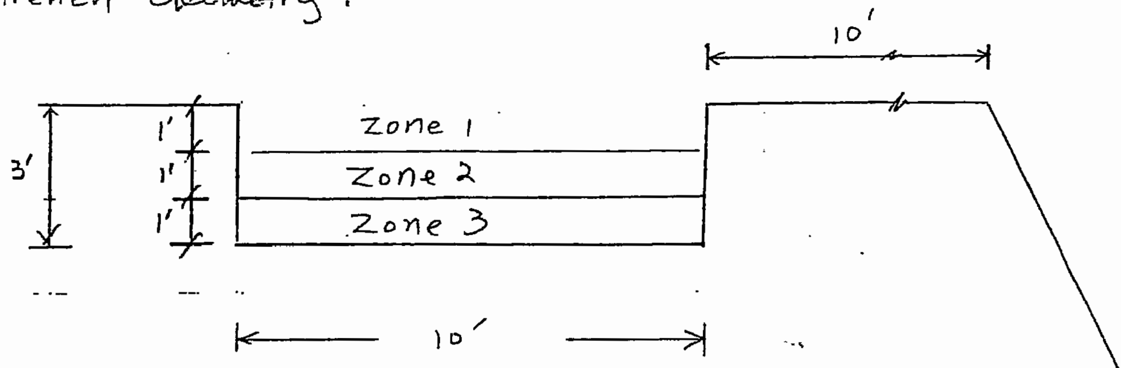
By JH Date 2/18/96 Subject Anchor Trench Sheet No. 8 of 10

Chkd. By WB 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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By JH Date 2/11/96 Subject Anchor Trench Sheet No. 9 of 10

Chkd. By WB Date 2/19/96 Table 1 Proj. No. 95-284

Friction Force No.	Interface	Interface: Friction angle (degree)	Interface: Adhesion (psf)	Refrence 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, δ = friction angle.

(2) H_i = Depth of Embedment, L = Trench Width. (4) Assumed shearing occur at internal face of GCL.

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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.

6. US Ecology Calculations, Anchor Trench, December, 1992.

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.

LCRS INFILTRATION RATE

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CALCULATION SUMMARY SHEET

PROJECT NUMBER: 073113

PROJECT NAME: USEN – Trench 12 Design, Supplemental Calculations

DATE: August 6, 2007

CALCULATION NUMBER: C.8 Revision: Update per 2007 Design

CALCULATION TITLE: LCRS Infiltration Rates

DESCRIPTION OF CALCULATION: LCRS Infiltration Rates

Determine infiltration rates into the Trench 12 Leachate Collection/Recovery System (LCRS) by same method used for previously approved calculation (1996) done for same purpose. Design change necessitating calculation update are 1) change in area of Trench 12 bottom, and 2) change in approach to managing all precipitation inside Trench 12 after first waste is disposed. All precipitation collected inside the cell, whether from floors or sidewalls, is to be considered leachate.

REFERENCES USED: Number of Reference Pages Attached:______

Previous calc: 1996 calculation titled "LCRS Flow Capacity and Pump Sizing"

Design drawing (2007) for Trench 12

REVIEW COMMENTS:

CALCULATION MADE BY: SLW DATE: 8/6/2007

CALCULATION CHECKED BY: CAB DATE: 8/7/2007

CALCULATION REVISED BY: CAB DATE: 8/27/07

CALCULATION REVIEWED BY: SLW DATE: 8/7/2007

Purpose of Calculation

Determine the potential flow rate into the LCRS sump(s)

Method

- Leachate: per discussions between USEN and NDEP (May 23, 2007 meeting), it was agreed that after the first waste is placed all precipitation collected inside Trench 12 must be managed as leachate.
- The sidewall liner design will include an upper (sacrificial) HDPE layer whose function will be to protect the liner components from direct exposure to solar UV radiation (and potential material deterioration from long-term UV exposure) and possibly to reduce the temperate change range in liner materials. That HDPE layer also will prevent direct entry of precipitation into the leachate collection member (geonet) of the liner system, instead routing all incident precipitation onto the cell floor.
- All incident precipitation (falling on the cell floor and sidewalls) will infiltrate into disposed waste and (ultimately) to the LCRS.
- The design storm is the 25-year, 24-hr storm, when is approximately 2.0 inches, based on Figure 28, NOAA Atlas 2, Volume VII.
- NOTE – precipitation that falls and collects within cell areas under construction (i.e., before waste placement) is not leachate, and can be managed by other means appropriate to stormwater management. Management of such stormwater is not the subject of this calculation.

Calculation Steps

Precipitation

1. The critical period (i.e., when the greatest flow is likely to occur) for sizing the leachate collection system is directly after placement of the two-feet thick layer of select waste over the bottom of the cell. Infiltration likely will reach the LCRS most quickly (after precipitation) and with least loss (i.e., moisture retention within waste) at this point.
2. The attached figure shows the configuration of the three phases (cells) and corresponding LCRS sumps of Trench 12, designated Phases 1, 2, and 3 and Sumps 12A, 12B, and 12C. The horizontal area (foot print) of the three phases, as determined in a separate calculation, is as follows.

Phases	LCRS Sump	Floor Area (sq ft)	Sidewall Area (sq ft)	Total Area (acres)	Design Precipitation* (acre-feet)
1	12A	149,149	44,492	4.45	0.74
2	12B	122,425	28,055	3.45	0.58
3	12C	93,409	48,410	3.26	0.54

* = Design Precipitation = Area (acres) x (2/12 feet)

<u>Infiltration into the LCRS</u>

1. No allowance is made for evaporation, so all precipitation is routed through the disposed waste (30 inches (2.5 feet) of selected waste).

