APPENDIX I
SOIL EROSION ANALYSES

Objective:
To estimate and evaluate the soil loss due to surface water erosion from the proposed final closure cover slopes of Landfill B-18.

Given:
The proposed final closure cover slopes of Landfill B-18 will generally consist of benches every 50 vertical feet (maximum) with 3.5H:1V (horizontal:vertical) slopes between benches. Therefore, the worst case cover slope with regard to soil erosion is:

Slope Inclination = 3.5H:1V

Slope = $\frac{1}{3.5}$ = 28.6%

Slope Vertical Height = 50 feet

Slope Horizontal Length = 50×3.5 = 175 feet

Slope Length = $\sqrt{50^2 + 175^2}$ = 182 feet

This worst case cover slope was analyzed as described below.
Based on guidance from the United States Environmental Protection Agency (USEPA, 1989), the soil erosion loss should be less than 2 tons/acre/year (t/ac/yr), as shown in Attachment #1.

Method:
In order to estimate the amount of soil loss on the Landfill B-18 cover due to water erosion, the Revised Universal Soil Loss Equation (RUSLE) was used. The RUSLE was developed by the United States Department of Agriculture, Natural Resources Conservation Service (NRCS) to estimate sheet-rill (both rill and inter-rill) erosion and it considers soil and vegetation type as well as physical and climatic features of the landfill area. The RUSLE is expressed mathematically as:

$$a = r \times k \times l \times s \times c \times p$$

where:

a = daily soil loss due to erosion (units of tons/acre/day);
r = rainfall and runoff erosivity factor;
k = soil erodibility factor;
l = slope length factor;
s = slope steepness factor;
c = cover-management factor; and
p = supporting practices factor.

The daily values of soil erosion loss ("a" values) are summed over an entire year to calculate the estimated annual soil erosion loss (in t/ac/yr). These soil erosion calculations are best made using a computer program.

The RUSLE – Version 2 (RUSLE2) computer program, developed by the NRCS, was used to calculate the potential soil erosion loss from the Landfill B-18 worst-case cover slope described above. The RUSLE2 program was downloaded from the following URL:

http://fargo.nserl.purdue.edu/rusle2_dataweb/RUSLE2_Index.htm

Assumptions:

Climate: The default climate "CA_Kings_R6" was selected from the Kings County climate zones in RUSLE2 as most accurately representing the climate at the Kettleman Hills Facility. The CA_Kings_R6 climate has an annual precipitation of 6.9 inches. For the period of July 1948 through December 2001, the mean annual precipitation for the site was 6.82 inches according to data obtained from the Western Regional Climate Center data base for the Kettleman Climatological Station (see Attachment #2).

Soils: The NRCS has identified three types of soils at the Kettleman Hills Facility. These soils are: Kettleman Loam (5-15% slopes), Kettleman Loam (15-30% slopes), and Kettleman-Cantua Complex (30-50% slopes). The properties of these three soil types, as listed in the RUSLE2 program files, are as follows:

Soil Property	Kettleman Loam (5-15% Slopes)	Kettleman Loam (15-30% Slopes)	Kettleman-Cantua (30-50% Slopes)
Sand Content	40%	40%	40%
Silt Content	38%	38%	38%
Clay Content	23%	23%	23%
Erodibility Factor (k)	0.37	0.37	0.37
RUSLE2 Soil No.	#127	#128	#129

Based on the similarity of the above-listed soils at the Kettleman Hills Facility, Kettleman Loam (15-30% Slopes, RUSLE2 Soil No. 128) was selected to model the Landfill B-18 cover soil. Sensitivity analyses indicated that the three soil types listed above result in the same calculated soil loss if all other variables are held constant.

Calculations:

The soil erosion calculations were performed using the RUSLE2 computer program downloaded from the above-listed URL. The calculations were performed for two scenarios:

1) Bare Slope – the final cover slope was modeled as a construction site with bare ground.
2) Vegetated Slope – the final cover slope was modeled as having permanent ground cover consisting of warm season grass that is not harvested. The amount of grass canopy was reduced 50% from the default (base) value, which increased the amount of calculated erosion.

Results:
The results of the RUSLE2 calculations are shown in Attachment #3, which contains the RUSLE2 Worksheet Erosion Calculation Record sheets for each of the two scenarios analyzed. A summary of the results is given in the table below:

Scenario	Final Cover Slope Inclination	Computed Soil Erosion Loss (t/ac/yr)
Bare Ground	3.5H:1V	9.2
Vegetated Ground	3.5H:1V	0.97

Conclusions:
Once vegetation is established on the Landfill B-18 final cover slopes, the amount of soil erosion loss is estimated to be approximately half of the recommended maximum of 2 t/ac/yr (USEPA, 1989). As discussed in the RUSLE2 program documentation, the results obtained from RUSLE2 are to be used as guides in evaluating soil erosion potential and mitigation measures for soil erosion. As shown in the current case for Landfill B-18, maintaining vegetative cover on the final landfill sideslopes is essential for limiting soil erosion loss to an acceptable amount.

Commentary:
Golder has been providing engineering services at the Kettleman Hills Facility for over 20 years, including over 10 years of annual post-closure inspections. During this period, several landfill units have been closed. Unit B-13, Unit B-15, and the Combined Closure Area (totaling over 100 acres) were closed in the early- to mid- 1990's. Based on observations of the final cover slopes of these areas, soil erosion does not appear to be a problem at the Kettleman Hills Facility when vegetation is present.

Reference:

United States Environmental Protection Agency, "Technical Guidance Document: Final Covers on Hazardous Waste Landfills and Surface Impoundments," EPA/530-SW-89-047, July 1989.

ATTACHMENT #1
SOIL EROSION ANALYSES

Attachment #1 p. 1/6

United States Environmental Protection Agency

Office of Solid Waste and Emergency Response Washington DC 20460

EPA/530-SW-89-047
July 1989

EPA

Technical Guidance Document:

Final Covers on Hazardous Waste Landfills and Surface Impoundments

Attachment #1 p. 4/6

EPA/530-SW-89-047
July 1989

TECHNICAL GUIDANCE DOCUMENT

FINAL COVERS ON HAZARDOUS WASTE LANDFILLS AND SURFACE IMPOUNDMENTS

Office of Solid Waste and Emergency Response
U.S. Environmental Protection Agency
Washington, DC 20460

In cooperation with

RISK REDUCTION ENGINEERING LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

Attachment #1 p. 3/6

Table 2. Synopsis of Minimum Technology Guidance for Covers

Layer	Thickness	Slope	Requirements
Top Layer			
Vegetation	--	--	Persistent, drought-resistant, adapted to local conditions.
OR			
Surface Armor	5-10 in. (13-25 cm)		Cobbles, gravel.
ON			
Soil	$\geq$ 24 in. ($\geq$ 60 cm)	3-5%	Erosion rate <2 ton/acre/yr (5.5 MT/ha/yr). *
Drainage Layer			
Soil	$\geq$ 12 in. ($\geq$ 30 cm)	$\geq$ 3%	SP (USCS) soil $K > 1 \times 10^{-2}$ cm/s; gravel toe drain.
OR			
Geosynthetic	variable	$\geq$ 3%	Performance equivalent to soil, hydraulic transmissivity $\geq 3 \times 10^{-5}$ m^2/sec.
Low-Permeability Layer			
FML	$\geq$ 20 mils ($\geq$ 0.5 mm)	$\geq$ 3%	In EPA Report No. EPA 600/2-88-052.
ON			
Low-Permeability Soil	$\geq$ 24 in. ($\geq$ 60 cm)	$\geq$ 3%	In-place $K < 1 \times 10^{-7}$ cm/s and test fill.
Optional Layers (site-specific design)			
Gas Vent Layer	$\geq$ 12 in. ($\geq$ 30 cm)	$\geq$ 2%	Similar to drainage layer.
Biotic Barrier	animal or root-dependent	--	Large materials, e.g., cobbles.

Attachment #1 p. 7/6

2. TOP LAYER

The Agency recommends a two-component top layer for a landfill cover system (Figure 1). The upper component should be vegetation or other surface treatment, designed to impede erosion but allowing surface runoff from major storm events. The Agency believes that, in most cases, vegetation underlain by soil, at least part of which is topsoil, will best accomplish these objectives. However, in some areas the prevailing climate may inhibit the establishment and maintenance of vegetation, or a planned alternative use of the site may preclude vegetation. In those cases, an armored surface without vegetation (Figure 2), and underlain by fill soil, might be used if it will minimize erosion and abrasion of the cover and allow, to the maximum practicable extent, surface drainage off the cover.

2.1 DESIGN

The Agency recommends that the vegetation component of the top layer meet the following specifications:

- o Locally adapted perennial plants.
- o Resistant to drought and temperature extremes.
- o Roots that will not disrupt the low-permeability layer.
- o Capable of thriving in low-nutrient soil with minimum nutrient addition.
- o Sufficient plant density to minimize cover soil erosion to no more than 2 tons/acre/year (5.5 MT/ha/yr), calculated using the USDA Universal Soil Loss Equation. *
- o Capable of surviving and functioning with little or no maintenance.

In landfill situations where the environment or other considerations make it inappropriate for maintaining sufficiently dense vegetation, armoring material may be substituted as the upper component of the top layer or in rare cases the whole layer. It is recommended that the material possess the following characteristics:

- capable of remaining in place and minimizing erosion of itself and the underlying soil component during extreme weather events of rainfall and/or wind;

- capable of accommodating settlement of the underlying material without compromising the purpose of the component;

- surface slope approximately the same as the underlying soil (at least 3 percent slope); and

- capable of controlling the rate of soil erosion from the cover to no more than 2 tons/acre/year (5.5 MT/ha/yr), calculated by using the USDA Universal Soil Loss Equation. *

Agency-recommended specifications for the lower soil component of the top layer include the following:

- for vegetation support, a minimum thickness of 60 cm (24 in.) including at least 15 cm (6 in.) of topsoil (soil of lower quality may be used beneath an armored surface); greater total thickness where required, e.g., where maximum frost penetration exceeds this depth, or where greater plant-available water storage is necessary or desirable;

- medium texture to facilitate seed germination and plant root development;

- final top slope, after allowance for settling and subsidence, of at least 3 percent, but no greater than 5 percent, to facilitate runoff while minimizing erosion; and

- minimum compaction to facilitate root development and sufficient infiltration to maintain growth through drier periods.

The owner or operator of the landfill should prepare a separate section specific to monitoring construction of the top layer to be included in the construction quality assurance (CQA) plan.

2.2 DISCUSSION

2.2.1 Upper Component of Top Layer

As noted in the design recommendations above, the upper component of the top layer may be vegetation (Agency-preferred where possible) or other erosion-impeding materials. These are discussed separately below.

Attachment #1 p. 6/6

and, in general, increase the long-term maintenance of the cover system. Owners and operators using final slopes based on site-specific conditions should determine that the slopes will not result in the formation of erosion rills and gullies and will limit total erosion to less than 2.0 tons/acre/year (5.5 MT/ha/yr). The U.S. Department of Agriculture's Universal Soil Loss Equation (USLE) is recommended as the tool for use in evaluating erosion potential (EPA, 1982a). The Agency believes that a maximum erosion rate of 2.0 tons/acre/year (5.5 MT/ha/yr) is realistically achievable for a wide range of soils, climates, and vegetation. The Agency also believes that reliance on this criterion will minimize gully development and cover maintenance. *

ATTACHMENT #2
SOIL EROSION ANALYSES

KETTLEMAN STN, CALIFORNIA (044536)

Period of Record Monthly Climate Summary

Period of Record : 7/ 1/1948 to 12/31/2001

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	55.2	61.9	67.3	75.1	84.3	93.1	99.2	97.1	91.3	80.7	66.6	55.9	77.3
Average Min. Temperature (F)	38.5	43.3	45.8	50.0	56.2	63.3	69.2	67.9	63.7	56.2	46.8	39.5	53.4
Average Total Precipitation (in.)	1.42	1.37	1.13	0.61	0.29	0.05	0.01	0.03	0.19	0.28	0.64	0.80	6.82
Average Total SnowFall (in.)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Average Snow Depth (in.)	0	0	0	0	0	0	0	0	0	0	0	0	0

Percent of possible observations for period of record.
Max. Temp.: 60.6% Min. Temp.: 60.1% Precipitation: 97.2% Snowfall: 97.4% Snow Depth: 97.4%
Check Station Metadata or Metadata graphics for more detail about data completeness.

Western Regional Climate Center, wrcc@dri.edu

Attachment #2 p. 1/1

ATTACHMENT #3
SOIL EROSION ANALYSES

NRCS Natural Resources Conservation Service

RUSLE2 Worksheet Erosion Calculation Record

Info: Kettleman Hills Facility - Landfill Unit B-18
3.5H:1V, 50-foot-tall final cover slopes (bare)

Inputs:

Owner name	Location
Waste Management, Inc.	California\Kings County\CA_Kings_ R6

Location	Soil	Slope length (horiz)	Avg. slope steepness, %
California\Kings County\CA_Kings_ R6	128 KETTLEMAN LOAM, 15 TO 30 PERCENT SLOPES\KETTLEMAN loam 85%	182	28.6

Outputs:

Management	Contouring	Strips / barriers	Diversion/terrace, sediment basin	Soil loss erod. portion, t/ac/yr	Soil detachment, t/ac/yr	Cons. plan. soil loss, t/ac/yr	Sed. delivery, t/ac/yr
Bare ground	a. rows up-and-down hill	(none)	(none)	9.2	9.2	9.2	9.2

Attachment #5
p. 1/2

NRCS Natural Resources Conservation Service

RUSLE2 Worksheet Erosion Calculation Record

Info: Kettleman Hills Facility - Landfill Unit B-18
3.5H:1V, 50-foot-tall final cover slopes (vegetated)

Inputs:

Owner name	*Location*
Waste Management, Inc.	**California\Kings County\CA_Kings_ R6**

Location	*Soil*	*Slope length (horiz)*	*Avg. slope steepness, %*
California\Kings County\CA_Kings_ R6	128 KETTLEMAN LOAM, 15 TO 30 PERCENT SLOPES\KETTLEMAN loam 85%	182	28.6

Outputs:

Management	*Contouring*	*Strips / barriers*	*Diversion/terrace, sediment basin*	*Soil loss erod. portion, t/ac/yr*	*Soil detachment, t/ac/yr*	*Cons. plan. soil loss, t/ac/yr*	*Sed. delivery, t/ac/yr*
Warm season grass; not harvested	a. rows up-and-down hill	(none)	(none)	0.97	0.97	0.97	0.97

Attachment #3
p. 2/2

APPENDIX J
SURFACE WATER DRAINAGE ANALYSES

APPENDIX J.1

PHASES I AND II HYDROLOGY AND DESIGN CRITERIA

ENVIRONMENTAL SOLUTIONS, INC.

By TJM Date 8-15-90 Subject LANDFILL B-18 Sheet No. 1 of 59
Chkd. By ___ Date ___ DRAINAGE DESIGN Proj. No. 89.07?

TABLE OF CONTENT

ENVIRONMENTAL SOLUTIONS, INC.

By TPL Date 7-24-90 Subject LANDFILL B-18 DRAINAGE SYSTEM Sheet No. 2 of 59

Chkd. By N.A. Date 8-14-90 Proj. No. 89-977

PURPOSE : DESIGN LANDFILL B-18 DRAINAGE SYSTEM

DESIGN CRITERIA : 1. MAJOR DRAINAGE FEATURES ARE SIZED FOR PROBABLE MAXIMUM PRECIPITATION.

2. LESS CRITICAL DRAINAGE FEATURES ARE SIZED FOR 25-YEAR STORM.

METHODOLOGY : MAXIMUM DISCHARGE FOR DIVERSION DITCHES AND CULVERTS ARE DETERMINED BY THE RATIONAL METHOD, WHICH TAKES THE FORM OF

$$Q = C i A$$

WHERE

Q = MAXIMUM DISCHARGE (CFS)
i = RAINFALL INTENSITY (IN/HR)
A = DRAINAGE AREA
C = RUNOFF COEFFICIENT

DIVERSION PIPES/DITCHES WILL BE SIZED USING THE MANNING'S FORMULA WHICH TAKES THE FORM OF

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$ WHERE

R = HYDRAULIC RADIUS
S = DITCH BED SLOPE
Q = MAXIMUM DISCHARGE CAPACITY
n = ROUGHNESS COEFFICIENT
A = EFFECTIVE DRAINAGE AREA.

ENVIRONMENTAL SOLUTIONS, INC.

By vpi Date 7-24-90 Subject LANDFILL B-18, RUN ON & RUNOFF CONTROL Sheet No. 3 of 59

Chkd. By N.A. Date 8-14-90 Proj. No. 89-977

DETERMINATION OF TIME OF CONCENTRATION

TIME OF CONCENTRATION MAY BE DETERMINED BY

1. $T_c = \left(\frac{11.9\ L^3}{H} \right)^{0.385}$ (REFERENCE)

WHERE

T_c = TIME OF CONCENTRATION
L = LENGTH OF FLOW PATH (MILES)
H = ELEVATION DIFFERENCE (FEET)

OR

2. $T_c = L \Big/ \left(\frac{1.486}{n} R^{2/3} S^{1/2} \right)$

WHERE

L = LENGTH OF FLOW PATH
n = ROUGHNESS COEFFICIENT
R = HYDRAULIC RADIUS
S = DITCH BED SLOPE

DETERMINATION OF RUNOFF COEFFICIENT

IF THE DRAINAGE BASIN UNDER CONSIDERATION CONSISTS OF MATERIALS WITH DIFFERENT SURFACE CHARACTERISTICS, A COMPOSITE RUNOFF COEFFICIENT WILL BE USED. THE COMPOSITE RUNOFF COEFFICIENT C_c MAY BE DETERMINED BY:

$$C_c = \frac{\Sigma C_i A_i}{\Sigma A_i}$$

WHERE C_i & A_i IS THE RUNOFF COEFFICIENT AND THE SURFACE AREA FOR EACH INDIVIDUAL MATERIAL, RESPECTIVELY. TYPICAL RUNOFF COEFFICIENT VALUES FOR DIFFERENT MATERIALS ARE SHOWN IN EXHIBIT 1.

WITH THE ON-SITE MATERIAL IS SIMILAR TO THE TYPE C SOIL IN SOLANO COUNTY, A RUNOFF COEFFICIENT C OF 0.4 IS USED

ENVIRONMENTAL SOLUTIONS, INC.

By VPL Date 8-12-90 Subject LANDFILL B-18 Sheet No. 4 of 59
Chkd. By N.A. Date 8-14-90 DRAINAGE DESIGN Proj. No. 89-977

CULVERT DESIGN IS BASED ON INLET CONTROL MONOGRAPH (EXHIBIT 2.).

SHORT-TERM RAINFALL INTENSITY / DURATION RELATIONSHIPS ARE BASED ON PMP RECORD FOR COALINGA STATION (EXHIBIT 3) THE RAINFALL INTENSITY DURATION CURVE IS SHOWN ON EXHIBIT 4. EXHIBIT 5 ALSO SHOWS THE THE 25-YEAR STORM RAINFALL INTENSITY / DURATION RELATIONSHIPS FOR COALINGA STATION FOR CULVERT DESIGN. EXHIBIT 6 SHOWS THE RELATIVE LOCATION BETWEEN THE SITE AND HE COALINGA STATION.

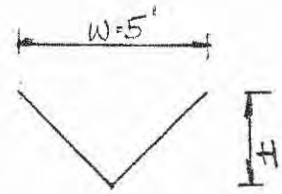
EXHIBIT 7 SHOWS THE PRECIPITATION DEPTH - DURATION - FREQUENCY DATA USED TO DEVELOPE THE RAINFALL INTENSITY CURVE.

ENVIRONMENTAL SOLUTIONS, INC.