2. Sandy soil is assumed to comprise the select soil layer. In the Engineering Documentation for the USEPA Help model, important properties of typical sandy soil types are provided. Conservatively, the most permeable of the sandy soil types is considered in this calculation.

Soil type (Unified Soil Classification System)	SM
Hydraulic conductivity	5.2 x 10-4 cm/sec
Porosity	0.473
Field capacity	0.222
Wilting point	0.104

3. For cell 12A, total precipitation, and total water available for infiltration, is 0.74 acre-feet. It is necessary to determine if saturated or unsaturated flow conditions will control infiltration through the waste. It is assumed that the soil layer contains, at most, sufficient moisture to be a field capacity (i.e., no gravity drainage occurs). A unit volume (1 ft x 1 ft x 2.5 ft thick) of the SM soil layer has available pore space (based above properties) of:

$$2.5 \text{ cu ft} \times 7.48 \text{ gallons/cu ft} \times (0.473 - 0.22) = 4.73 \text{ gallons}$$

4. The rate at which the 0.74 ac-ft (i.e., 2" over full 4.45 ac Phase 1 footprint) design storm saturates the 1 ft x 1 ft unit volume of soil on the 3.42 ac cell floor during a 24 hour period is:

0.74 ac-ft / 3.42 ac X 7.48 gal/cu ft X 1 day / 1440 min = 1.12 x 10-3 gal/min for each 1 ft x 1 ft unit volume

If the design storm occurs when the soil layer already is near Field Capacity (i.e., maximum moisture content without drainage), what fraction of the remaining air-filled porosity is filled by the design storm?

Porosity, ϕ = 0.473 or 47.3% of 30 inches = 14.19 inches

FC = 0.222 or 22.2% of 30 inches = 6.66 inches

Remaining air space before storm = 14.19 – 6.66 = 7.53 inches

Design storm = 2 inches over the full 4.45 acres of Phase 12A, which, when all precipitation falling on the 44,492 sq ft (1.02 ac) of the Phase 12A side slopes is routed to the cell floor, is equivalent to 2.6 inches over the 149,149 sq ft (3.42 ac) cell floor. 2.6 inches is less than 7.53 inches, so the design storm does not fully saturate the select waste layer unit volume. The combination of FC and design storm moisture results in the following water-filled geometery in the unit volume:

6.66" + 2.6" = 9.26", or 9.26" / 14.19" = 65% of available air space

This is a volumetric moisture content of:

0.222 + 2.6" / 30" = 0.222 + 0.087 = 0.309

Assuming that drainage does not begin during the design storm, 65% of the total air space is saturated following the storm. Since 65% is less than 100%, unsaturated flow controls drainage from the unit volume of soil (1 ft x 1 ft x 2.5 ft thick select waste layer) following the design storm.

Laminar flow through a porous medium is estimated by d'Arcy's Law, or

Q = Kia

Where,

Q = flow (typical unit is cu ft / min

K = hydraulic conductivity (typical unit is ft / min)

i = gradient (feet / feet or unit-less)

a = area through which flow occurs (square feet)

for unsaturated flow, unsaturated hydraulic conductivity (Ku) replaces K

5. For the unsaturated flow through the select waste layer,

   K = 5.2 x 10-4 cm/sec or 1.02 x 10-3 ft / min

   i = 2.5 ft / 2.5 ft = 1.0

   a = 1 sq ft

6. Unsaturated hydraulic conductivity is estimated by Campbell's equation (Engineering Documentation for the USEPA Help model):

   $Ku = Ks\ [(\theta - \theta r) /(\phi - \theta r)]$**$[3 + (2 / \lambda)]$

   Where

   Ku = unsaturated hydraulic conductivity, cm/sec
   Ks = saturated hydraulic conductivity, cm/sec
   θ = actual volumetric water content, vol/vol
   θr = residual volumetric water content, vol/vol
   ϕ = total porosity, vol/vol
   λ = pore-size distribution index, dimensionless

7. The residual water content, θr, is the amount of water remaining in the soil pores under infinite capillary suction. Residual water content is estimated by Rawls (Help documentation) as follows.

   $$\theta r = 0.014 + 0.25\ X\ Wilting\ Point,\ where\ WP \geq 0.04$$

   In this case, WP = 0.104 and is ≥ 0.04 and the equation is applicable. The residual water content is:

   $$\theta r = 0.014 + 0.25\ X\ 0.104 = 0.040$$

8. As discussed in the Help documentation reference, residual water content and pore-size distribution index, λ, are constants in the Brooks-Corey equation relating volumetric water content to matrix potential (capillary pressure and adsorptive forces).

$$(\theta - \theta r) / (\phi - \theta r) = (\psi b / \psi)^{**}\lambda$$

Where
ψ = capillary pressure, bars
ψ_b = bubbling pressure, bars

9. This equation is solved assuming that the volumetric water content is the field capacity at 0.33 bars of capillary suction, ψ, and equal to the wilting point at 15 bars capillary suction, ψb. Solving the two equation for the values assumed for the site soil provides the following results.

Constants	Variables
$\theta r = 0.040$	Field capacity = 0.222 @ ψ = 0.33
$\phi = 0.473$	Wilting point = 0.104 @ ψ = 15

Two equations in two unknowns solved on next page, results are:

ψb = 0.013

λ = 0.270

10. Assuming drainage does not start during the design storm, the volumetric water content following the storm is:

$$\theta = \text{Field Capacity} + 2.6\text{"} / 30\text{"}$$

$$\theta = 0.222 + 0.087$$

$$= 0.309$$

11. Based on the preceding, the unsaturated hydraulic conductivity is calculated as follows using Campbell's equation.

$$Ku = Ks\ [(\theta - \theta r) /(\phi - \theta r)]^{**}[3 + (2 / \lambda)]$$

$$Ku = 1.02 \times 10\text{-}3 * [(0.309\text{–}0.040) / (0.473\text{–}0.040)]^{**}[3 + (2 / 0.27)]$$

$$= 7.10 \times 10\text{-}6 \text{ ft/min}$$

12. Conservatively using a unit hydraulic gradient and assuming no evaporation, the rate at which infiltration drains through and from the 2.5 feet thick select soil layer into the LCRS per unit area (1 sq ft) is:

Q = Ku * i * A

Q = (7.10 x 10-6 ft / min)(1.0)(1.0 sq ft)

= 7.10 x 10-6 cu ft / min per square foot area

= 5.31 x 10-5 gal/ min per square foot area

Drainage over the entire 3.42 acre area of the Phase 1 floor is:

Q = (5.31 gal x 10-5 gal / sq ft – min) X 3.42 acre X 43,560 sq ft / acre

Q = 7.9 gallons per minute

The calculation was repeated for cells 12B and 12C. The results of the analyses are provided in the attached table.

Calculation result

When the 2.5-feet thick select waste layer, before placement of any additional waste, is subject to the 2-inch design storm, the resulting infiltration into the LCRS drainage net is dependent on unsaturated flow conditions and result in the following flow.

Cell	Infiltration (gal/min/cell)
12A	7.9
12B	5.4
12C	8.6

Infiltration Into the LCRS

Cell	Floor (ft2)	Sidewalls (ft2)	Design Storm (in)	Design Storm Precip to Floor (in)	Vol Water Content After Storm %	Ku (cm/sec)	Ku (ft/min)	*q* ($ft^3/min/ft^2$)	*q* ($gal/min/ft^2$)	*q* (gal/min/cell)
12A	149,149	44,492	2	2.60	0.309	3.61E-06	7.10E-06	7.10E-06	5.31E-05	7.9
12B	122,425	28,055	2	2.46	0.304	3.01E-06	5.93E-06	5.93E-06	4.43E-05	5.4
12C	93,409	48,410	2	3.04	0.323	6.27E-06	1.23E-05	1.23E-05	9.23E-05	8.6

Variables

		k (cm/sec)	Porosity %	FC %	WP %	Ψb	λ	θr	*i*	A (ft^2)
Soil Type	SM	5.20E-04	0.473	0.222	0.104	0.013	0.27	0.04	1	1

$A = \psi_b \qquad B = \lambda$

$$\left(\frac{A}{0.33}\right)^B = 0.42, \qquad \left(\frac{A}{15}\right)^B = 0.15$$

$$B \log A - B \log 0.33 = \log 0.42$$
$$B \log A - B \log 15 = \log 0.15$$

$$B \log A = \log 0.42 + B \log 0.33$$
$$B \log A = \log 0.15 + B \log 15$$

$$\log 0.42 + B \log 0.33 = \log 0.15 + B \log 15$$
$$B \log 0.33 - B \log 15 = \log 0.15 - \log 0.42$$
$$B (\log 0.33 - \log 15) = \log 0.15 - \log 0.42$$

$$B = \frac{\log 0.15 - \log 0.42}{\log 0.33 - \log 15} = \frac{-0.447}{-1.658} = 0.27$$