By TJR Date 7-31-90 Subject LANDFILL B+B DRAINAGE DESIGN Sheet No. 5 of 59
Chkd. By N.A. Date 8-14-90 Proj. No. 89.971

TRIANGULAR DITCH CAPACITY

(W=5 ft)



A_F = FLOW AREA (ASSUME FLOW FULL)

P_W = WETTED PERIMETER $= 2 \times \sqrt{2.5^2 + H^2}$

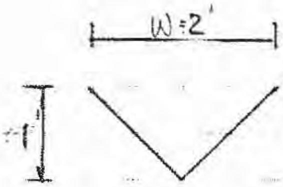
R = HYDRAULIC RADIUS = A_F / P_W

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

n = 0.013 FOR SMOOTH ASPHALT (EXHIBIT 9)

TYPE	H (FT)	A_F (FT²)	P_W (FT)	R (FT)	Q (CFS)	V (FPS)
2	1	2.5	5.38	0.46	$171.3 S^{1/2}$	$68.52 S^{1/2}$
3	1.25	3.13	5.59	0.56	$243.0 S^{1/2}$	$77.66 S^{1/2}$
4	1.5	3.75	5.83	0.64	$319.4 S^{1/2}$	$84.89 S^{1/2}$
5	2	5	6.40	0.78	$484.5 S^{1/2}$	$96.9 S^{1/2}$
6	2.5	6.25	7.07	0.88	$658.0 S^{1/2}$	$105.3 S^{1/2}$
7	3	7.5	7.81	0.96	$834.2 S^{1/2}$	$111.2 S^{1/2}$

BROW DITCH (see page 17)



W = 2 ft TYPE 1

$A_F = 1$ FT²

$P_W = 2 \times 1 \times \sqrt{2} = 2.83$ FT

R = 0.35 FT

$Q = 57.1\ S^{1/2}$ (CFS)

ENVIRONMENTAL SOLUTIONS, INC.

DROP INLET CAPACITY

USING THE ORIFICE EQUATION, DROP INLET CAPACITY MAY BE ESTIMATED AS

$$Q = CA\sqrt{2gh}$$

WHERE

C = INLET COEFFICIENT = 0.61
g = 32.2 ft/sec²
h = HEAD (ft)
A = PIPE AREA (ft²)

PIPE DIAMETER	Q (cfs)
12"	$3.78\sqrt{h}$
18"	$8.50\sqrt{h}$
24"	$15.1\sqrt{h}$
30"	$24.0\sqrt{h}$

ENVIRONMENTAL SOLUTIONS, INC.

By JPL Date 8-12-90 Subject LANDFILL B-18 DRAINAGE DESIGN Sheet No. 7 of 59

Chkd. By N.A Date 8-14-90 Proj. No. 81-977

FIGURES 1 AND 2 SHOW THE RUN-OFF AREAS FOR LANDFILL B-18 PHASE I AND PHASE II DEVELOPMENT. PAGES 10 THROUGH 12 TABULATED THE AREA, AND MAXIMUM DISCHARGE FOR THE RUN-OFF AREAS.

Page 0157

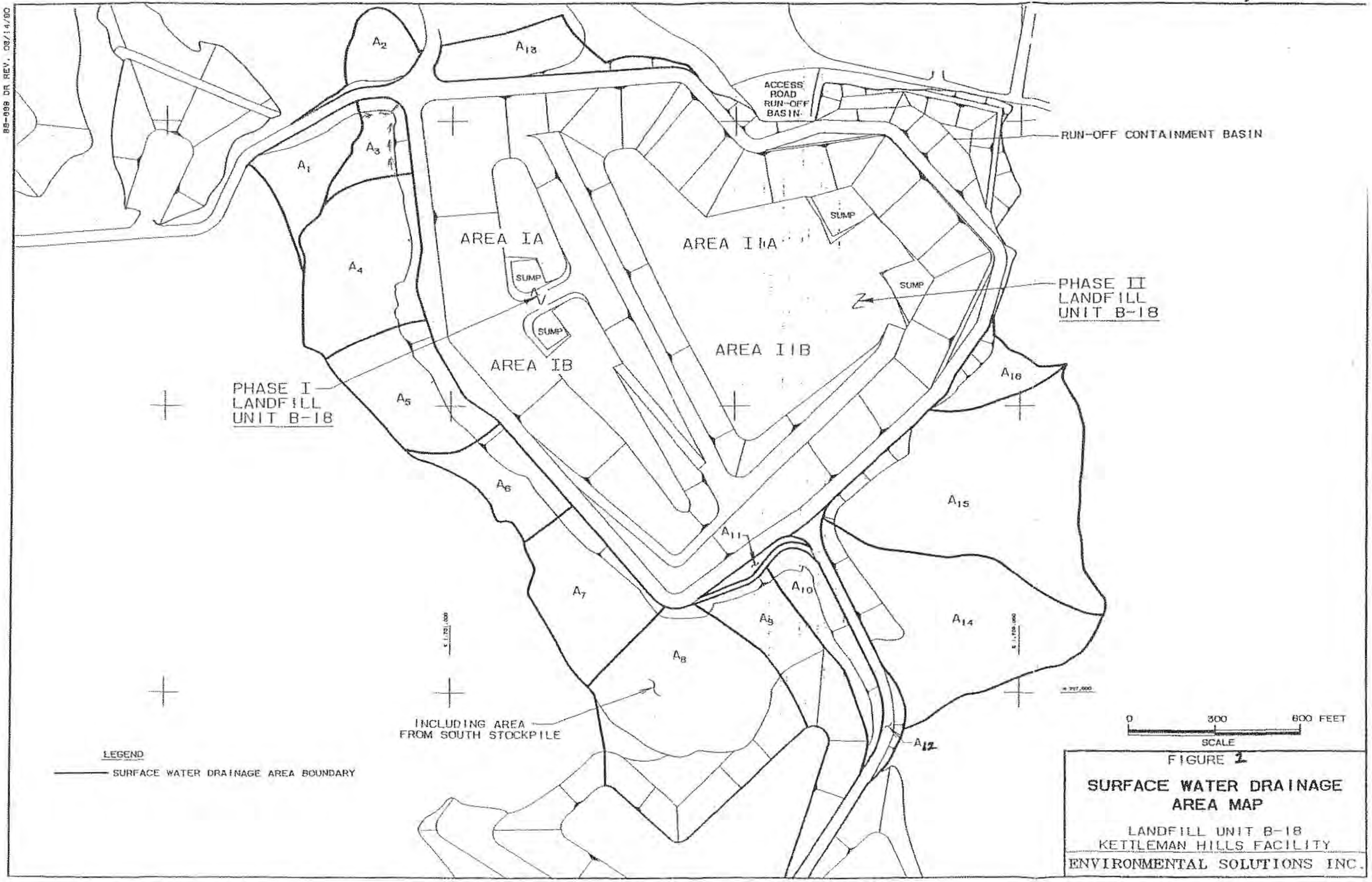


FIGURE 2

SURFACE WATER DRAINAGE AREA MAP

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS INC.

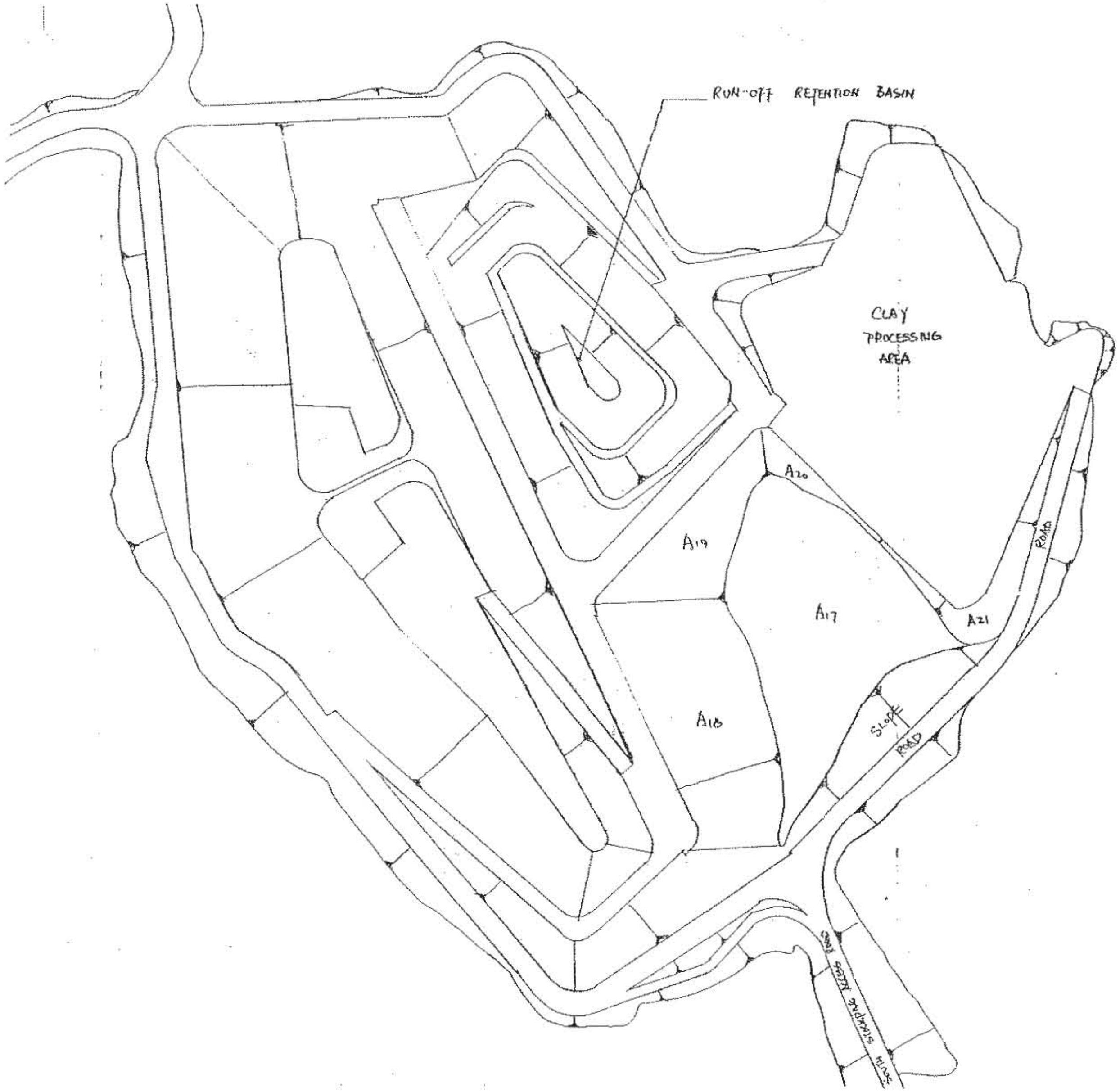


FIGURE 2
SURFACE WATER DRAINAGE
AREA MAP
LANDFILL UNIT 3-1B

ENVIRONMENTAL SOLUTIONS, INC.

Run-off **DRAINAGE DESIGN CALCULATION** for Phase 1 and Phase II

BY DPL DATE 8-6-90 SUBJECT LANDFILL E-18 DRAINAGE DESIGN SHEET NO 10 OF 59
CHKD N.A. DATE 8-14-90 PROJECT NO 89-977

DRAINAGE AREA	AREA (Ac)	LONGEST FLOW PATH (MI)	ELEVATION DIFFERENCE (Ft)	(see page 3) TIME OF CONCENTRATION (Min)	(see Exhibit 4) RAINFALL (2) INTENSITY (in/Hr)	RUNOFF COEFFICIENT	MAXIMUM DISCHARGE (cfs)
A1	1.86	0.1	94	2	7.92 (1)	0.4	5.9
CUTSLOPE	0.19	0.05	38		7.92	0.4	0.6
TOTAL	2.05						
A3	0.96	0.08	74	2	7.92 (1)	0.4	3.0
CUT SLOPE	0.29						
TOTAL	1.25						
A4	4.35	0.1	153	2	7.92 (1)	0.4	13.8
CUT SLOPE	0.55						
TOTAL	4.90						
A5	2.65	0.07	135	1	7.92 (1)	0.4	8.4
CUTSLOPE	0.73						
TOTAL	3.38						
A6	1.41	0.08	90	2	7.92 (1)	0.4	4.5
CUTSLOPE	0.62						
TOTAL	2.03						
A7	3.41	0.07	96	1	7.92 (1)	0.4	10.8
CUTSLOPE	0.36						
TOTAL	3.77						
A8	8.77	0.15	115	3	7.92 (1)	0.4	27.8
CUT SLOPE	0.14						
TOTAL	8.91						

Notes :
1. Assumed minimum time of concentration. (5 minutes)
2. See Figure 1.

DRAINAGE DESIGN CALCULATION

BY DPL DATE 8-6-90 SUBJECT LANDFILL B-18 SHEET NO 11 OF 59
CHKD N.A. DATE 8-14-90 DRAINAGE DESIGN PROJECT NO 89-977

DRAINAGE AREA	AREA (Ac)	LONGEST FLOW PATH (Mi)	ELEVATION DIFFERENCE (Ft)	TIME OF CONCENTRATION (Min)	RAINFALL (2) INTENSITY (in/Hr)	RUNOFF COEFFICIENT	MAXIMUM DISCHARGE (cfs)
A9	1.91	0.1	100	2	7.92 (1)	0.4	6.1
CUTSLOPE	0.15						
TOTAL	2.06						
A10	2.60	0.14	40	4	7.92 (1)	0.4	8.2
CUTSLOPE	0.22						
TOTAL	2.82						
A11	0.29						
A12	4.02	0.27	120	6	7.00	0.4	11.3
(STOCKPILE	ACCESS ROAD	AND SIDE SLOPES)					
TOTAL	35.48						
A2	1.08	0.05	38	1	7.92 (1)	0.4	3.4
CUTSLOPE	0.58	0.09	38		7.92	0.4	
ROAD	0.91						
TOTAL	2.37						
A13	1.96				7.92 (1)	0.9	13.9
CUTSLOPE	0.66				7.92 (1)	0.9	
TOTAL	2.62						
A14	7.7	0.20	80	5	7.92	0.4	24.3
A15	10.8	0.22	98	5	7.92	0.4	34.2
A16	1.81	0.09	70	2	7.92 (1)	0.4	5.73

(see page 18)

Notes :
1. Assumed minimum time of concentration. (5 minutes)
2. See Figure 1.

DRAINAGE DESIGN CALCULATION

BY Upi DATE 8-8-90 SUBJECT LANDFILL B-18 SHEET NO 12 OF 59

CHKD N.A. DATE 8-14-90 DRAINAGE DESIGN PROJECT NO 89-977

DRAINAGE AREA	AREA (Ac)	LONGEST FLOW PATH (MI)	ELEVATION DIFFERENCE (Ft)	TIME OF CONCENTRATION (Min)	RAINFALL (2) INTENSITY (in/Hr)	RUNOFF COEFFICIENT	MAXIMUM DISCHARGE (cfs)
A17	4.08	0.14	66	3	7.92 (1)	0.4	12.9
A18	3.14				7.92 (1)	0.4	10
A19	1.29				7.92 (1)	0.4	4.1
A20	0.42				7.92 (1)	0.4	1.3
A21	0.54				7.92 (1)	0.4	1.71

Notes :

1. Assumed minimum time of concentration. (5 minutes)
2. See Figure 2

APPENDIX J.2

PHASES I AND II RUN-ON CONTROL

ENVIRONMENTAL SOLUTIONS, INC.

By TJPL Date 8-6-90 Subject LANDFILL B-18 Sheet No. 13 of 59

Chkd. By N.A. Date 8-14-90 DRAINAGE DESIGN Proj. No. 02-977

RUN-ON CONTROL PERIMETER DITCH (PHASE 1) FIGURE 3

DRAINAGE AREAS A1 & A3 DESIGN POINT A

TOTAL AREA: 3.30 ACRES (see page 10)

FLOW LENGTH: 890 ft

USE TYPE 2 V-DITCH AND 0.6 % SLOPE D = 1.0'

$V = 68.52 S^{1/2} = 68.52 \times \sqrt{0.006} = 5.3$ fps

TIME OF CONCENTRATION $t_c = 2 + \frac{890}{5.3} = 4.8$ min SAY 5

RAINFALL INTENSITY = 7.92 in/HR (see Exhibit 4)

$Q = 0.4 \times 3.3 \times 7.92 = 10.4$ cfs

Q DESIGN $= 171.3 S^{1/2} = 171.3 \times \sqrt{0.006} = 13.3 > 10.4$ cfs O.K.
(see page 5)

DRAINAGE AREAS A1, A3 & A6 DESIGN POINT B

TOTAL AREA: 13.61 ACRES

FLOW LENGTH: 2270 ft

USE TYPE 5 V-DITCH AND 0.6 % SLOPE D = 2.0 ft

$V = 96.9 S^{1/2} = 96.9 \times \sqrt{0.006} = 7.5$ fps

$t_c = 4.8 + \frac{1380}{7.5} = 7.8$ min

ENVIRONMENTAL SOLUTIONS, INC.

By JPL Date 8-6-90 Subject LANDFILL B-18 Sheet No. 14 of 59

Chkd. By N.A. Date 8-14-90 DRAINAGE DESIGN Proj. No. 89-077

RAINFALL INTENSITY: 6.2 in/HR

Q = 0.4 x 13.61 x 6.2 = 33.8 cfs.

$Q_{DESIGN} = 484.5\, S^{1/2} = 484.5 \times \sqrt{0.006} = 37.5$ cfs > 33.8 cfs O.K.

DRAINAGE AREAS A1, A3 & A7 DESIGN POINT C

TOTAL AREA: 17.62 ACRES

FLOW LENGTH: 2750 ft

USE TYPE 6 V-DITCH AND 0.6 % SLOPE D: 2.5

$V = 105.3\, S^{1/2} = 105.3 \times \sqrt{0.006} = 8.15$

$t_c = 7.8 + \frac{480}{8.15} = 8.8$ min

RAINFALL INTENSITY: 5.8 in/HR

Q = 0.4 x 17.62 x 5.8 = 40.9 cfs

$Q_{DESIGN} = 658.0 \times \sqrt{0.006} = 50.9$ cfs > 40.9 O.K.

ENVIRONMENTAL SOLUTIONS, INC.

By DPL Date 8-6-90 Subject LANDFILL B-18 Sheet No. 15 of 57

Chkd. By N.A. Date 8-14-90 DRAINAGE DESIGN Proj. No. 89-977

DRAINAGE AREAS A1, A3 & A8 DESIGN POINT D

TOTAL AREA: 26.53 AC.

FLOW LENGTH: 2930 ft.

USE TYPE 7 V-DITCH AND 0.6 % SLOPE D=3

$V = 111.2\, S^{1/2} = 111.2 \times \sqrt{0.0075} = 9.6$ fps

$t_c = 8.8 + \frac{180}{9.6} = 9.1$ min

RAINFALL INTENSITY = 5.8 IN/HR

$Q = 0.4 \times 26.53 \times 5.8 = 61.5$ cfs

$Q_{DESIGN} = 834.2 \times \sqrt{0.006} = 64.6$ cfs OK.

DRAINAGE AREAS A1, A3 & A11 DESIGN POINT E

TOTAL AREA = 31.64 AC

FLOW LENGTH: 2945 ft

$t_c = 9.1$ min

RAINFALL INTENSITY = 5.8 IN/HR

$Q = 31.64 \times 0.4 \times 5.8 = 73.4$ cfs

TYPE 7 V-DITCH AND 0.8 % SLOPE

$Q = 834.2 \times \sqrt{0.008} = 74.6$ cfs > 73.4 cfs O.K.

ENVIRONMENTAL SOLUTIONS, INC.