$$(0.27) \log A = \log 0.42 + (0.27) \log 0.33$$

$$\log A = \frac{-0.507}{+0.27} = -1.877$$

$$A = 10^{-1.877} = 0.013$$

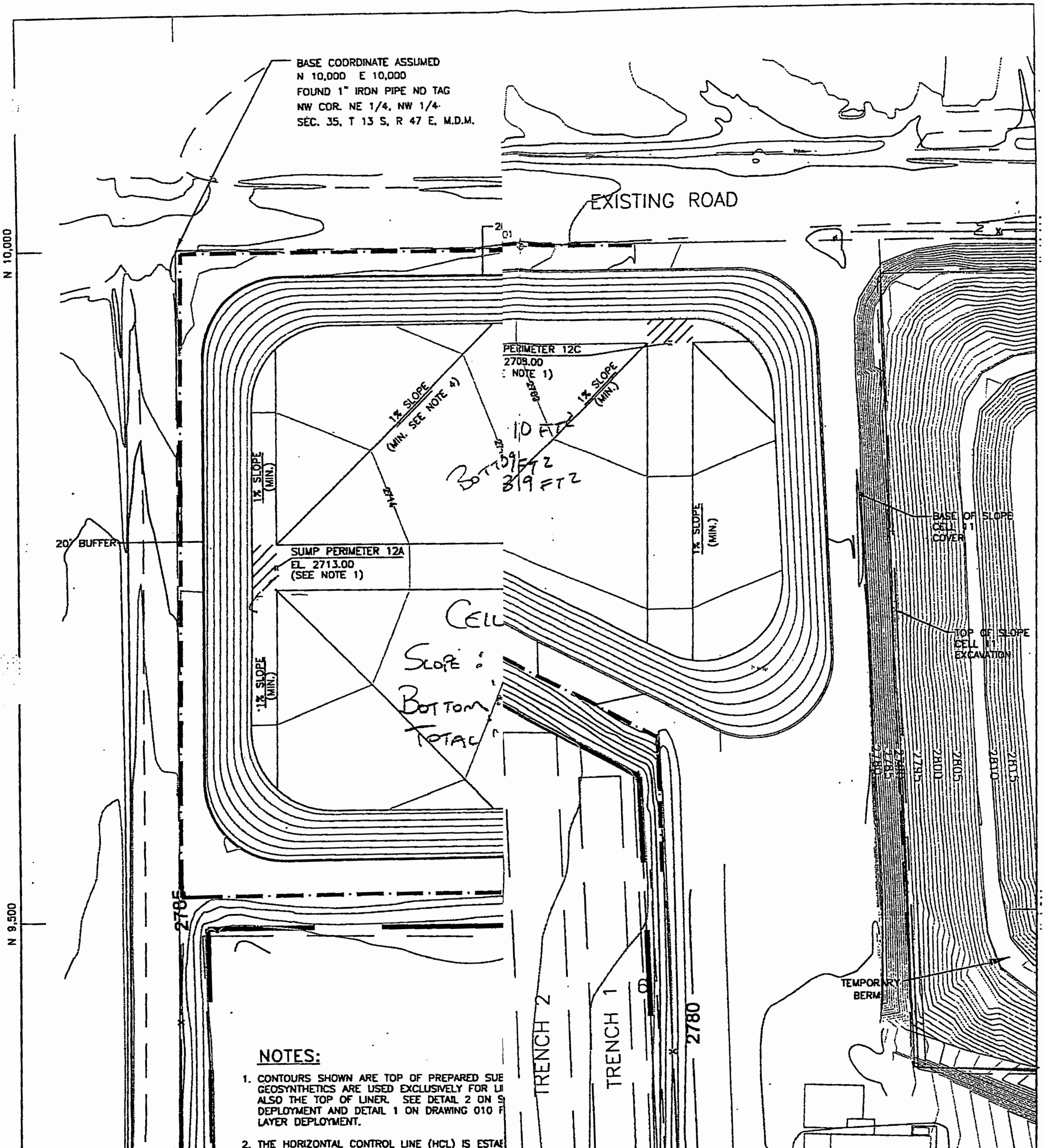
$\psi_b = 0.013$

$\lambda = 0.27$

REFERENCE

AREAS FROM

2007 DESIGN DRAWINGS



NOTES:

1. CONTOURS SHOWN ARE TOP OF PREPARED SUB GEOSYNTHETICS ARE USED EXCLUSIVELY FOR LI ALSO THE TOP OF LINER. SEE DETAIL 2 ON S DEPLOYMENT AND DETAIL 1 ON DRAWING 010 F LAYER DEPLOYMENT.
2. THE HORIZONTAL CONTROL LINE (HCL) IS ESTAB BUFFERS. THE HCL IS THE LIMIT OF EXCAVATI REQUIRED TO REPLACE LOOSE SURFACE SOIL A
3. ALL SUBGRADE EXCAVATION SIDE SLOPES ARE EXCEPT AT CURVED SECTIONS WHERE A CONST FROM THE TOP OF SLOPE (i.e., THE HCL).
4. THE MINIMUM BOTTOM SLOPE, TOWARDS THE SU INDICATED.
5. PROVIDE LOCAL GRADING AS NECESSARY TO PR FROM THE HCL.
6. DURING FILLING OF CELL 12, PROVIDE A BERM 12 TOP OF SLOPE TO PREVENT RUN-ON ONTO

DRWN. BY	DSGN. BY	CHECK BY	APP'D BY	DATE

US ECOLOGY INC.

PROJECT NUMBER: 073113 DATE: 5/17/07

BEATTY NEVADA
HAZARDOUS WASTE MANAGEMENT FACILITY

Figure 1
EXCAVATION PLAN

CHKD.	DATE	APP'D	SCALE

REFERENCE

HELP MODEL SOIL VALUES

THE HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE (HELP) MODEL

ENGINEERING DOCUMENTATION FOR VERSION 3

by

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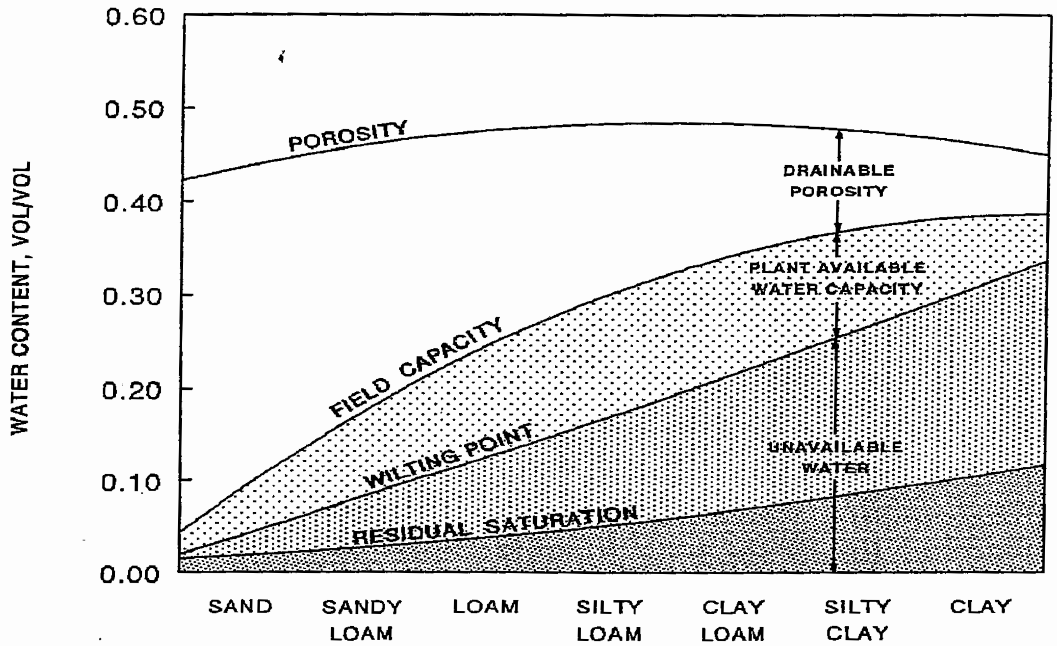


Figure 2. Relation Among Moisture Retention Parameters and Soil Texture Class

are not specified, the program assumes values near the steady-state values (allowing no long-term change in moisture storage) and runs a year of simulation to initialize the moisture contents closer to steady state. The soil water contents at the end of this year are substituted as the initial values for the simulation period. The program then runs the complete simulation, starting again from the beginning of the first year of data. The results of the volumetric water content initialization period are not reported in the output.

3.3.2 Unsaturated Hydraulic Conductivity

Darcy's constant of proportionality governing flow through porous media is known quantitatively as hydraulic conductivity or coefficient of permeability and qualitatively as permeability. Hydraulic conductivity is a function of media properties, such as particle size, void ratio, composition, fabric, degree of saturation, and the kinematic viscosity of the fluid moving through the media. The HELP program uses the saturated and unsaturated hydraulic conductivities of soil and waste layers to compute vertical drainage, lateral drainage and soil liner percolation. The vapor diffusivity for geomembranes is specified as a saturated hydraulic conductivity to compute leakage through geomembranes by vapor diffusion.

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):

$$K_u = K_s \left[\frac{\theta - \theta_r}{\phi - \theta_r} \right]^{3+\left(\frac{2}{\lambda}\right)} \tag{5}$$

where

K_u = unsaturated hydraulic conductivity, cm/sec

K_s = saturated hydraulic conductivity, cm/sec

θ = actual volumetric water content, vol/vol

θ_r = residual volumetric water content, vol/vol

ϕ = total porosity, vol/vol

λ = 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:

$$\theta_r = \begin{cases} 0.014 + 0.25\,WP & \text{for } WP \geq 0.04 \\ 0.6\,WP & \text{for } WP < 0.04 \end{cases} \tag{6}$$

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):

$$\frac{\theta - \theta_r}{\phi - \theta_r} = \left(\frac{\psi_b}{\psi}\right)^{\lambda} \tag{7}$$

where

ψ = capillary pressure, bars

ψ_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:

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	1.0×10^{-2}
2	S	SW	0.437	0.062	0.024	5.8×10^{-3}
3	FS	SW	0.457	0.083	0.033	3.1×10^{-3}
4	LS	SM	0.437	0.105	0.047	1.7×10^{-3}
5	LFS	SM	0.457	0.131	0.058	1.0×10^{-3}
6	SL	SM	0.453	0.190	0.085	7.2×10^{-4}
7	FSL	SM	0.473	0.222	0.104	5.2×10^{-4}
8	L	ML	0.463	0.232	0.116	3.7×10^{-4}
9	SiL	ML	0.501	0.284	0.135	1.9×10^{-4}
10	SCL	SC	0.398	0.244	0.136	1.2×10^{-4}
11	CL	CL	0.464	0.310	0.187	6.4×10^{-5}
12	SiCL	CL	0.471	0.342	0.210	4.2×10^{-5}
13	SC	SC	0.430	0.321	0.221	3.3×10^{-5}
14	SiC	CH	0.479	0.371	0.251	2.5×10^{-5}
15	C	CH	0.475	0.378	0.251	2.5×10^{-5}
21	G	GP	0.397	0.032	0.013	3.0×10^{-1}

a = constant representing the effects of various fluid constants and gravity, 21 cm^3/sec

ϕ = total porosity, vol/vol

θ_r = residual volumetric water content, vol/vol

ψ_b = bubbling pressure, cm

λ = 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.