By DPW Date 8-7-90 Subject LANDFILL B-18 Sheet No. 16 of 59
Chkd. By N.A. Date 8-14-90 DRAINAGE DESIGN Proj. No. 89977

DRAINAGE AREAS A1, A3~A9, A11, A12, A14, A15, A16 DESIGN POINT F

AREAS A_1, $A_3 \sim A_{11}$

$T_c = 9.1$ min , $i = 5.8$ in/HR , $Q = 66.8$ c/s

AREAS A_{12} & A_{14}

A_{12} = $T_c = 6$ $i = 7$ $Q = 11.3$ (see p. 11)
A_{14} = $T_c = 5$ $i = 7.92$ $Q = 24.3$ (see p. 11)

$$Q_{12-14} = 24.3 + \frac{7.92}{7} \times \frac{5}{6} \times 11.3 = 34.9 \text{ c/s}$$

Q FOR A_1, $A_3 \sim A_{11}$, A_{12}, A_{14}

$$Q = 73.4 + \frac{5.8}{7.92} \times 34.9 = 98.9 \text{ c/s}$$

Q FOR A_1, $A_3 \sim A_{11}$, A_{12}, A_{14}, AND A_{15}

$$Q = 98.9 + \frac{5.8}{7.92} \times 34.2 = 123.9 \text{ c/s.}$$

Q FOR A_1, $A_3 \sim A_{11}$, A_{12}, A_{14}, A_{15} AND A_{16}

$$Q = 123.9 + \frac{5.8}{7.92} \times 5.73 = 128.1 \text{ c/s}$$

USE TYPE 5 V-DITCH WITH 8.3 % SLOPE

$$Q = 484.5 \times \sqrt{0.083} = 139.6 \text{ c/s} > 128.1 \text{ c/s}$$

DRAINAGE DESIGN CALCULATION

for Brow ditches & drop Inlet
Phase 1

BY JJM DATE 8-6-90 SUBJECT LANDFILL B-18 SHEET NO 17 OF 59
CHKD N.A. DATE 8-14-90 DRAINAGE DESIGN PROJECT NO 39.977

	(see p. 10 (11))	(see page 5) BROW DITCH		(see figure 3) DROP INLET			
DRAINAGE AREA	MAXIMUM FLOW (cfs)	DESIGN (1) SLOPE (%)	DESIGN (2) FLOW (cfs)	PIPE DIAMETER (in)	HEAD (ft)	DESIGN FLOW (cfs)	TYPE
A1	5.9	10.0	18.1		NOT REQUIRED		
A2	3.4	10.9	18.9		NOT REQUIRED		
A3	3.0	11.5	19.4	12	1	3.8	D1
A4	13.8	7.7	15.8	24	1	15.1	D2
A5	8.4	8.9	17.0	18	1	8.5	D3
A6	4.5	2.4	8.8	12	1.5	4.6	D4
A7	10.8	3.6	10.8	NOT REQUIRED			
A8	27.8	11.2	19.1	30	1.5	29.4	D5
A9	6.1	14.3	21.6	NOT REQUIRED			
A10	8.2	5.6	13.5	18	1	8.5	D6
A13	6.2	6.6	14.7	NOT REQUIRED			

NOTES

1. ONLY CONSIDER DITCHES WITH THE Existing SHALLOWEST SLOPE IN THE DRAINAGE AREA.
2. UNLESS SPECIFIED, DITCHES ARE DESIGNED FOR MAXIMUM FLOW IN THE DRAINAGE AREA

ENVIRONMENTAL SOLUTIONS, INC.

By DRI Date 8-7-90 Subject LANDFILL B-18 DRAINAGE Sheet No. 18 of 59
Chkd. By N.A. Date 8-14-90 DESIGN Proj. No. 89.977

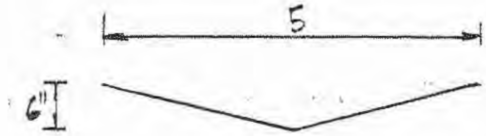
DRAINAGE AREA A2 DESIGN POINT G SWALE DESIGN

AREA : A2 = 1.08 (see p. 1)
SLOPE = 0.58
ROAD = 0.71
TOTAL = 2.37

RUN-OFF COEFFICIENT : $\frac{1.08 \times 0.4 + 0.58 \times 0.4 + 0.71 \times 0.9}{2.37} = 0.55$

RAINFALL INTENSITY = 7.92 IN/HR (5 MIN. minimum TIME OF CONCENTRATION)

$Q = 0.55 \times 2.37 \times 7.92 = 10.3$ cfs



TRY SWALE AS SHOWN :

$A_f = 0.5 \times 5 \times 0.5 = 1.25\ ft^2$

$P_w = 2 \times \sqrt{0.5^2 + 2.5^2} = 5.1\ ft$

$R = \frac{1.25}{5.1} = 0.25\ ft$

slope = 0.09 (Existing shallowest slope in the drainage area)

$Q = \frac{1.486}{0.013} \times 1.25 \times 0.25^{2/3} \times \sqrt{0.09}$

n = 0.013 EXHIBIT TROWEL FINISH CONCRETE OR EQUIVALENT

$= 16.7$ cfs > 10.3 cfs O.K.

ENVIRONMENTAL SOLUTIONS, INC.

By DPL Date 8-7-90 Subject LANDFILL B-18 Sheet No. 19 of 57

Chkd. By N.A. Date 8-14-90 DRAINAGE DESIGN Proj. No. 89-917

DRAINAGE AREAS A2 & A13 DESIGN POINT H (see p. 11)

TOTAL AREA = 4.28

TIME OF CONCENTRATION = < 5 min

RAINFALL INTENSITY = 7.92 IN/HR (USE MIN 5 MIN)

$$C = \frac{0.4 \times 1.08 + 0.58 \times 0.4 + 0.9 \times (0.66 + 1.96)}{4.28} = 0.71$$

$Q = 0.71 \times 7.92 \times 4.28 = 24.0$ cfs

USE TYPE 3 V-DITCH AND 1% SLOPE D=1.25 (see p. 5)

$Q_{DESIGN} = 243 \times \sqrt{0.01} = 24.3$ cfs > 24.0 O.K.

DRAINAGE AREAS A2, A13, AND SLOPE DESIGN POINT I

TOTAL AREA = 4.33 + 0.66 = 4.99

TIME OF CONCENTRATION = < 5 min Use 5 minutes

RAINFALL INTENSITY = 7.92 IN/HR. see Exhibit 4

$$C = \frac{0.4\,(1.08 + 0.58 + 0.66) + 0.9\,(0.66 + 1.96)}{4.99} = 0.66$$

$Q = 0.66 \times 4.99 \times 7.92 = 26.08$

USE TYPE 2 V-DITCH AND 8% SLOPE

$Q_{DESIGN} = 171.3 \times \sqrt{0.08} = 48.5$ cfs > 26.0 cfs O.K.

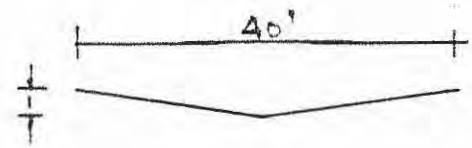
ENVIRONMENTAL SOLUTIONS, INC.

By TJM Date 8-7-90 Subject LANDFILL B-1B Sheet No. 20 of 59

Chkd. By N.A. Date 8-14-90 DRAINAGE DESIGN Proj. No. 89-977

Design pt. J SWALE DESIGN

USE SWALE SIZE AS SHOWN IN CLAY PROCESSING AREA



$A_T = 0.5 \times 40 \times 1 = 20 \text{ ft}^2$

$P_w = 2 \times \sqrt{1 + 20^2} = 40 \text{ ft}$

$R = \frac{20}{40} = 0.5 \text{ ft}$

slope = 0.015

n = 0.022

FOR EARTH CLEAN, AFTER WEATHERING

$Q = \frac{1.486}{0.022} \times 0.5^{2/3} \times 20 \times \sqrt{0.015}$

$= 104 \text{ cfs} > 26.4 \text{ cfs}$ OK.

CUTSLOPE AND ACCESS ROAD EAST OF B-1B STOCK PILE ACCESS ROAD

TOTAL AREA: 1.47 Ac. DESIGN POINT K

ASSUMED RAINFALL = 7.92 IN/HR 5 min TIME OF CONCENTRATION

$C = \frac{0.99 \times 0.4 + 0.48 \times 0.9}{1.47} = 0.56$

$Q = 0.56 \times 1.47 \times 7.92 = 6.55$

USE TYPE 2 V-DITCH & 6 % SLOPE (see p. 5)

$Q = 171.3 \times \sqrt{0.06}$

$= 41.9 \text{ cfs} > 6.55 \text{ cfs}$ O.K.

ENVIRONMENTAL SOLUTIONS, INC.

DRAINAGE AREA A18 DESIGN POINT L

$Q = 10$ cfs (see p. 12)

USE TYPE 2 V-DITCH & 0.5 % SLOPE (see p. 5)

$Q_{DESIGN} = 171.3 \times \sqrt{0.005} = 12.1$ cfs > 10 cfs O.K.

DRAINAGE AREAS A18 & A19 DESIGN POINT M

$Q = 10 + 4.1 = 14.1$ $T_{18} = T_{19} = 5$ min (see p. 12)

USE TYPE 2 V-DITCH & 1 % SLOPE (see p. 5)

$Q_{DESIGN} = 171.3 \times \sqrt{0.01} = 17.1$ cfs > 14.1 cfs OK

DRAINAGE AREAS A17 ~ A20 DESIGN POINT N

$Q = 12.9 + 10 + 4.1 + 1.3 = 28.3$ cfs $T_{17} = T_{18} = T_{19} = T_{20} = 5$ min (see p. 12)

USE TYPE 4 V-DITCH AND 0.8 % SLOPE (see p. 5)

$Q_{DESIGN} = 319.4 \times \sqrt{0.008} = 28.6$ cfs > 28.3 cfs OK.

DRAINAGE AREA A17 ~ A21 DESIGN POINT O

$Q = 12.9 + 10 + 4.1 + 1.3 + 1.71 = 30.01$ cfs (see p. 12)

USE TYPE 4 V-DITCH AND 1 % SLOPE see p. 5

$Q_{DESIGN} = 319.4 \times \sqrt{0.01} = 31.9$ cfs > 30.01 cfs OK

ENVIRONMENTAL SOLUTIONS, INC.

By DPI Date 8-8-90 Subject LANDFILL B-18 Sheet No. 22 of 59

Chkd. By M.A. Date 8-14-90 DRAINAGE DESIGN Proj. No. 80-977

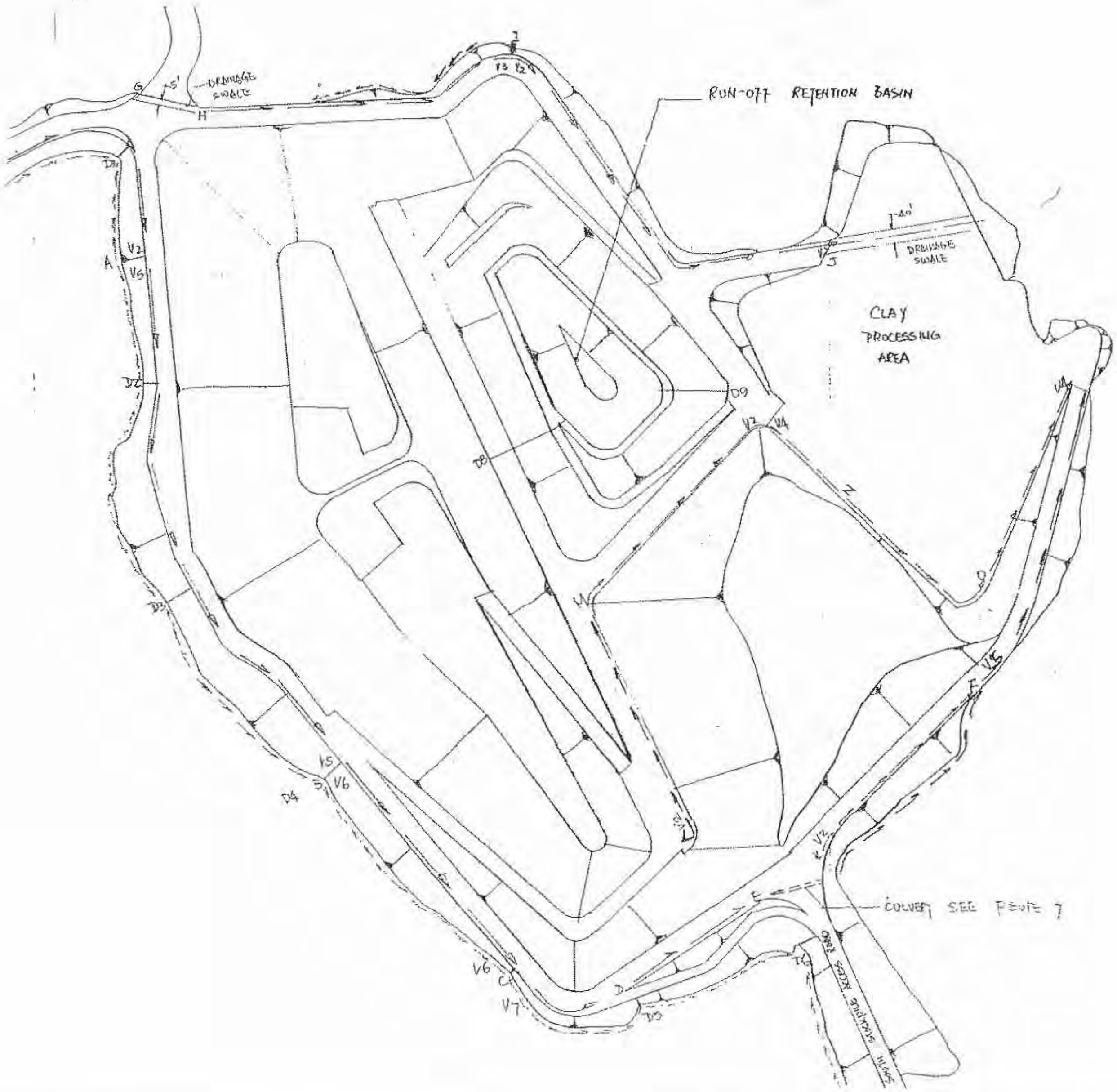
RUN-ON CONTROL PERIMETER DITCH PHASE II FIGURE 4

DRAINAGE AREAS A1, A3 & A16 DESIGN POINT A

TOTAL Q = 121.5 cfs (see p. 16)

USE TYPE 6 V-DITCH AND 3.6 % SLOPE (see p. 5)

$Q = 658 \times \sqrt{0.036} = 124.8$ cfs > 121.5 cfs OK.

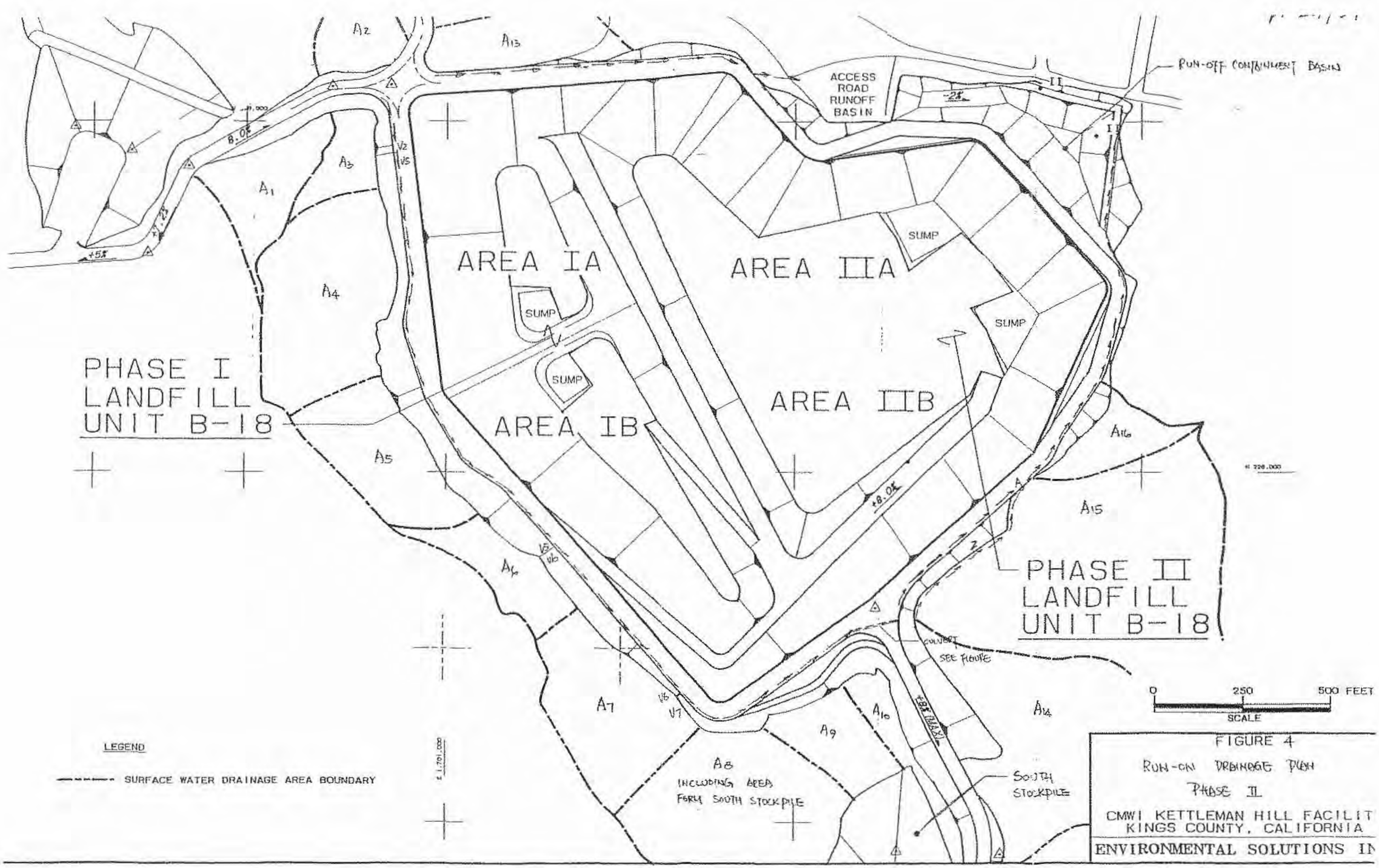


DROP INLET SCHEDULE (see p. 6)

TYPE	HEAD (ft)	PIPE DIAMETER (in)	PIPE LENGTH (ft)
D1	1	12	30
D2	1	24	30
D3	1	18	60
D4	1.5	12	45
D5	1.5	30	20
D6	1	18	40

FIGURE 3
RUN-ON DRAINAGE PLAN
PHASE 1

ENVIRONMENTAL SOLUTIONS, INC.


RUN-OFF CONTAINMENT BASIN
ACCESS ROAD RUNOFF BASIN
-2%
8.0%
+5%
AREA IA
AREA IIA
AREA IB
AREA IIB
SUMP
SUMP
SUMP
SUMP
PHASE I LANDFILL UNIT B-18
PHASE II LANDFILL UNIT B-18
+8.0%
A1
A2
A3
A4
A5
A6
A7
A8
INCLUDING AREA FORM SOUTH STOCKPILE
A9
A10
A13
A14
A15
A16
V2
V5
V6
V7
CULVERT SEE FIGURE
SOUTH STOCKPILE
0 250 500 FEET
SCALE
LEGEND
SURFACE WATER DRAINAGE AREA BOUNDARY
FIGURE 4
RUN-ON DRAINAGE PLAN
PHASE II
CMWI KETTLEMAN HILL FACILIT
KINGS COUNTY, CALIFORNIA
ENVIRONMENTAL SOLUTIONS IN

APPENDIX J.3

PHASES I AND II RUN-OFF CONTROL AND RUN-OFF CONTROL FOR PHASE IIIA

ENVIRONMENTAL SOLUTIONS, INC.

By JJPL Date 8-8-90 Subject LANDFILL E-18 Sheet No. 25 of 59
Chkd. By N.A. Date 8-14-90 DRAINAGE DESIGN Proj. No. 89-977

RUN-OFF CONTROL PERIMETER DITCHES PHASE I FIGURE 5

WESTERN & SOUTHERN BENCH ROAD DESIGN POINT A

LENGTH = 2600 ft WIDTH = 40 ft

AREA = 2.38 ACRES RUNOFF COEFFICIENT = 0.9

USE TYPE 3 V-DITCH & 0.6 % SLOPE D = 1.25 (see p. 5)

$V = 77.6 \times \sqrt{0.006}$
$= 6$ fps

$Tc = \frac{2600}{6} = 432.4$ sec $= 7.2$ min

$i = 6.7$ IN/HR see Exhibit 4

$\therefore Q = 0.9 \times 6.7 \times 2.38 = 14.4$ cfs

$Q_{DESIGN} = 243 \times \sqrt{0.006} = 18.8$ cfs > 14.4 cfs O.K.

DROP INLET AT END OF DITCH

USE 18" ⌀ CMP & 3 ft HEAD

$Q = 15.1$ cfs > 14.4 cfs O.K.

USE SAME TYPE OF DITCH TO DIVERT RUN-OFF TO DROP INLET AT TOP OF CLAY PIT. $Q = 243 \times \sqrt{0.005} = 17.2 > 14.4$

NORTHERN BENCH POND TO CLAY PIT DESIGN POINT B (see fig 5)

LENGTH : 1200 ft WIDTH = 40'

AREA = 1.1 ACRE RUNOFF COEFFICIENT = 0.9

t_c = 5 min i = 7.92 IN/HR

$Q = 0.9 \times 7.92 \times 1.1 = 7.8$ cfs

USE TYPE 2 V-DITCH AND 1% SLOPE D=1'

$Q_{DESIGN} = 171.3 \times \sqrt{0.01} = 17.1$ cfs $>$ 7.8 cfs

DROP INLET

USE 18" ϕ CMP & 12" HEAD

$Q = 8.5\sqrt{1} = 8.5$ cfs $>$ 7.8 cfs OK.

PHASE I / II DIVIDING BERM

SOUTHERN SECTION

LENGTH = 850 FT WIDTH = 40'

AREA = .78 Acre RUNOFF Coefficient = 0.9

Tc = 5 min i = 7.92 IN/HR

Q = 0.9 × 7.92 × .78 = 5.56 CFS

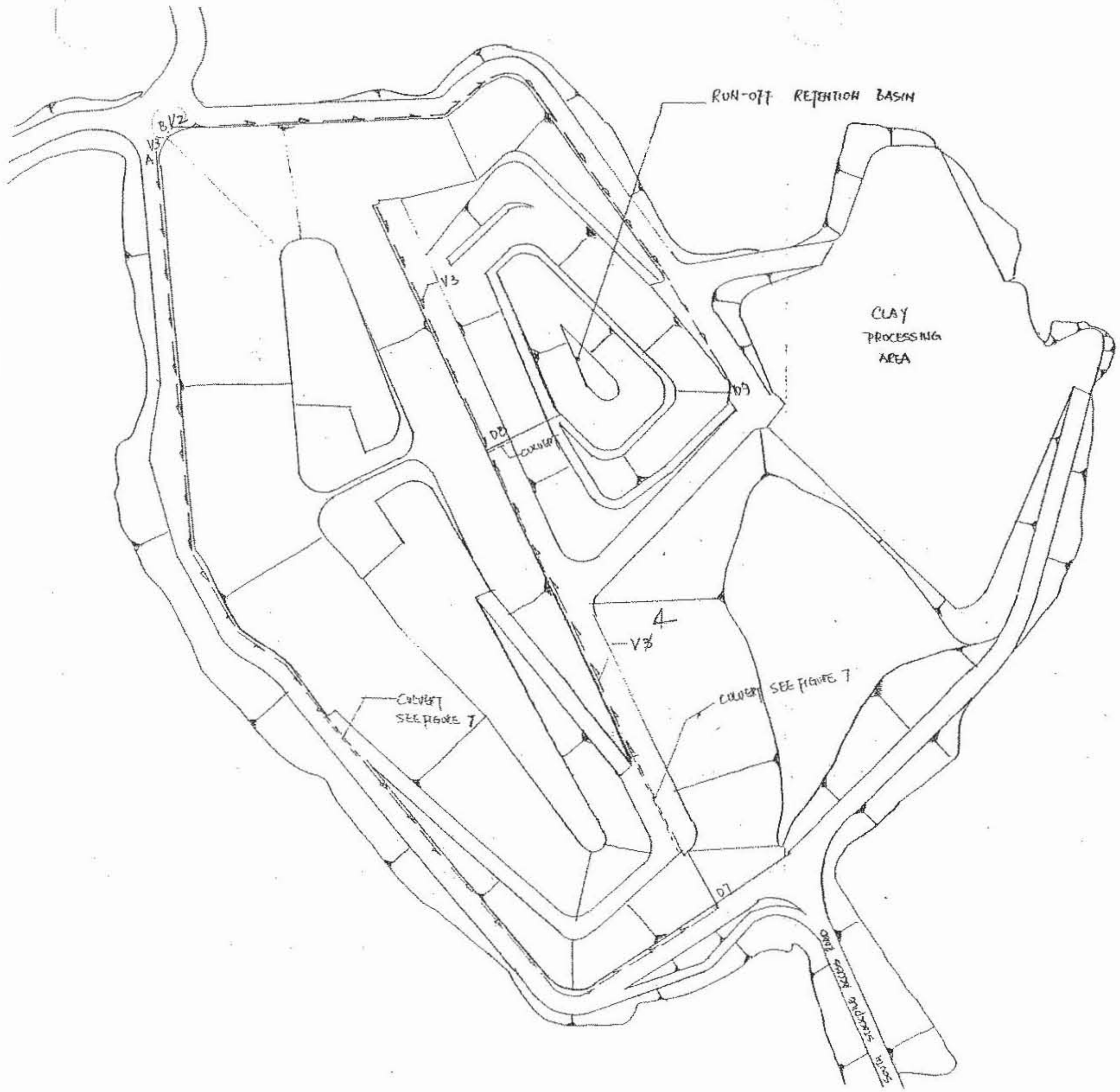
QT = 14.4 CFS + 5.56 CFS = 19.96 CFS

(SOUTHERN) SECTION

Use Type 4 $Q = 319.4\, S^{1/2} = 319.4\,(.01)^{1/2} = 31.94$ CFS

31.94 $>$ 19.96 OK

TYPE 4 NEEDED BECAUSE OF CULVERT



DROP INLET SCHEDULE (see page 25)

TYPE	HEAD (ft)	PIPE DIAMETER (in)	PIPE LENGTH (ft)
D7	3	18	140
D8	2.5	24	140
D9	1	18	110

FIGURE 5
PHASE I RUN-OFF CONTROL PLAN

ENVIRONMENTAL SOLUTIONS, INC.

RUN - OFF CONTROL PERIMETER DITCH PHASE II FIGURE 6

SOUTHERN BEACH ROAD DESIGN POINT A

LENGTH: 1600 ft WIDTH = 40'

AREA = 1.47 ACRES RUN-OFF COEFFICIENT = 0.9

USE TYPE 2 V-DITCH D= 1.25

ALONG 6.1 % SLOPE PORTION

$V = 68.5 \times \sqrt{0.061} = 17$ fps

$t_c = \frac{830}{17} = 0.8$ min

ALONG 3.6 % SLOPE PORTION

$V = 68.5 \times \sqrt{0.036} = 13$ fps

$t_c = \frac{520}{13} = 0.7$ min

$T_c = 7.2 + 0.8 + 0.7 = 8.7$

$i = 6.1$ IN/HR

$Q = 0.9 \times 6.1 \times (2.38 + 1.47) = 21.1$ cfs

$Q_{DESIGN} = 171.3 \times \sqrt{0.036} = 32.5$ cfs > 21.1 cfs O.K.

ENVIRONMENTAL SOLUTIONS, INC.

By JJPL Date 6-8-90 Subject LANDFILL B-18 Sheet No. 29 of 59
Chkd. By N.A Date 8-14-90 DRAINAGE DESIGN Proj. No. 89 977

NORTHERN BENCH ROAD DESIGN POINT B (see figure 6)

LENGTH = 1720 ft WIDTH = 40' AREA = 1.58 ACRES

$T_c = < 5$ min

$i = 7.92$ IN/HR

$Q = 0.9 \times 1.58 \times 7.92 = 11.3$ cfs

USE TYPE 2 V-DITCH AND 1% SLOPE min

$Q_{DESIGN} = 171.3 \times \sqrt{0.01} = 17.1$ cfs > 11.3 cfs O.K.

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29

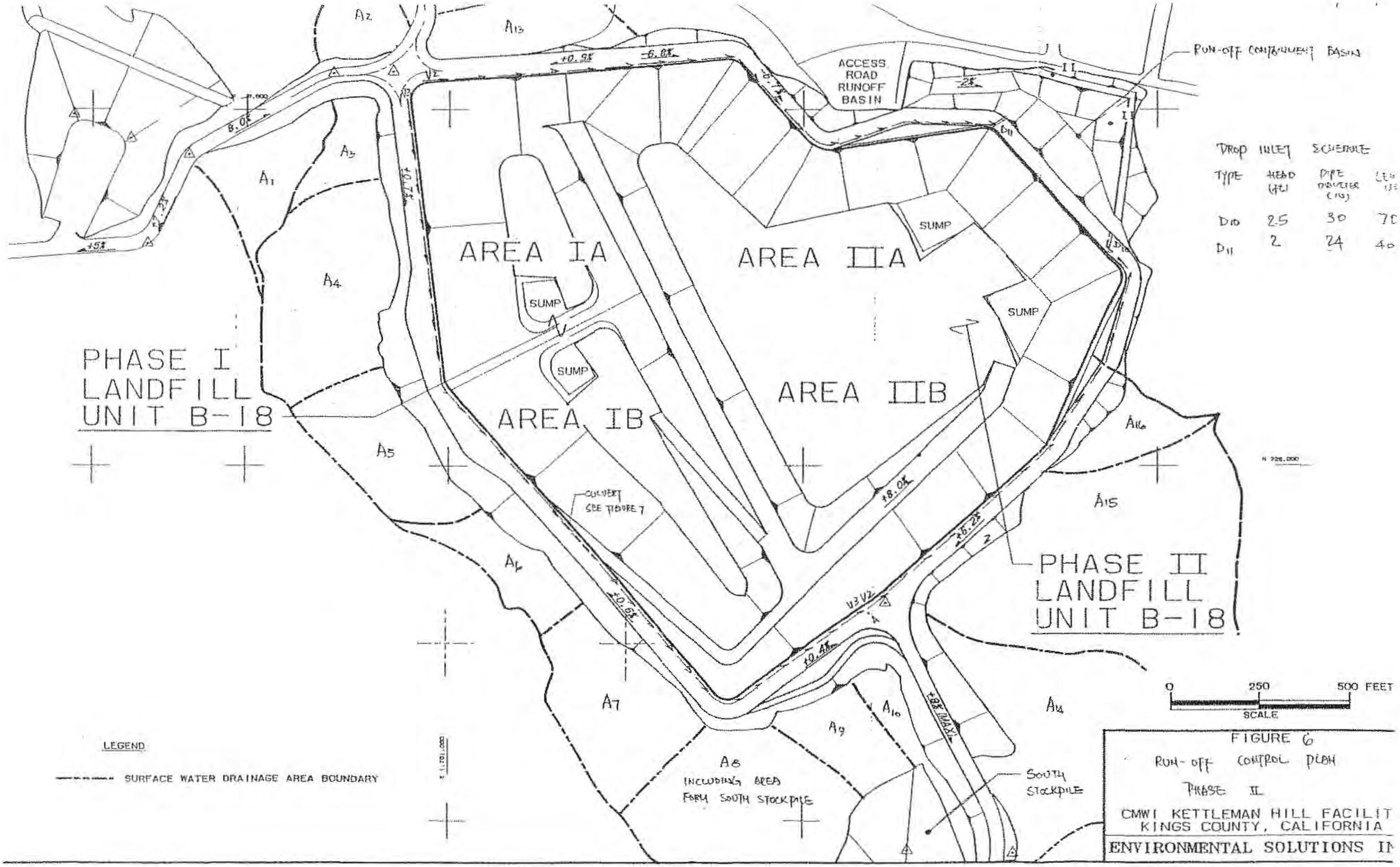


FIGURE 6
RUN-OFF CONTROL PLAN
PHASE II
CMWI KETTLEMAN HILL FACILIT
KINGS COUNTY, CALIFORNIA
ENVIRONMENTAL SOLUTIONS IN

CULVERT DESIGN FIGURE 7 & FIGURE 8

ALL CULVERTS WILL BE DESIGNED FOR INLET CONTROL.

CULVERT C1

LOCATION : PHASE 1 PRISM ACCESS ROAD & WEST PERIMETER BENCH ROAD

LENGTH = 150 ft

DIAMETER = 12" φ CONCRETE ENCASED

DRAINAGE AREA : NORTHERN HALF OF WEST PERIMETER ROAD

LENGTH: 1170 ft WIDTH = 40'

AREA = (1170 X 40)/43560 = 1.07 ACRES

SLOPE = 0.6 %

APPROACH VELOCITY : $V = 77.64 \times \sqrt{0.006} = 6$ fps. (TYPE 2 V-DITCH)

TIME OF CONCENTRATION TO CULVERT INLET = 1170/6 = 3.25 min

USE 5 min

i = 2.1 in/HR (25-YEAR)

$Q = CiA = 0.9 \times 2.1 \times 1.07 = 2$ cfs.

FROM INLET CONTROL NOMOGRAPH

HW/D = 0.9 ∴ HW = 0.9 X 12 = 10.8". < 12" OK

ENVIRONMENTAL SOLUTIONS, INC.

By [illegible] Date 8-9-90 Subject LANDFILL B-18 Sheet No. 32 of 59
Chkd. By [illegible] Date 8-14-90 DRAINAGE DESIGN Proj. No. 89-977

CULVERT 2

LOCATION : SOUTH STOCKPILE ROAD AND SOUTH PERIMETER ROAD

LENGTH = 120 ft

DIAMETER = 24" ϕ CMP

DRAINAGE AREAS : A1, A3 – A9, AND A11

AREA = 28.82 ACRES

SLOPE = 0.5 %

APPROACH VELOCITY : $V = 111.2 \sqrt{0.006} = 8.6$ fps

TIME OF CONCENTRATION TO CULVERT $t_c = 9.1$ min

$i = 1.7$ IN/HR (25-YEAR STORM)

$Q = 0.4 \times 1.7 \times 28.82 = 19.6$ cfs

FROM INLET CONTROL MONOGRAPH :

$HW/D = 1.39$ $\therefore HW = 1.39 \times 24 = 33.36"$

USE 24" ϕ CMP INLET & 3 ft HEAD

$Q = 15.1 \sqrt{3} = 26.2$ cfs > 19.6 cfs O.K.

CULVERT 3

LOCATION : ENTRANCE OF THE PHASE I/II BERM ROAD AND PHASE I PRISM ACCESS ROAD

LENGTH : 220 ft

DIAMETER : 18" CMP

DRAINAGE AREA = WESTERN BENCH ROAD

AREA : 2.38 ACRES

SLOPE = 0.5 %

TIME OF CONCENTRATION TO INLET = 7.2 min

i = 1.8 in/HR (25-YEAR STORM)

$Q = 0.9 \times 1.8 \times 2.38 = 3.9$ cfs

FROM INLET CONTROL NOMOGRAPH

$Hw/D = 0.74 \quad \therefore \; Hw = 0.74 \times 18 = 13.32'' < 18''$ OK

ENVIRONMENTAL SOLUTIONS, INC.

By DPL Date 8-10-90 Subject LANDFILL B-18 Sheet No. 34 of 57
Chkd. By JM Date 8-14-90 DRAINAGE DESIGN Proj. No. 89-977

CULVERT 4

LOCATION : PHASE I/II BERM CREST AT TOP OF CLAY PIT

LENGTH : 60 ft

DIAMETER = 30" ϕ CMP

DRAINAGE AREA = WESTERN BENCH ROAD AND TOP OF PHASE I/II BERM

AREA : 3.75 ACRES

SLOPE : 0.5 %

TIME OF CONCENTRATION TO INLET = 9.5 min

i = 5.8 IN/HR PMP STORM

$$C = \frac{1.37 \times 0.4 + 2.38 \times 0.9}{3.75} = 0.72$$

$Q = 0.72 \times 5.8 \times 3.75 = 15.6$ cfs

FROM INLET CONTROL MONOGRAPH

$HW/D = 0.79$ $\therefore$ $HW = 0.79 \times 30 = 23.7$

USE 30 ϕ CMP DROP INLET & 2 ft HEAD

$Q = 24\sqrt{2} = 33.8$ cfs > 15.6 cfs O.K

ENVIRONMENTAL SOLUTIONS, INC.

By JTPL Date 8-10-90 Subject LANDFILL B-18 Sheet No. 35 of 59
Chkd. By AL Date 8-14-90 DRAINAGE DESIGN Proj. No. 89-977

CULVERT 5

LOCATION : EASTERN END OF SOUTH PERIMETER BENCH ROAD

LENGTH : 70 ft

DIAMETER : 30" ϕ CMP

DRAINAGE AREA : WESTERN & SOUTHERN BENCH ROAD

AREA : 3.85 ACRES

SLOPE = 0.5 %

TIME OF CONCENTRATION = 8.7 min

i = 6.2 IN/HR

$Q = 0.9 \times 6.2 \times 3.85 = 21.$ cfs

FROM INLET CONTROL NOMOGRAPH

$H_w/D = 0.95$ $\therefore H_w = 28.5$

USE 30" ϕ CMP DROP INLET & 2.5' HEAD

$Q = 24 \times \sqrt{2.5}$ 33.9 cfs > 21. cfs O.K.

ENVIRONMENTAL SOLUTIONS, INC.