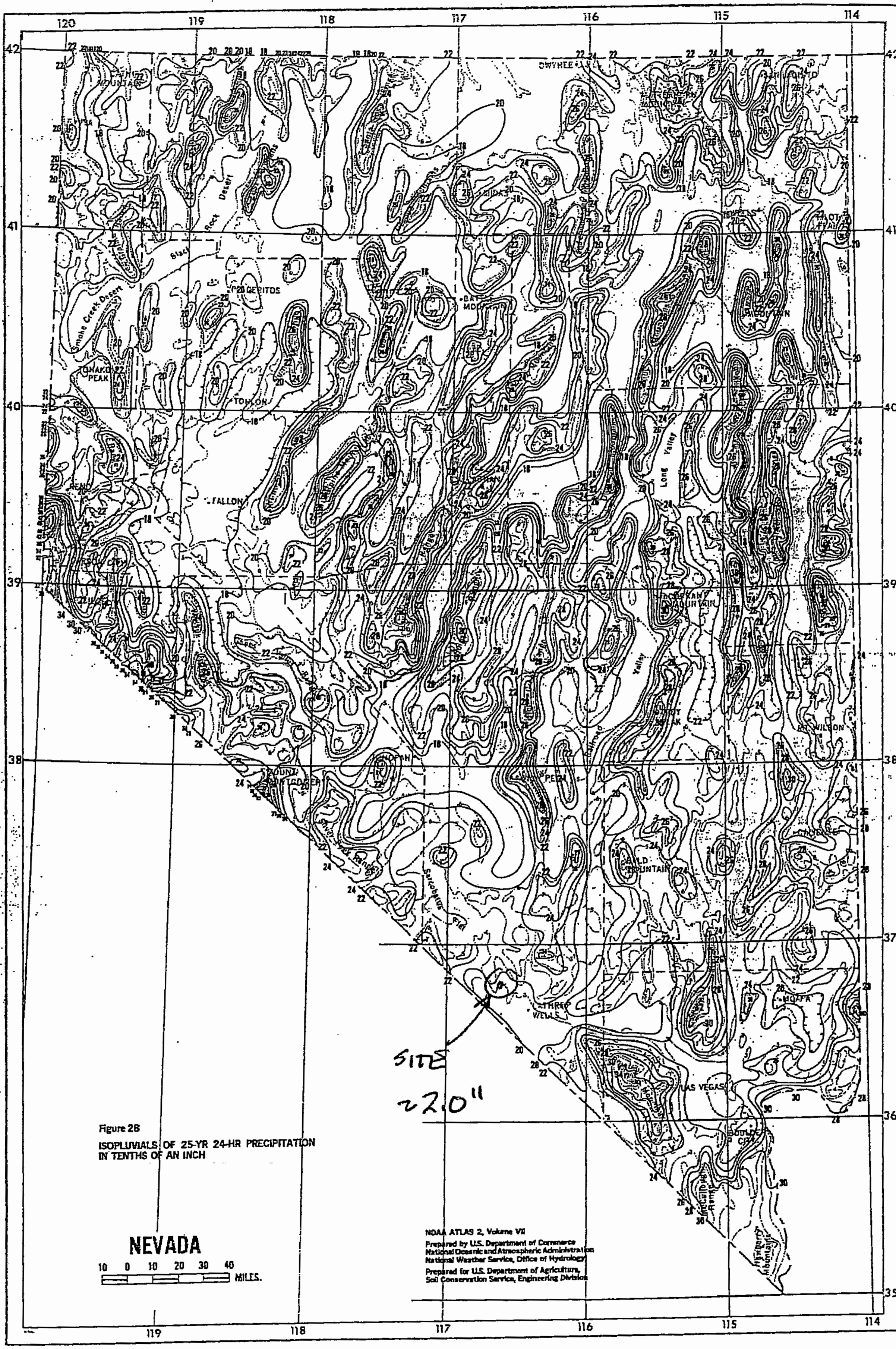


Figure 2B
ISOPLUVIALS OF 25-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

REFERENCE

1996 DESIGN CALCULATION

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	1.2×10^{-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:

$$2 ft^3 \times 7.48 \frac{gal}{ft^3} \times (0.398 - 0.244) = 2.3 gal/ft^2$$

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:

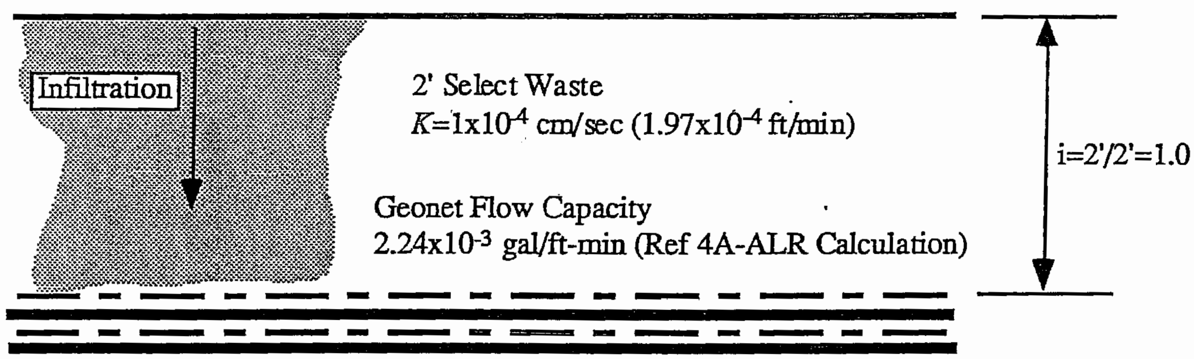
$$Q = Kia \qquad (1)$$

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, Θ_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, λ, 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, ψ, 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:

Θr	0.048			
φ	0.398			
Variables	Θ	ψ	Θ–Θr/φ–Θr	(ψb/ψ)λ
Field Capacity	0.244	0.33	0.56	0.56
Wilting Point	0.136	15	0.25	0.25
Solved Values				
ψb	0.021			
λ	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 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/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 2.24×10^{-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.

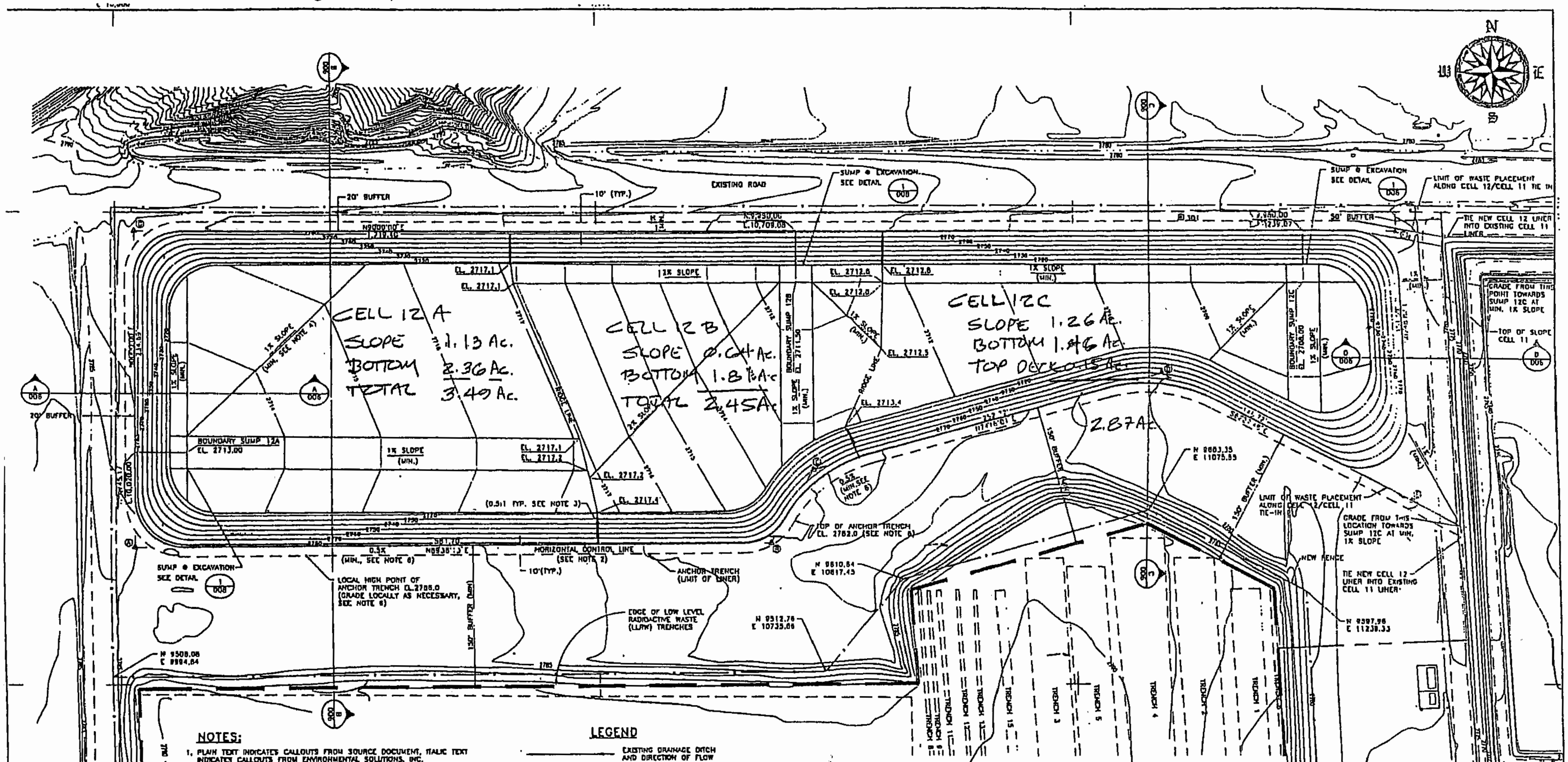
5. Construction of the starter berms, buttress fills and cover system will begin once waste filling is within 3 feet of the top of the excavation slope. The maximum waste area of (8.9 acres) within the horizontal control line (Figure 1) is potentially exposed to precipitation. As a conservative estimate 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).