By TJpi Date 8-10-90 Subject LANDFILL B-1B Sheet No. 36 of 59
Chkd. By Tit Date 8-14-90 DRAINAGE DESIGN Proj. No. 89-977

CULVERT 6

LOCATION = EASTERN END OF NORTH BENCH ROAD

LENGTH = 40'

DIAMETER = 24" CMP

DRAINAGE AREA: NORTHERN BENCH ROAD

AREA: 1.58 ACRES

TIME OF CONCENTRATION TO INLET - <5 min

i = 7.92 IN/HR

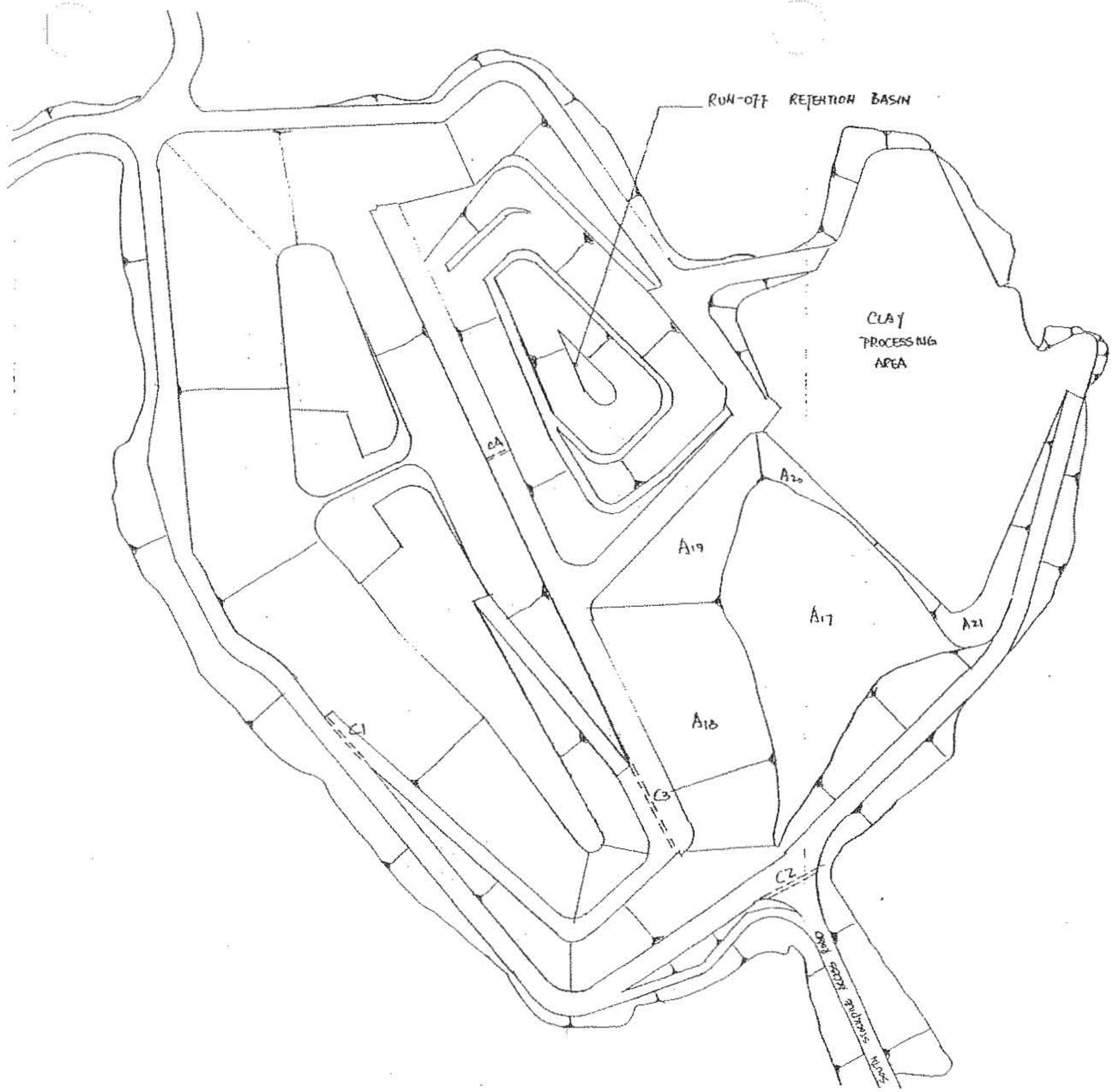
$Q = 0.9 \times 7.92 \times 1.58 = 11.2$ cfs

FROM INLET CONTROL NOMOGRAPH

$H_w/D = 0.9$ $\therefore H_w = 0.9 \times 24 = 21.6$

USE 24" CMP DROP INLET & 24" HEAD

$Q = 15.1 \times \sqrt{2} = 21.3$ cfs > 11.2 cfs OK.



CULVERT SCHEDUEL

TYPE	CULVERT DIAMETER (IN)	LENGTH (FT)
C1	12	150
C2	24	120
C3	18	220
C4	30	60
C5 (1)	24	70
C6 (1)	24	40

NOTE

(1) SAME AS DROP INLET D10 AND D11
SEE FIGURE 6 FOR LOCATIONS.

FIGURE 7

CULVERT LOCATIONS PLAN

LANDFILL 6-8

ENVIRONMENTAL SOLUTIONS, INC.

EXHIBIT 1

HYDROLOGY AND DRAINAGE DESIGN PROCEDURE, COUNTY OF SOLANO, CALIFORNIA

TABLE 6

RUNOFF COEFFICIENT FOR 10-YEAR* RETURN PERIOD

P. 51/59

I. LAND USE

Nonagricultural	Coefficient C
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban lots > 1/2 acre)	0.25 to 0.40
Apartment dwelling	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

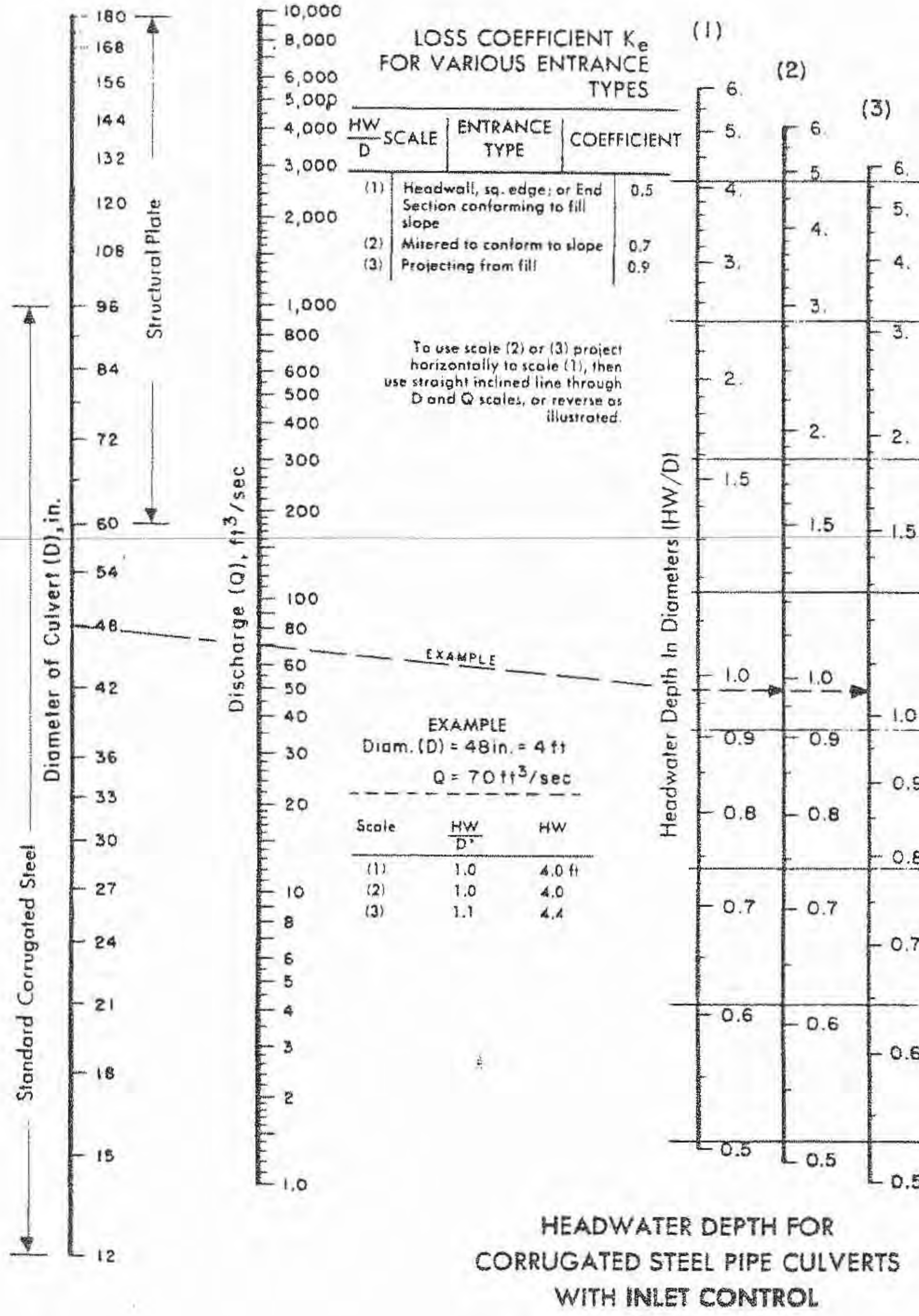
Agricultural/Open Space

Hydrologic Soil Group (See Table 2 and Fig. 2)	Cultivated	Pasture	Oak Timber and Brush
A	0.20	0.15	0.10
B	0.40	0.35	0.30
C	0.45	0.40	0.35
D	0.50	0.45	0.40

II. SURFACE TYPE

Character of Surface	Coefficient C
Pavement	
Asphaltic and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent or less	0.05 to 0.10
Average, 2 to 7 percent	0.10 to 0.15
Steep, 7 percent or more	0.15 to 0.20
Lawns, heavy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

*For other return periods, determine runoff coefficient from Figure 10.



FHWA HEC 5

Figure 4-28 *Inlet control* nomograph for corrugated steel *pipe* culverts. The manufacturers recommended keeping *HW/D* to a maximum of 1.5 and preferably to no more than 1.0.

EXHIBIT 3

MAXIMUM ANNUAL PRECIPITATION (UNITS=INCHES)
(TO CONVERT TO MILLIMETERS MULTIPLY BY 25.4)

STATION NO. BSN ORDER SUB	STATION NAME	ELEV	SEC	TWP	RNG	LOT	PMN	LATITUDE	LONGITUDE	COUNTY
C00 1867 0	COALINGA 1SE	663	04	21S	15E	J	M	36.128	120.344	FRESNO

M=MINUTES,H=HOURS,D=DAYS,C YR=CALENDAR YEAR,W YR=WATER YEAR,F YR=FISCAL YEAR
***** = NO DATA AVAILABLE

YEAR	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H	C-YR
	DURATION										
1940	0.09	0.14	0.21	0.28	0.43	0.64	0.84	1.24	1.75	1.77	10.76
1941	0.14	0.19	0.22	0.30	0.43	0.56	0.65	0.88	1.06	1.41	14.21
1942	0.10	0.14	0.19	0.22	0.25	0.45	0.65	0.93	0.95	0.98	5.10
1943	0.12	0.17	0.18	0.22	0.30	0.37	0.50	0.80	0.84	1.10	8.08
1944	0.07	0.10	0.13	0.16	0.21	0.37	0.49	0.69	0.76	1.39	5.49
1945	0.10	0.12	0.17	0.20	0.25	0.29	0.40	0.53	0.65	0.77	6.90
1946	0.06	0.10	0.12	0.18	0.37	0.48	0.61	0.80	0.87	0.90	5.73
1947	*****	*****	*****	*****	0.17	0.22	0.22	0.23	0.23	0.32	1.98
1948	0.05	0.08	0.12	0.17	0.23	0.46	0.50	0.56	0.64	0.86	5.48
1949	0.07	0.11	0.12	0.14	0.25	0.41	0.50	0.72	0.76	1.19	5.23
1950	0.13	0.15	0.17	0.17	0.22	0.37	0.47	0.65	0.81	0.85	5.86
1951	0.09	0.13	0.16	0.22	0.22	0.35	0.45	0.49	0.60	0.75	5.59
1952	0.20	0.28	0.47	0.49	0.50	0.57	0.57	0.83	0.90	0.90	10.92
1953	*****	*****	*****	*****	0.40	0.70	0.90	1.14	1.57	1.57	2.00
1954	0.03	0.05	0.08	0.15	0.24	0.44	0.60	1.15	1.24	1.30	5.60
1955	0.16	0.26	0.27	0.52	0.92	1.06	1.07	1.07	1.08	1.49	11.85
1956	0.20	0.26	0.39	0.54	0.59	0.60	0.61	0.65	0.65	0.65	4.30
1957	0.16	0.23	0.29	0.52	0.67	0.70	0.77	0.77	0.82	0.99	9.73
1958	0.10	0.16	0.18	0.21	0.32	0.56	0.59	0.69	0.87	0.94	11.04
1959	0.02	0.04	0.07	0.13	0.24	0.33	0.42	0.58	0.80	1.02	3.86
1960	0.05	0.07	0.09	0.16	0.27	0.51	0.63	0.78	1.41	1.76	5.15
1961	0.15	0.24	0.28	0.37	0.51	0.60	0.60	1.00	1.33	1.50	5.12
1962	0.06	0.08	0.11	0.20	0.38	0.54	0.75	1.10	1.15	1.37	7.98
1963	0.17	0.19	0.22	0.32	0.48	0.73	0.85	0.96	1.05	1.05	8.51
1964	0.05	0.07	0.10	0.19	0.34	0.41	0.42	0.79	0.98	1.00	4.31
1965	0.21	0.23	0.25	0.36	0.45	0.58	0.84	1.15	1.17	1.63	9.96
1966	0.09	0.14	0.16	0.23	0.32	0.47	0.50	0.52	1.40	2.27	5.79
1967	0.09	0.14	0.17	0.26	0.35	0.36	0.37	0.53	0.69	0.74	6.17
1968	0.02	0.06	0.09	0.15	0.25	0.38	0.38	0.45	0.65	0.96	5.30
1969	0.16	0.18	0.27	0.36	0.50	0.70	0.90	1.53	2.27	2.66	13.75
1970	0.03	0.06	0.09	0.14	0.25	0.41	0.54	0.72	0.79	0.86	6.47
1971	0.04	0.07	0.10	0.14	0.20	0.28	0.44	0.75	0.81	0.85	4.27
1972	0.04	0.07	0.08	0.13	0.23	0.27	0.32	0.45	0.62	0.62	3.28
1973	*****	*****	*****	0.29	0.34	0.50	0.64	0.80	0.85	1.00	8.53
1974	*****	*****	*****	*****	0.16	0.31	0.41	0.62	0.85	1.55	6.86

PRECIPITATION DEPTH-DURATION-FREQUENCY TABLE

STATION NO. BSN ORDER SUB	STATION NAME	ELEV	SEC	TWP	RNG	LOT	PMN	LATITUDE	LONGITUDE	COUNTY
C00 1867 0	COALINGA 1SE	663	04	21S	15E	J	M	36.128	120.344	FRESNO

MAXIMUM PRECIPITATION (IN) FOR INDICATED DURATION D=DAYS H=HOURS M=MINUTES

RETURN PERIOD IN YEARS	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H	C-YR
2	0.09	0.13	0.17	0.23	0.37	0.45	0.54	0.74	0.89	1.08	6.46
5	0.13	0.18	0.23	0.32	0.44	0.62	0.74	1.01	1.23	1.49	8.88
10	0.15	0.21	0.27	0.38	0.52	0.73	0.88	1.20	1.45	1.76	10.41
20	0.17	0.24	0.31	0.44	0.60	0.84	1.01	1.38	1.66	2.02	11.82
25	0.18	0.25	0.32	0.45	0.62	0.87	1.05	1.43	1.73	2.10	12.26
40	0.19	0.27	0.35	0.49	0.67	0.94	1.13	1.55	1.87	2.27	13.15
50	0.20	0.28	0.36	0.51	0.69	0.97	1.17	1.60	1.93	2.34	13.57
100	0.22	0.31	0.40	0.56	0.77	1.07	1.29	1.76	2.13	2.58	14.83
200	0.24	0.33	0.44	0.61	0.84	1.17	1.41	1.92	2.32	2.82	16.05
1000	0.28	0.40	0.52	0.73	0.99	1.39	1.68	2.29	2.76	3.35	18.79
10000	0.35	0.49	0.63	0.89	1.21	1.70	2.05	2.79	3.37	4.09	22.53
PMP	0.66	0.92	1.20	1.68	2.30	3.22	3.89	5.30	6.41	7.77	46.28
MEAN	0.099	0.139	0.181	0.254	0.347	0.485	0.586	0.799	0.965	1.171	6.892
CLOCK HR. COR.	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
CALCULATED SKEW	0.480	0.500	1.349	1.194	1.697	1.158	0.561	0.477	1.402	1.172	0.776
REGIONAL SKEW	1.300	1.300	1.300	1.300	1.300	1.300	1.300	1.300	1.300	1.300	1.000
SKEW USED	1.300	1.300	1.300	1.300	1.300	1.300	1.300	1.300	1.300	1.300	1.000

SLOPE OF LOG INTENSITY / LOG TIME = -.504 ; INTERCEPT (TIME=1 HOUR) =0.346 ; COEFFICIENT OF DETERMINATION = 0.999
1HR INTERCEPT / MEAN YR =0.05025 ; AVERAGE CALC CV / USED CV = 1.22

	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H	C-YR
KURTOSIS	2.373	2.435	4.877	3.644	7.132	5.620	3.239	3.731	6.249	5.112	5.178
N	31	31	31	32	35	35	35	35	35	35	35
RECORD YEAR	1965	1952	1952	1956	1955	1955	1955	1969	1969	1969	1941
RECORD MAXIMUM	0.210	0.280	0.470	0.540	0.920	1.060	1.070	1.530	2.270	2.660	14.210
NORMILIZED MAX	1.984	2.029	2.966	2.330	3.673	3.407	2.556	2.749	3.418	3.139	2.448
CALC. COEF. VAR	0.568	0.500	0.540	0.484	0.450	0.348	0.323	0.333	0.396	0.405	0.434
REGN. COEF. VAR	0.376	0.376	0.376	0.376	0.376	0.376	0.376	0.376	0.376	0.376	0.381
USED COEF. VAR	0.376	0.376	0.376	0.376	0.376	0.376	0.376	0.376	0.376	0.376	0.381
MEAN/A	0.0143	0.0202	0.0262	0.0368	0.0503	0.0704	0.0851	0.1159	0.1400	0.1699	1.0000
RP10/A	0.0215	0.0303	0.0394	0.0554	0.0757	0.1058	0.1279	0.1743	0.2105	0.2554	1.5107
RP25/A	0.0257	0.0362	0.0470	0.0660	0.0902	0.1262	0.1525	0.2078	0.2510	0.3045	1.7783
RP50/A	0.0287	0.0404	0.0525	0.0737	0.1008	0.1410	0.1704	0.2321	0.2804	0.3401	1.9685
RP100/A	0.0316	0.0445	0.0579	0.0813	0.1111	0.1554	0.1878	0.2559	0.3090	0.3749	2.1516
RP1000/A	0.0410	0.0578	0.0751	0.1054	0.1441	0.2016	0.2436	0.3319	0.4009	0.4863	2.7264
RP10000/A	0.0501	0.0705	0.0917	0.1287	0.1760	0.2461	0.2975	0.4053	0.4895	0.5939	3.2696
PMP/A	0.0951	0.1340	0.1740	0.2445	0.3342	0.4674	0.5649	0.7697	0.9296	1.1278	6.7150

PEARSON TYPE III DISTRIBUTION USED
PROBABLE MAXIMUM PRECIPITATION ESTIMATE BASED ON 15 STANDARD DEVIATIONS
WHERE N IS SMALL (<25) RESULTS ARE NOT DEPENDABLE

ENVIRONMENTAL SOLUTIONS, INC.