2' | 1 1/2' 0.5' ← ~35' • 2 MIDHEIGHT PERIMETER (MHP)

35' × 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 (NY-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

50 25 0 50
SCALE FEET

DATE OF AERIAL TOPOGRAPHY: JUNE 1994

SURVEY CONTROL TABLE						
CURVE	COORDINATES		RADIUS	LENGTH	TANGENT	DELTA
A	N 9,844.99	E 10,020.00	50.00'	78.86'	50.32'	90°21'42"
B	N 9,849.18	E 10,680.76	50.00'	52.05'	28.66'	59°38'42"
C	N 9,220.63	E 10,725.59	150.00'	115.91'	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°20'44"
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"

Rev	Drawn	Chect.	Appr	Date	Description
3	DKK			5/95	REDESIGN OF CELL 12 ABOVE GRADE
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
NY-148-TRE-003

BOB JONES

HAZARDOUS WASTE MANAGEMENT FACILITY
U.S. ECOLOGY

EXCAVATION PLAN AND LIMITS OF GRADING CELL 12

ENVIRONMENTAL SOLUTIONS, INC.

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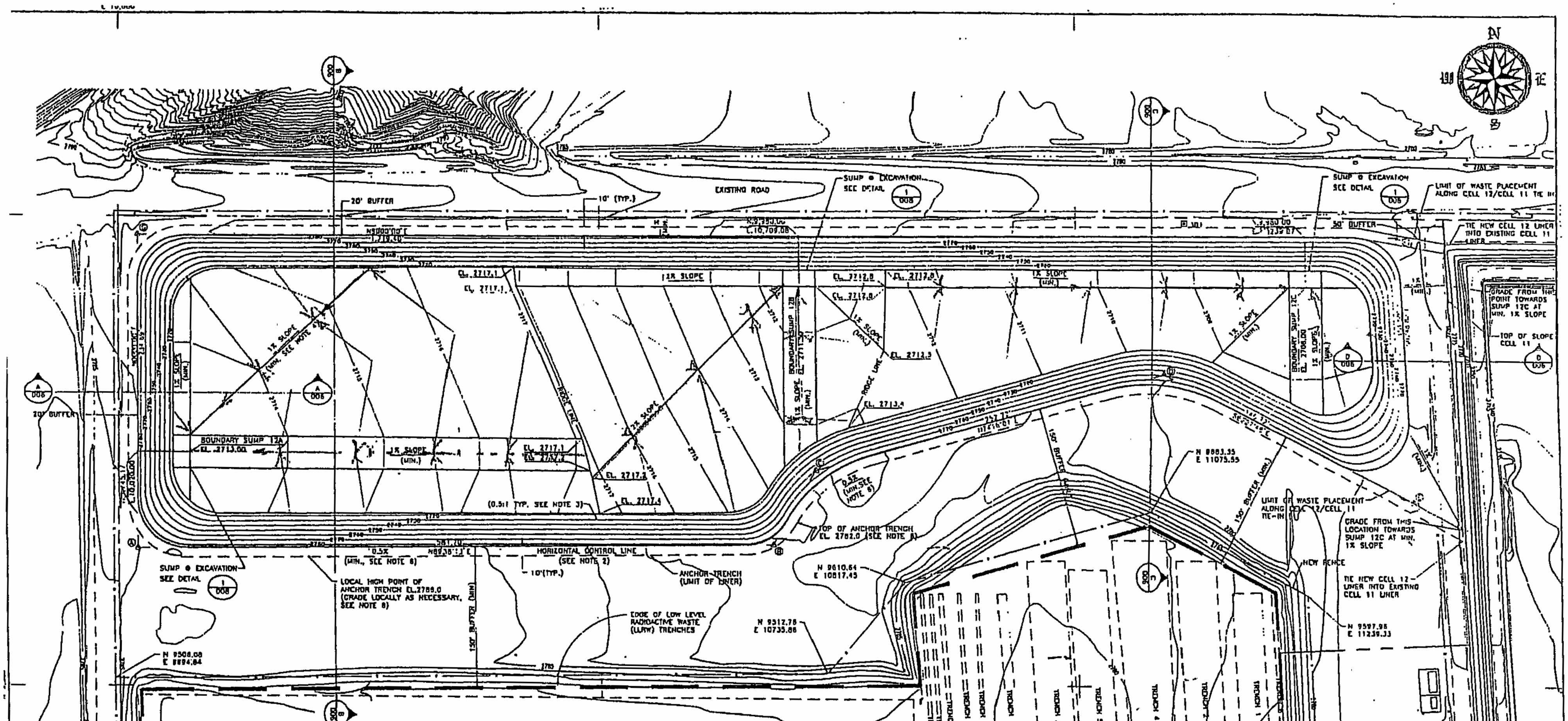
Figure 1