By VDI Date 6-1-90 Subject LANDFILL B-18 DRAINAGE DESIGN Sheet No. 54 of 59

Chkd. By AA Date Proj. No. 89 997

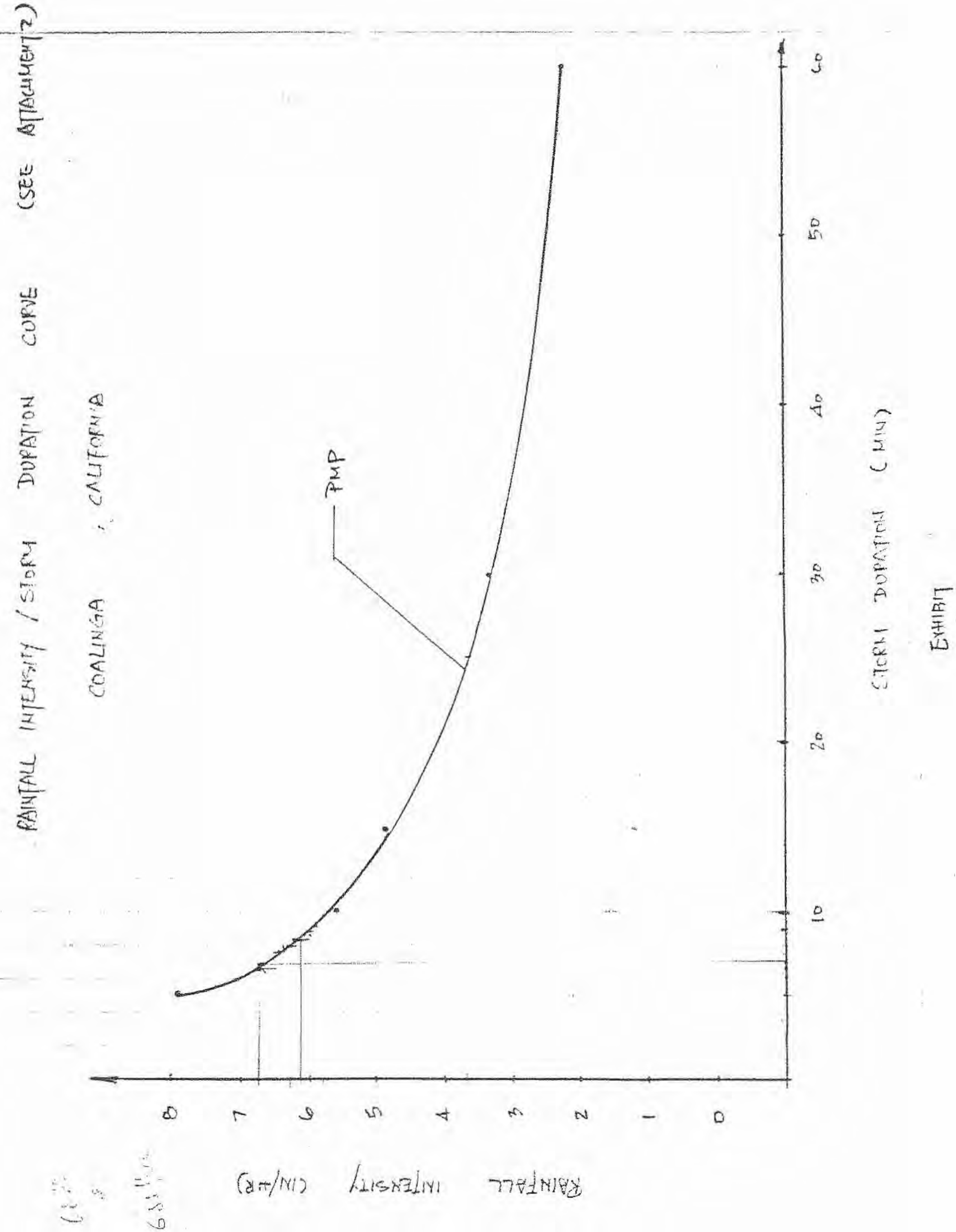


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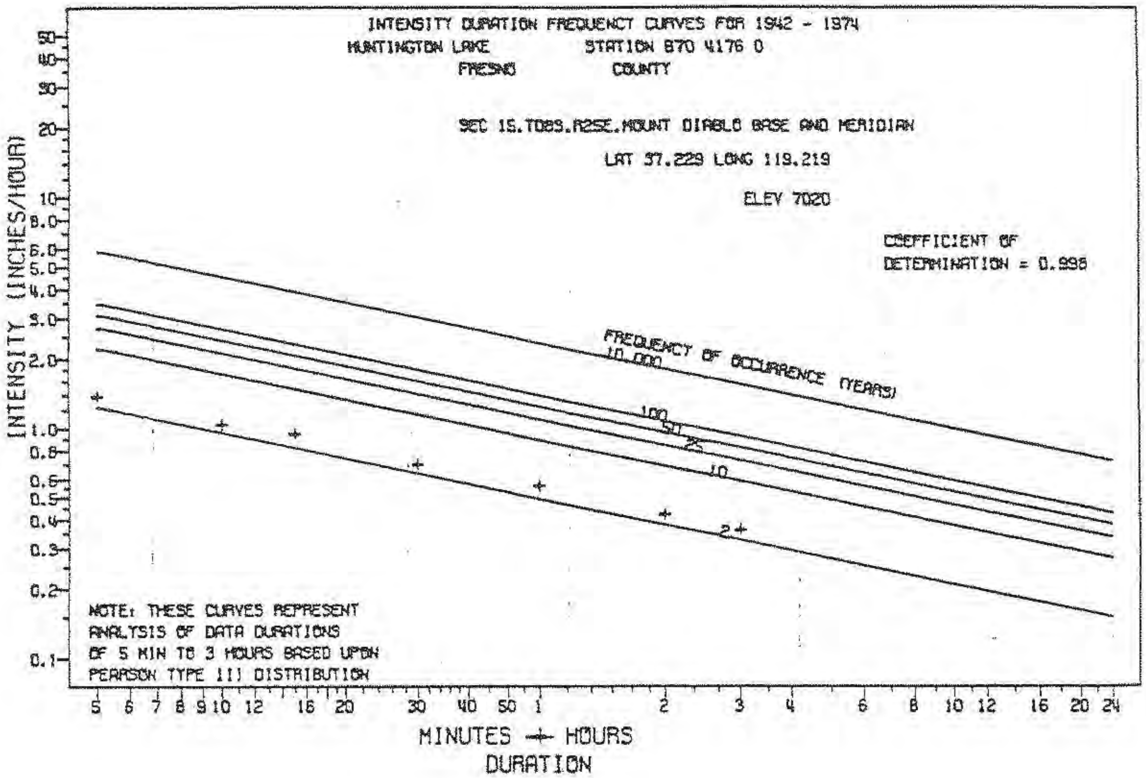
EXHIBIT 5



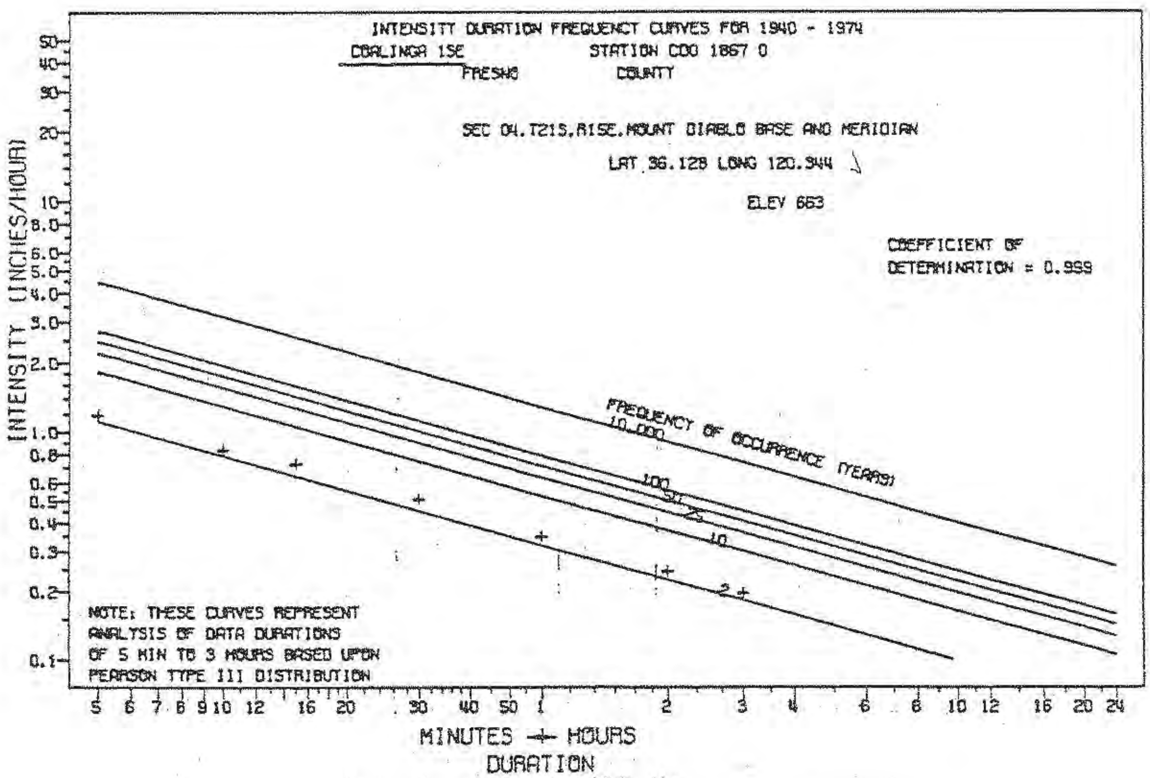
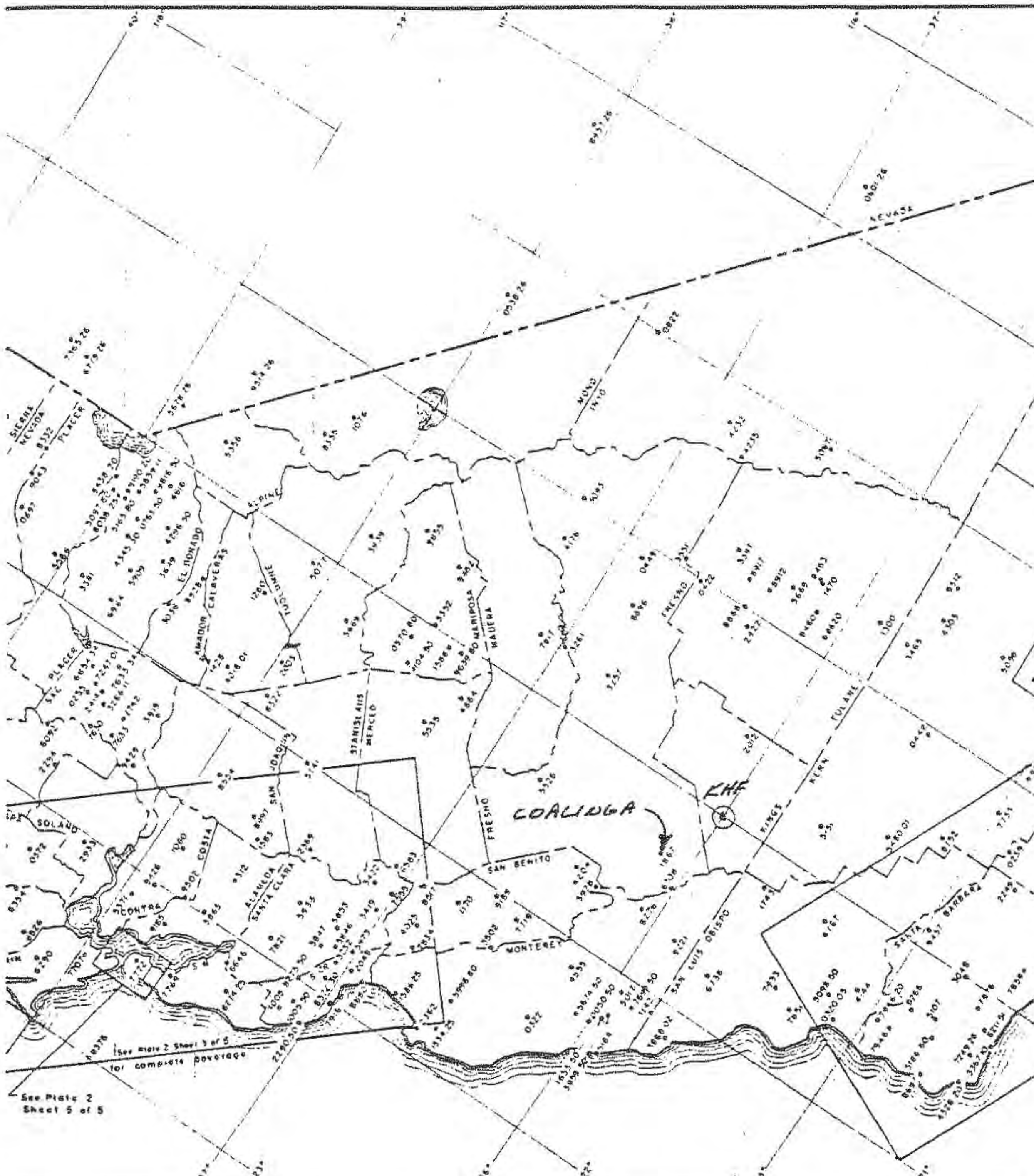


EXHIBIT 6

P. 56/59

VOL. III



13-B

KHF
⊙ LOCATION OF SITE

Table 2

MINIMUM RUNOFF COEFFICIENTS FOR BUILT-UP AREAS

RESIDENTIAL AREAS:	C = 0.55 to 0.70
HOTEL-APARTMENT AREAS:	C = 0.70 to 0.90
BUSINESS AREAS:	C = 0.80 to 0.90
INDUSTRIAL AREAS:	C = 0.80 to 0.90

The type of soil, the type of open space and ground cover and the slope of the ground shall be considered in arriving at reasonable and acceptable runoff coefficients.

Table 3

APPROXIMATE AVERAGE VELOCITIES OF RUNOFF FOR CALCULATING TIME OF CONCENTRATION

TYPE OF FLOW	VELOCITY IN FPS FOR SLOPES (in percent) INDICATED			
OVERLAND FLOW:	0–3%	4–7%	8–11%	12–15%
Woodlands	1.0	2.0	3.0	3.5
Pastures	1.5	3.0	4.0	4.5
Cultivated	2.0	4.0	5.0	6.0
Pavements	5.0	12.0	15.0	18.0
OPEN CHANNEL FLOW:				
Improved Channels	Determine Velocity by Manning's Formula			
Natural Channel* (not well defined)	1.0	3.0	5.0	8.0

**These values vary with the channel size and other conditions so that the ones given are the averages of a wide range. Wherever possible, more accurate determinations should be made for particular conditions by Manning's formula.*

PRECIPITATION DEPTH-DURATION-FREQUENCY TABLE

STATION NO. DSN ORDER SUB	STATION NAME	ELEV	SEC	TWP	RNG LOT	BWM	LATITUDE	LONGITUDE	COUNTY CODE
COO 4536 00	KETTLEMAN STATION	508	25	21S	17E L	M	36.075	120.085	16

MAXIMUM PRECIPITATION FOR INDICATED DURATION D-DAYS H-HOURS

RETURN PERIOD IN YEARS	1D	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	365D
2	.98	1.21	1.31	1.38	1.44	1.50	1.60	1.76	2.00	2.14	2.58	3.49	6.08
5	1.35	1.71	1.89	2.00	2.09	2.18	2.33	2.53	2.93	3.10	3.65	4.98	8.35
10	1.59	2.05	2.28	2.42	2.53	2.64	2.83	3.04	3.56	3.74	4.34	5.96	9.79
20	1.83	2.38	2.65	2.82	2.95	3.08	3.30	3.52	4.17	4.35	4.98	6.88	11.12
25	1.90	2.48	2.77	2.94	3.08	3.21	3.45	3.67	4.36	4.54	5.17	7.16	11.53
40	2.05	2.70	3.01	3.20	3.36	3.50	3.76	3.99	4.75	4.93	5.27	7.75	12.37
50	2.12	2.80	3.13	3.33	3.48	3.64	3.90	4.14	4.94	5.12	5.77	8.03	12.76
100	2.34	3.10	3.48	3.70	3.88	4.05	4.34	4.59	5.50	5.68	6.35	8.87	13.95
200	2.55	3.41	3.82	4.07	4.27	4.45	4.78	5.03	6.06	6.23	6.91	9.69	15.10
1000	3.03	4.09	4.60	4.91	5.15	5.37	5.78	6.03	7.33	7.49	8.18	11.54	17.67
10000	3.71	5.05	5.70	6.09	6.38	6.66	7.17	7.42	9.10	9.23	9.92	14.11	21.19
PMP	7.06	9.47	10.72	11.49	12.05	12.58	13.55	14.32	17.25	17.88	20.09	27.91	43.52
MEAN	1.060	1.329	1.454	1.530	1.598	1.666	1.779	1.934	2.226	2.358	2.792	3.807	6.481
CLOCK HR. COR.	1.140	1.070	1.040	1.020	1.010	1.010	1.000	1.000	1.000	1.000	1.000	1.000	1.000
CALCULATED SKEW	.663	.605	.815	1.008	1.037	1.133	1.237	1.084	1.499	1.444	1.302	1.936	1.578
REGIONAL SKEW	1.300	1.400	1.400	1.400	1.400	1.400	1.400	1.300	1.400	1.300	1.100	1.200	1.000
SKEW USED	1.300	1.400	1.400	1.400	1.400	1.400	1.400	1.300	1.400	1.300	1.100	1.200	1.000
KURTOSIS	3.212	3.150	2.880	3.426	3.574	3.701	4.396	4.095	6.012	5.746	4.328	7.584	5.922
N	24	24	24	24	24	24	24	24	24	24	24	24	23
RECORD YEAR	1956	1956	1962	1974	1974	1974	1969	1969	1969	1969	1969	1969	1969
RECORD MAXIMUM	1.680	2.170	2.600	3.120	3.360	3.360	3.860	4.190	5.650	5.660	6.390	10.370	14.710
NORMALIZED MAX	1.557	1.768	1.869	2.284	2.369	2.351	2.638	2.631	3.092	3.033	2.712	3.328	2.748
CALC. COEF. VAR	.376	.358	.422	.455	.465	.433	.444	.443	.498	.462	.475	.518	.462
REGN. COEF. VAR	.376	.408	.425	.434	.436	.437	.441	.427	.450	.439	.413	.422	.381
USED COEF. VAR	.376	.408	.425	.434	.436	.437	.441	.427	.450	.439	.413	.422	.381
MEAN/A	.1635	.2051	.2243	.2361	.2466	.2570	.2744	.2984	.3434	.3638	.4307	.5874	1.0000
RF10/A	.2458	.3170	.3517	.3731	.3903	.4071	.4362	.4690	.5500	.5777	.6693	.9197	1.5107
RF25/A	.2931	.3832	.4271	.4542	.4754	.4959	.5320	.5669	.6722	.7005	.7982	1.1049	1.7783
RF50/A	.3274	.4316	.4822	.5134	.5375	.5608	.6019	.6381	.7615	.7897	.8905	1.2384	1.9685
RF100/A	.3609	.4789	.5362	.5714	.5983	.6244	.6704	.7074	.8490	.8766	.9798	1.3681	2.1516
RF1000/A	.4682	.6315	.7100	.7583	.7944	.8292	.8911	.9297	1.1308	1.1552	1.2621	1.7809	2.7264
RF10000/A	.5717	.7798	.8789	.9398	.9849	1.0282	1.1055	1.1443	1.4046	1.4242	1.5309	2.1769	3.2696
PMP/A	1.0857	1.4605	1.6543	1.7733	1.8594	1.9416	2.0899	2.2093	2.6615	2.7594	3.0991	4.3056	6.7150

PEARSON TYPE III DISTRIBUTION USED
PROBABLE MAXIMUM PRECIPITATION ESTIMATE BASED ON 15 STANDARD DEVIATIONS
WHERE N IS SMALL RESULTS ARE NOT DEPENDABLE

EXHIBIT 8

B

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT n
(Boldface figures are values generally recommended in design)

Type of channel and description	Minimum	Normal	Maximum
A. CLOSED CONDUITS FLOWING PARTLY FULL			
A-1. Metal			
a. Brass, smooth	0.009	**0.010**	0.013
b. Steel			
1. Lockbar and welded	0.010	0.012	0.014
2. Riveted and spiral	0.013	0.016	0.017
c. Cast iron			
1. Coated	0.010	0.013	0.014
2. Uncoated	0.011	0.014	0.016
d. Wrought iron			
1. Black	0.012	0.014	0.015
2. Galvanized	0.013	0.016	0.017
e. Corrugated metal			
1. Subdrain	0.017	0.019	0.021
2. Storm drain	0.021	**0.024**	0.030
A-2. Nonmetal			
a. Lucite	0.008	0.009	0.010
b. Glass	0.009	**0.010**	0.013
c. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
d. Concrete			
1. Culvert, straight and free of debris	0.010	0.011	0.013
2. Culvert with bends, connections, and some debris	0.011	**0.013**	0.014
3. Finished	0.011	0.012	0.014
4. Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
5. Unfinished, steel form	0.012	0.013	0.014
6. Unfinished, smooth wood form	0.012	**0.014**	0.016
7. Unfinished, rough wood form	0.015	0.017	0.020
e. Wood			
1. Stave	0.010	0.012	0.014
2. Laminated, treated	0.015	0.017	0.020
f. Clay			
1. Common drainage tile	0.011	**0.013**	0.017
2. Vitrified sewer	0.011	0.014	0.017
3. Vitrified sewer with manholes, inlet, etc.	0.013	0.015	0.017
4. Vitrified subdrain with open joint	0.014	0.016	0.018
g. Brickwork			
1. Glazed	0.011	0.013	0.015
2. Lined with cement mortar	0.012	0.015	0.017
h. Sanitary sewers coated with sewage slimes, with bends and connections	0.012	0.013	0.016
i. Paved invert, sewer, smooth bottom	0.016	0.019	0.020
j. Rubble masonry, cemented	0.018	0.025	0.030

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT n (*continued*)

Type of channel and description	Minimum	Normal	Maximum
B. LINED OR BUILT-UP CHANNELS			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	**0.012**	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	**0.013**	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	**0.013**	0.015
2. In cement mortar	0.012	**0.015**	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017
i. Asphalt			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
j. Vegetal lining	0.030	.....	0.500

EXH IT 9

P. 54/59

METHOD OF CALCULATION

Project Number: 083-91887	MADE BY: PM CHECK BY: RH	Date: 8-1-2011
Project Name: Kettleman Hills B-18 Phase IIIA	REVIEW BY: RH	SHEET 1 OF 3

RE: TEMPORARY PHASE IIIA STORMWATER BERM AND CAPACITY OF NE B-18 CONTAINMENT BASIN DURING PHASE III CONSTRUCTION

1.0 OBJECTIVES

- Design the height of the proposed Phase IIIA temporary stormwater containment berm. This berm is required to be designed to function without failure to capture and retain the volume from the 24-hour, Probable Maximum Precipitation (PMP) storm event on the north side of the berm (i.e., this berm will contain stormwater run-off from the lower portion of the interim Phase IIIA waste slope and the surrounding areas).
- Evaluate the capacity versus demand of the existing NE B-18 Containment Basin during construction of Phase III. During the Phase III construction (i.e., before the South Containment Basin comes online), an outlet control system will be required during the 24-hour, PMP storm to prevent overtopping of the existing NE B-18 Containment Basin.

2.0 METHODOLOGY

The SCS Runoff Curve Number method was used to calculate the Phase IIIA interim drainage berm retention volume demand for the 24-hour, PMP storm event. This was compared to the proposed storage capacity of the Phase IIIA temporary basin to evaluate if the proposed berm is tall enough.

HEC-HMS modeling software (USACE) was used to evaluate the required outlet control peak flow rate to prevent overtopping of the existing NE B-18 Containment Basin during the Phase III construction.

3.0 ASSUMPTIONS

- The 24-hour PMP rainfall event equals 10.3 inches
- SCS Type 1 rainfall synthetic distribution was used
- SCS Curve Number (CN) of 81 was used for all basins

4.0 INTERIM PHASE IIIA DRAINAGE BERM CALCULATIONS

4.1 Storage Capacity

The interim Phase IIIA drainage berm will be constructed 10 feet high and have a maximum storage capacity of 52,100 cubic feet on its north side. This storage capacity assumes a freeboard of 1 foot (i.e., the 52,100 cubic feet of storage capacity is for a 9-foot depth of water contained by the berm on its north side).

It should be noted that stormwater run-on contained by the interim Phase IIIA drainage berm on its south side will be clean stormwater and will have a maximum depth of approximately 2 feet during the 24-hour PMP. This maximum depth corresponds to the elevation difference between the toe of the south side of the berm and the local high point on the Phase IIIB lined "floor bench" that lies to the south. It follows that the south side of the interim Phase IIIA drainage berm has an unlimited stormwater run-on storage capacity since the top of this berm is much higher than the local high point to the south.

4.2 PMP Volume Calculation

The SCS Curve Number method was used to evaluate the runoff volume from the 24-hour, PMP storm event for the north side of the Phase IIIA interim drainage berm. This interim drainage berm will capture 0.85 acres of storm water (see Figure 1). A Curve Number, CN, of 81 was used.

$$Q = (P - 0.2S)^2 / (P + 0.8S)$$

Where: Q = runoff, in

P = rainfall (10.3 in)

S = potential maximum retention after runoff, in

Where: S = 1000/CN - 10

The estimated runoff was calculated to be 7.94 inches over the 0.85-acre drainage basin. The minimum required volume for the Phase IIIA interim drainage basin to contain the 24-hour PMP is therefore 24,500 cubic feet.

5.0 EXISTING NE B-18 CONTAINMENT BASIN CALCULATIONS

The existing NE B-18 Containment Basin is approximately 25 feet deep with a capacity of approximately 30 acre-feet. A HEC-HMS analysis was performed using the existing conditions of the basin. If the 24-hour, PMP storm event was to occur during the Phase III construction (i.e., before the South Containment Basin is online), it is predicted that runoff to the NE B-18 Containment Basin will exceed capacity by approximately 14 acre-feet. Therefore, an outlet control device will be used to prevent overtopping of this basin during the 24-hour, PMP storm event. Excess water will be conveyed through the outlet and into a gravity pipe that will convey the overflow to the site's existing East Retention Basin located approximately 2,000 feet to the north.

5.1 Outlet Control System

HEC-HMS modeling software was used to calculate the peak flow rate required for an outlet control device set approximately 3 feet below the top of the existing NE B-18 Containment Basin embankment. Using a 21-inch orifice outlet device, it was calculated that a peak flow of 17 cfs was sufficient to prevent the basin from overtopping during the 24-hour PMP.

A preliminary minimum pipe size was calculated to convey the required 17 cfs from the NE B-18 Containment Basin. Pipe calculations were performed using the Federal Highway Administration software program Hydraulic Toolbox 2.1. A pipeline with a minimum slope of 1% and a Manning's coefficient of 0.010 was used to calculate the minimum size required to convey the required flow rate of 17 cfs. A minimum 21-inch inside diameter pipe is needed to convey the flow rate of 17 cfs.

6.0 CONCLUSIONS

The stormwater run-off volume from the 24-hour, PMP storm event captured on the north side of the proposed interim Phase IIIA drainage berm was calculated to be 24,500 cubic feet. The proposed interim Phase IIIA berm will be constructed to a height of 10 feet and will have a capacity of 52,100 cubic feet (assuming 1 foot of freeboard). Therefore, the proposed interim Phase IIIA drainage berm will have sufficient capacity to contain the flows from the 24-hour, PMP event with a freeboard greater than 1 foot.

The existing NE B-18 Containment Basin has a capacity of approximately 30 acre-feet. If the 24-hour, PMP storm event occurs during the construction of Phase III (i.e., before the South Containment Basin comes online), it is predicted that runoff to the existing NE B-18 Containment Basin will exceed its capacity by approximately 14 acre-feet. A 21-inch orifice outlet set approximately 3 feet below the top of the existing NE B-18 Containment Basin berm will prevent overtopping of this basin during the 24-hour, PMP event. The peak flows from the orifice outlet will be 17 cfs. The excess water from the outlet system will be conveyed by gravity pipe to the site's existing East Retention Basin located approximately 2,000 feet to the north.

7.0 REFERENCES

Hydraulic Toolbox [computer software] 2011 Federal Highway Administration (FHWA), Version 2.1

Ernest F. Brater and Horace H. King 1976. Handbook of Hydraulics, 6th edition. McGraw-Hill Inc.

U.S Department of Commerce, National Oceanic and Atmospheric Administration, U.S. Army Corps of Engineers. 1999. *Hydrometeorological Report No. 59* Probable Maximum Precipitation for California.

HEC-HMS Hydrologic Modeling System [computer software] US Army Corps of Engineers Version 3.1.0

8.0 ATTACHMENTS

Figure 1: Watershed Area for Phase IIIA Temporary Stormwater Berm

Attachment 1: HEC-HMS Kettleman B-18 Basin Schematic

Attachment 2: HEC-HMS NE B-18 Containment Basin Outlet Control Discharge Results

Attachment 3: NE B-18 Containment Basin Conveyance Pipe Calculation Results

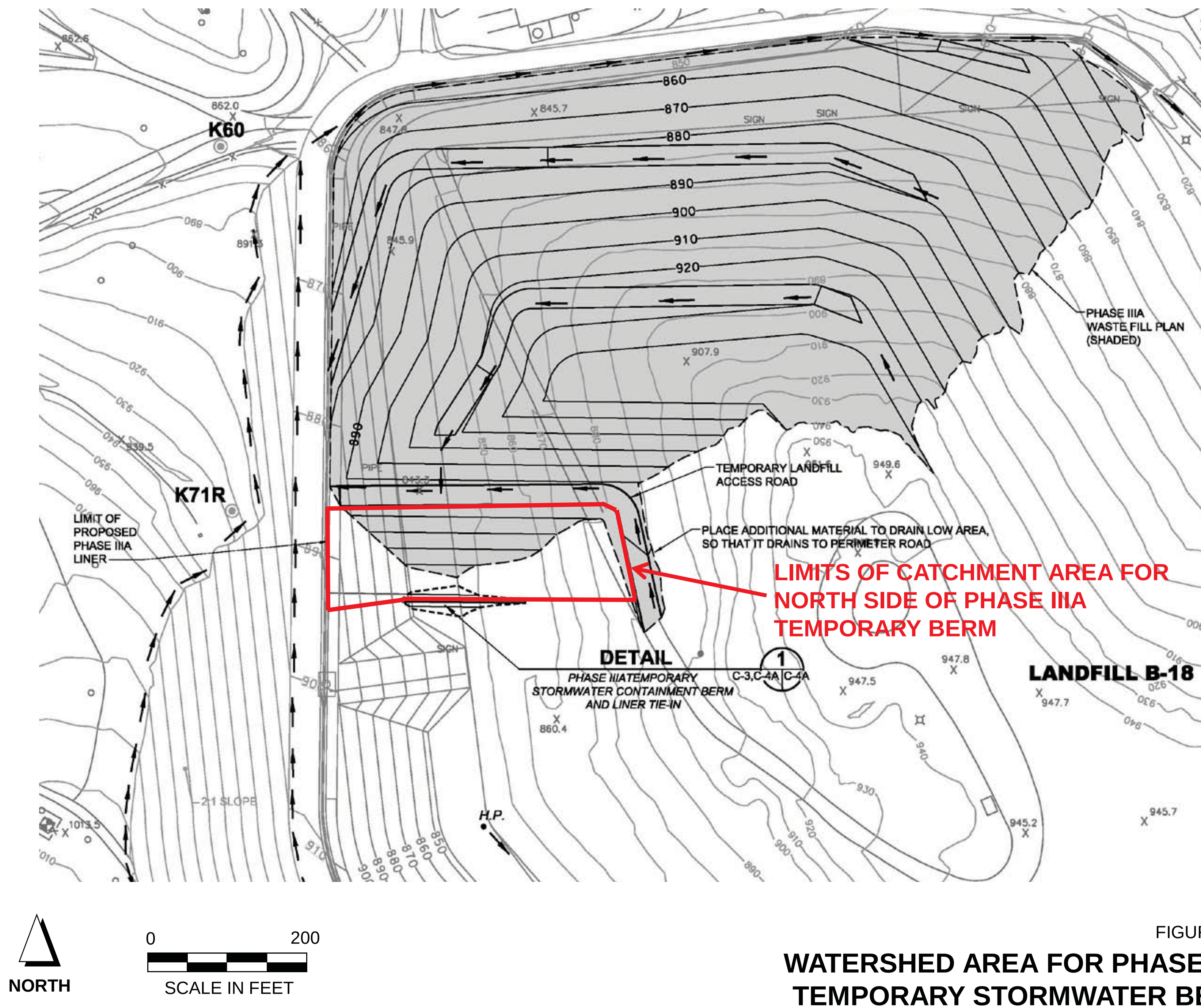


FIGURE 1

WATERSHED AREA FOR PHASE IIIA TEMPORARY STORMWATER BERM

KETTLEMAN HILLS FACILITY, LANDFILL UNIT B-18

PROJECT NO.: 083-91887 DRAWN BY: RH CHECKED BY: RH DATE: AUGUST 2011

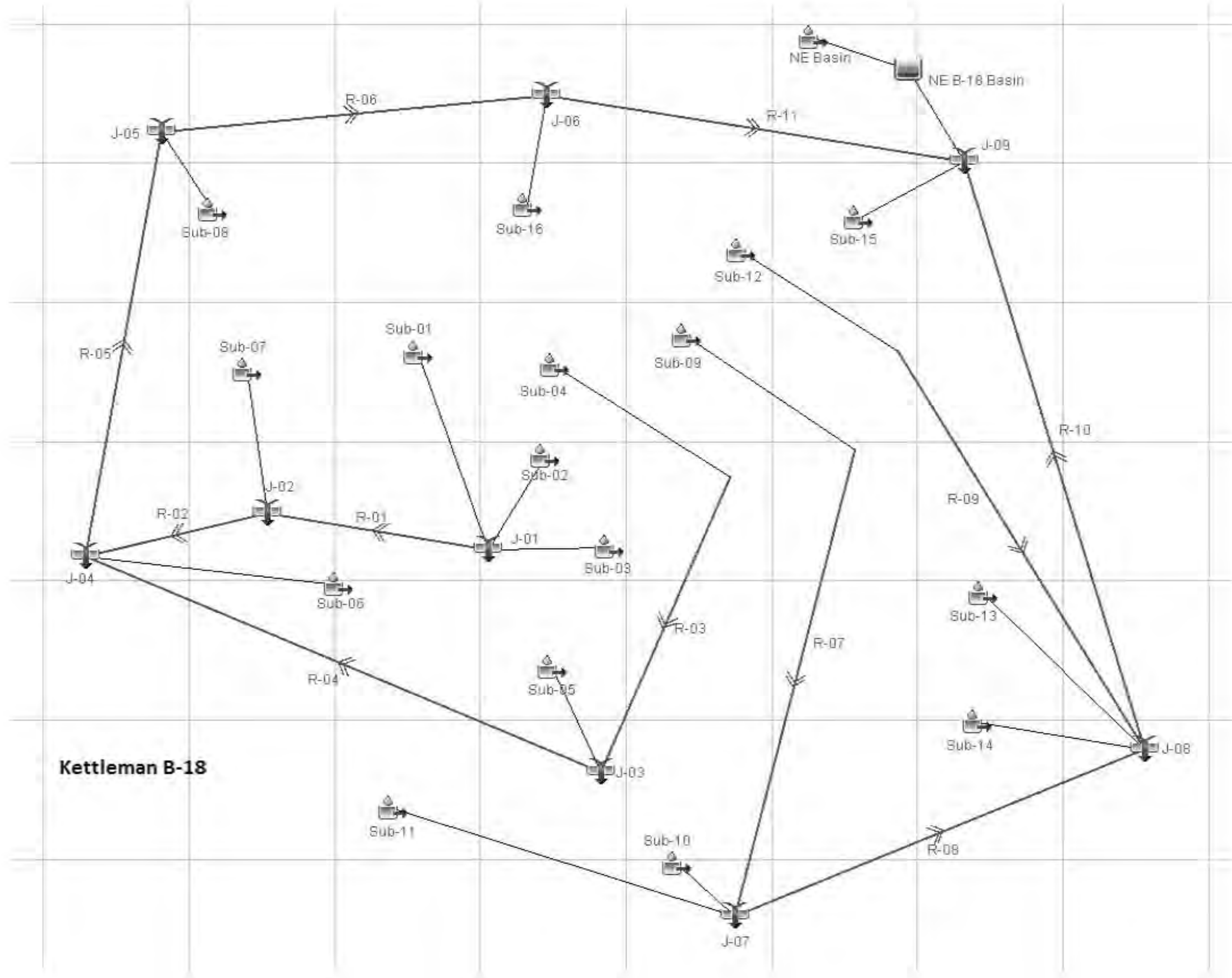
Golder Associates

Attachment 1

HEC-HMS Kettleman B-18 Basin Schematic

Attachment 2
HEC-HMS
B-18 Cover
Basin Layout

HEC-HMS Basin Model Schematic



Golder Associates, Inc.

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8/10/2011

Attachment 2

HEC-HMS NE B-18 Containment Basin Outlet Control Discharge Results

Project: Kettleman_B18_Rev2

Simulation Run: PMP24hr-Outlet Control Reservoir: NE B-18 Basin

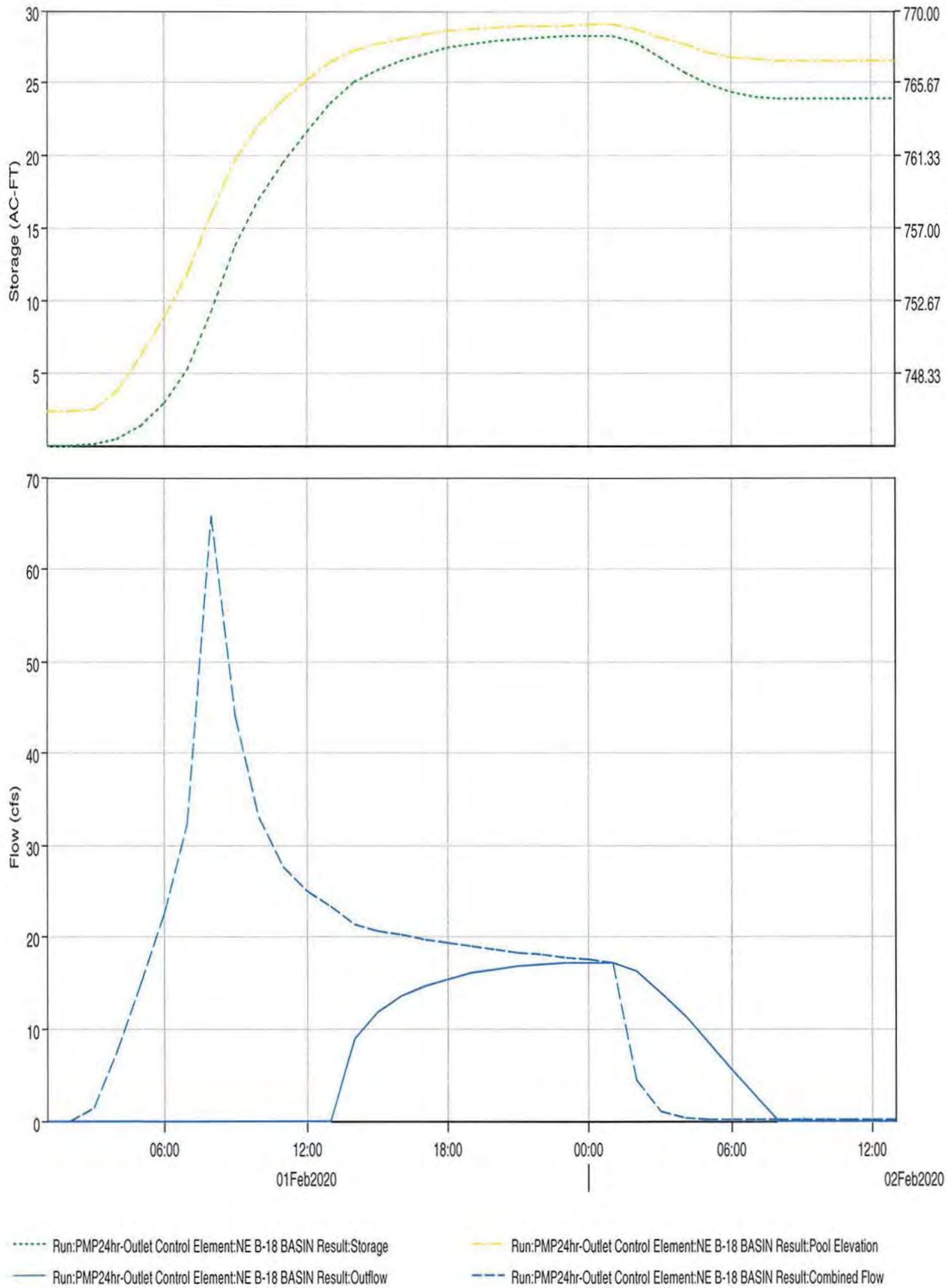
Start of Run:	01Feb2020, 01:00	Basin Model:	B-18 Cover-Outlet Control
End of Run:	02Feb2020, 13:00	Meteorologic Model:	LocalPMP24hr
Compute Time:	10Aug2011, 10:15:53	Control Specifications:	1Hr36Hr