SHEET 10 of 11



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- UNIMPROVED ROAD
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- 1' EXISTING CONTOUR INTERVAL, FEET (MSL)
- SITE COORDINATE SYSTEM MARKER LOCATION IN NORTHING AND EASTING
- • 302 MONITOR WELL LOCATION AND NUMBER

SURVEY CONTROL TABLE

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SCALE 50 25 0 50 FEET

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5333 WESTHEIMER
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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.

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FIGURE 3

ENVIRONMENTAL SOLUTIONS, INC.

By RVH Date 1/20/96 Subject BEATTY LANDFILL-CELL 12 Sheet No. 11 of 11

Chkd. By [illegible] Date 1/23/96 AMERICAN ECOLOGY CORP. Proj. No. 95-284

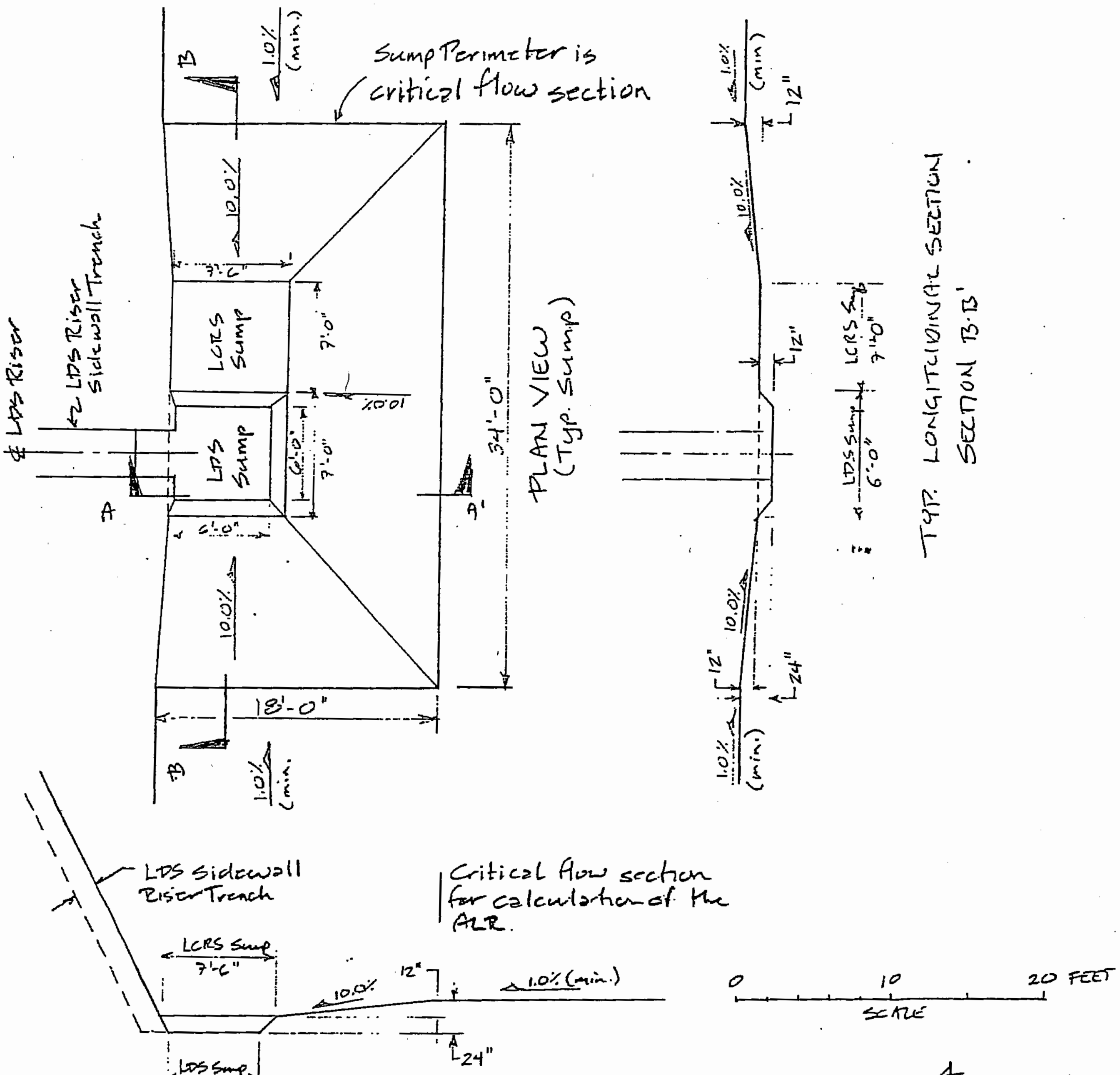

FIGURE 4

Notes:

1) Liner elements & riser pipes not shown for clarity.

TYPICAL SUMP GEOMETRY

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 (ℓ) = 21,650 ft.
3. Avg. land slope (γ) = $^{160}/(3.87)\ (5280)$ = 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 .04 cfs/ac ?