Volume Units: AC-FT

Computed Results

Peak Inflow :	65.7 (CFS)	Date/Time of Peak Inflow :	01Feb2020, 08:00
Peak Outflow :	17.2 (CFS)	Date/Time of Peak Outflow :	02Feb2020, 01:00
Total Inflow :	43.8 (AC-FT)	Peak Storage :	28.3 (AC-FT)
Total Outflow :	19.9 (AC-FT)	Peak Elevation :	769.1 (FT)

Reservoir "NE B-18 Basin" Results for Run "PMP24hr-Outlet Control"



Project: Kettleman_B18_Rev2
Simulation Run: PMP24hr-Outlet Control Reservoir: NE B-18 Basin

Start of Run: 01Feb2020, 01:00 Basin Model: B-18 Cover-Outlet C
End of Run: 02Feb2020, 13:00 Meteorologic Model: LocalPMP24hr
Compute Time: 10Aug2011, 10:15:53 Control Specifications: 1Hr36Hr

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Feb2020	01:00	0.0	0.0	746.0	0.0
01Feb2020	02:00	0.0	0.0	746.0	0.0
01Feb2020	03:00	1.3	0.1	746.2	0.0
01Feb2020	04:00	7.4	0.4	747.2	0.0
01Feb2020	05:00	14.9	1.3	749.3	0.0
01Feb2020	06:00	22.7	2.9	751.7	0.0
01Feb2020	07:00	32.4	5.2	754.2	0.0
01Feb2020	08:00	65.7	9.2	757.8	0.0
01Feb2020	09:00	44.1	13.7	761.1	0.0
01Feb2020	10:00	33.0	16.9	763.1	0.0
01Feb2020	11:00	27.6	19.4	764.6	0.0
01Feb2020	12:00	25.0	21.6	765.8	0.0
01Feb2020	13:00	23.3	23.6	766.8	0.0
01Feb2020	14:00	21.4	25.0	767.6	8.9
01Feb2020	15:00	20.7	25.9	768.0	11.9
01Feb2020	16:00	20.2	26.5	768.3	13.5
01Feb2020	17:00	19.8	27.0	768.5	14.6
01Feb2020	18:00	19.4	27.4	768.7	15.4
01Feb2020	19:00	19.0	27.7	768.9	16.0
01Feb2020	20:00	18.6	27.9	769.0	16.5
01Feb2020	21:00	18.3	28.0	769.0	16.8
01Feb2020	22:00	18.0	28.1	769.1	17.0
01Feb2020	23:00	17.7	28.2	769.1	17.1
02Feb2020	00:00	17.5	28.2	769.1	17.2
02Feb2020	01:00	17.1	28.3	769.1	17.2

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
02Feb2020	02:00	4.3	27.8	768.9	16.2
02Feb2020	03:00	1.0	26.7	768.4	14.0
02Feb2020	04:00	0.2	25.7	767.9	11.4
02Feb2020	05:00	0.1	24.9	767.5	8.5
02Feb2020	06:00	0.0	24.3	767.2	5.6
02Feb2020	07:00	0.0	24.0	767.0	2.6
02Feb2020	08:00	0.0	23.9	767.0	0.0
02Feb2020	09:00	0.0	23.9	767.0	0.0
02Feb2020	10:00	0.0	23.9	767.0	0.0
02Feb2020	11:00	0.0	23.9	767.0	0.0
02Feb2020	12:00	0.0	23.9	767.0	0.0
02Feb2020	13:00	0.0	23.9	767.0	0.0

Attachment 3

NE B-18 Containment Basin Conveyance Pipe Calculation Results

Pipe Flow Results

Project Data

Project Title: Kettleman B-18 Phase IIIA

Designer: Golder

Project Date: Wednesday, August 10, 2011

Project Units: U.S. Customary Units

Channel Analysis: 18" Pipe - 10 cfs

Input Parameters

Channel Type: Circular

Pipe Diameter: 1.5000 (ft)

Longitudinal Slope: 0.0100 (ft/ft)

Manning's n: 0.0100

Flow: 10.0000 (cfs)

Result Parameters

Depth: 0.9533 (ft)

Area of Flow: 1.1848 (ft^2)

Wetted Perimeter: 2.7680 (ft)

Hydraulic Radius: 0.4280 (ft)

Average Velocity: 8.4402 (ft/s)

Top Width: 1.4438 (ft)

Froude Number: 1.6419

Critical Depth: 1.2188 (ft)

Critical Velocity: 8.8096 (ft/s)

Critical Slope: 0.0054 (ft/ft)

Critical Top Width: 1.1709 (ft)

Calculated Max Shear Stress: 0.5949 (lb/ft^2)

Calculated Avg Shear Stress: 0.2671 (lb/ft^2)

Channel Analysis: 21" Pipe - 18 cfs

Input Parameters

Channel Type: Circular

Pipe Diameter: 1.7500 (ft)

Longitudinal Slope: 0.0100 (ft/ft)

Manning's n: 0.0100

Flow: 18.0000 (cfs)

Result Parameters

Depth: 1.2668 (ft)

Area of Flow: 1.8647 (ft^2)

Wetted Perimeter: 3.5614 (ft)

Hydraulic Radius: 0.5236 (ft)

Average Velocity: 9.6531 (ft/s)

Top Width: 1.5647 (ft)

Froude Number: 1.5583

Critical Depth: 1.5441 (ft)

Critical Velocity: 9.6806 (ft/s)

Critical Slope: 0.0069 (ft/ft)

Critical Top Width: 1.1278 (ft)

Calculated Max Shear Stress: 0.7905 (lb/ft^2)

Calculated Avg Shear Stress: 0.3267 (lb/ft^2)

APPENDIX J.4
FINAL CLOSURE DRAINAGE

OBJECTIVE:

The cover system and drainage control systems for the existing Landfill B-18 are required to be designed to function without failure when subjected to capacity, hydrostatic and hydrodynamic loads resulting from a 24-hour, Probable Maximum Precipitation storm [CCR 22, 66264.25]. Design surface water conveyance channels and bench channels, design the proposed retention pond (Reservoir 1) and analyze the existing retention pond (Reservoir 2) for the Kettleman Hills B-18 Landfill cover configuration. All runoff from the B-18 Landfill configuration is to be routed to the proposed retention pond (Reservoir 1) located on the southeast section of the landfill or to the existing retention pond (Reservoir 2) located on the northeast section of the landfill.

METHOD:

The local PMP (Probable Maximum Precipitation) storm event results in a higher precipitation intensity and thus higher peak channel flow, the 6-hour PMP, was used to evaluate all channels. The 24-hour rainfall event for the PMP was used for evaluating retention volume. The 6-hour and 24-hour PMPs were derived from the Hydrometeorological Report No. 59 (Reference Attachment C). The surface water parameters as described below were used to model the Kettleman Hills B-18 Landfill. Figure 1 presents the watershed delineation map for sub-basin boundaries. Figures 2 & 3 present typical drainage channel geometries. Basin areas and curve numbers (CNs) were entered into HEC-HMS modeling software (USACE) and routed to calculate the peak flows for each basin. Kinematic wave transform methodology was used to develop hydrographs for each sub-basin except for offsite sub-basins Offsite 3, Offsite 4, and Offsite 5 which were modeled using the SCS unit hydrograph method. The peak flows were then used to size the landfill perimeter and bench channels, assuming normal depth. All model output can be found attached in this appendix.

ASSUMPTIONS:

- The 6-hour rainfall event for the local PMP equals 6.5 inches (used for designing channels). Ref. Attachment C.
- The 24-hour rainfall event for the PMP equals 10.3 inches (used for checking retention volume). Ref. Attachment C.
- SCS Type I rainfall synthetic distribution.
- Landfill final cover SCS Curve Numbers (CN):

Location	Soil Type	Hydrologic Soil Group	Assumed Cover	SCS CN
Landfill Final Cover	Mercey Loam	C	Herbaceous: fair cover	81
Natural Terrain (south)	Mercey Loam	C	Herbaceous: fair cover	81
Natural Terrain (west)	Mercey Loam	C	Herbaceous: good cover	74

- Manning's n for routing and channel design:

Channel Lining	Manning's n for Stability	Manning's n for Capacity
Grass	0.030	0.035
Turf Reinf. Mat	0.030	0.035
Rip-rap	0.035	0.040
Asphalt	0.016	0.016

- Grass lining was used with velocities up to 7 fps, turf reinforcement mat was used with velocities up to 10 fps or at directional changes in grass lined channels, and hard lined (asphalt or shotcrete) or riprap for velocities higher than 10 fps. Given the short term nature of the peak velocity during the conservatively assumed Local PMP, these velocities are considered acceptable.

CALCULATIONS:

The HEC-HMS modeling software (USACE) was used to calculate flows at design points and the retention volume. All channel calculations were performed using a spreadsheet to calculate normal depth for both stability and capacity and FlowMaster to evaluate road/ditch sections.

CONCLUSIONS/RESULTS:

The proposed retention pond (Reservoir 1) and the existing retention pond (Reservoir 2) have the capacity to contain the PMP (Probable Maximum Precipitation) flows. Subbasin delineation can be found in **Figure 1**. Attached are spreadsheet and other calculations. A summary of subbasins can be found in **Table 1**, times of concentration can be found in **Table 2**, flow results from HEC-HMS in **Table 3**. Also attached are pond sizing calculations, HEC-HMS input & routing diagrams, and Flowmaster calculations for channels.

Based on the calculations, the maximum velocity for the bench channel on the landfill is 6.1feet per second. This is below the design criteria of 7 feet per second for grass-lined channels. The closure access road will contain stormwater flows within the asphalt lined channel. The peak velocity within the asphalt-lined channel during a PMP, 6-hour event will be 23.4 feet per second which is below the maximum allowable velocity of 25 feet per second. The perimeter road channel will exceed the flow capacity of the roadside asphalt-lined channel. The peak PMP, 6-hour stormwater flows will be contained within the roadway. Velocities within the channel will be less than the maximum allowable 25 feet per second. During the 24-hour PMP, it is predicted that run-off to the existing retention basin (Reservoir 2) located on the north side of the proposed landfill will exceed capacity by approximately 2 AC-FT. In the event of a PMP storm event, the excess stormwater will have to be pumped to the proposed retention basin (Reservoir 1).

REFERENCES:

HEC-HMS Hydrologic Modeling System [computer software] US Army Corps of Engineers Version 3.1.0

Bentley FlowMaster [computer software] 2005 Bentley Systems Inc. Service Pack 3

Natural Resource Conservation Service. 1986. Technical Release 55: Urban Hydrology for Small Watersheds. United States Department of Agriculture.

U.S. Bureau of Reclamation (USBR). 1977. *Design of small dams 2nd ed.* Washington D.C. : United States Government Printing Office.

Ernest F. Brater and Horace H. King 1976. Handbook of Hydraulics, 6th edition. McGraw-Hill Inc.

U.S Department of Commerce, National Oceanic and Atmospheric Administration, U.S. Army Corps of Engineers. 1999. *Hydrometeorological Report No. 59* Probable Maximum Precipitation for California.

TITLE

B-18 LANDFILL COVER
BASIN MAP

PROJECT

KETTLEMAN HILLS
B-18 LANDFILL HYDROLOGY
KETTLEMAN HILLS, CA.

Golder Associates
DENVER, COLORADO

PROJECT No. 063-61087
FILE No. 063616870A001
REV. A | SCALE 1"=300'
DESIGN PM 10/31/08
CADD PM 10/31/08
CHECK RH 11/4/08
REVIEW

FIGURE 1

Golder Associates

SUBJECT: TYPICAL DRAINAGE CHANNELS		
Job No.	Made by SGS	Date 9-11-08
Ref. KHFB18	Checked RH	Sheet 1 of 1
	Reviewed	

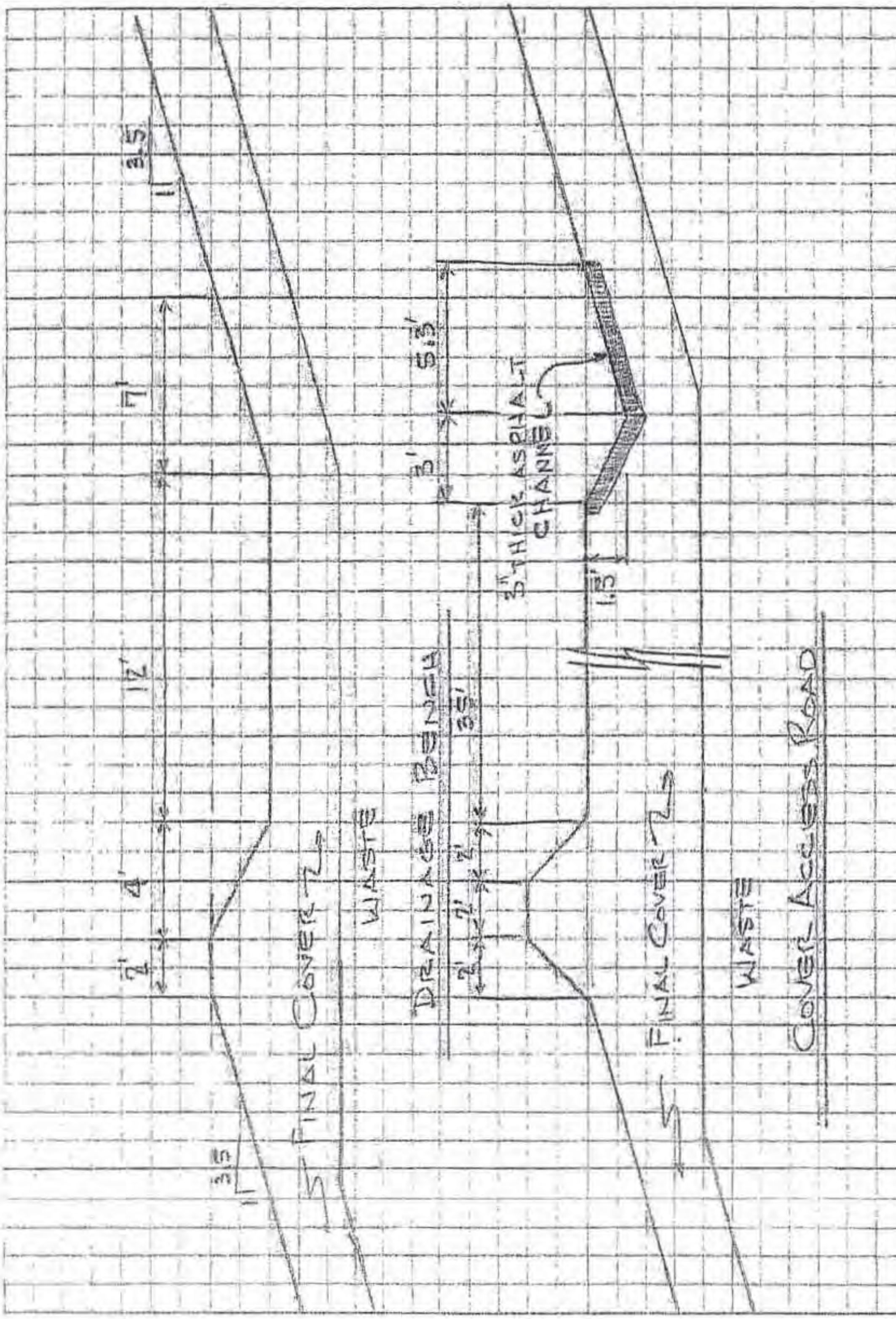


FIGURE 2.
DRAINAGE BENCH & COVER ACCESS ROAD CHANNEL CONFIGURATIONS (TYPICAL)

Golder Associates

SUBJECT TYPICAL DRAINAGE CHANNELS		
Job No.	Made by SGS	Date 9-11-08
Ref. KHF 818	Checked KH	Sheet 1 of 1
	Reviewed	

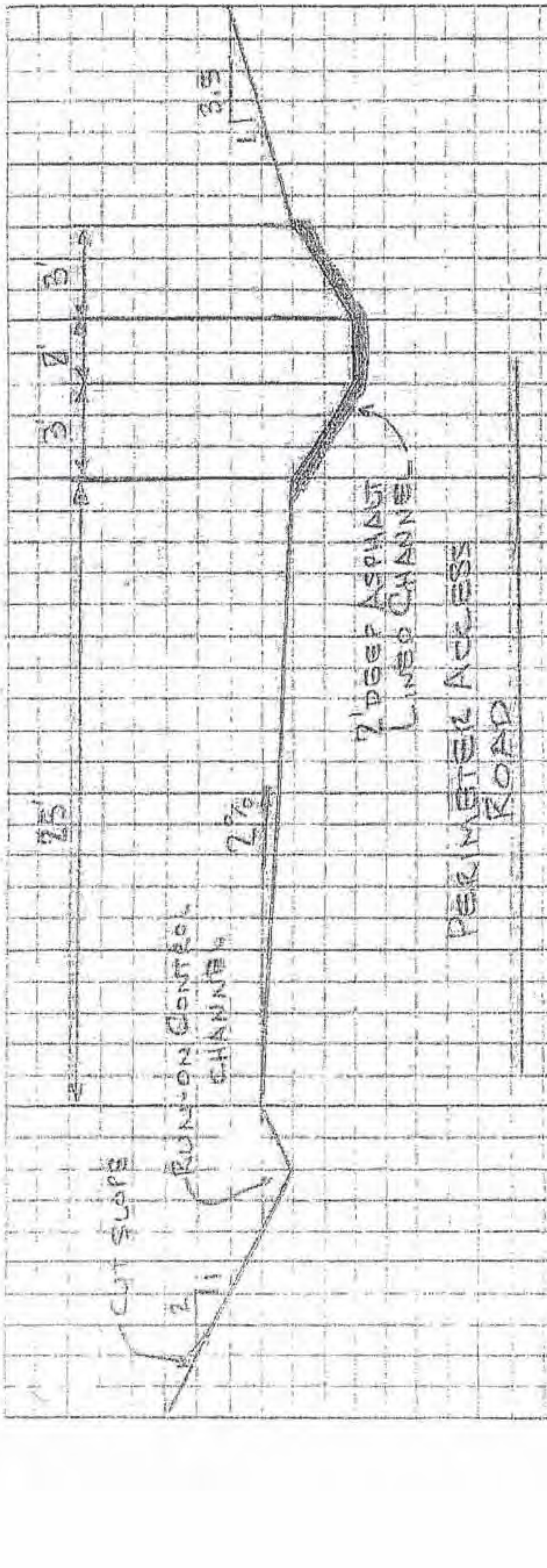


FIGURE 3.
TYPICAL PERIMETER ACCESS ROAD CHANNEL CONFIGURATION

TABLE 1
SUBBASIN SUMMARY TABLE

Kettleman B-18 Hydrology
Project Number: 083-91887

Date:	10/31/08
By:	PM
Chkd:	RH
Apprvd:	[signature]

Design Storm ... PMP - Recurrance Interval

Storm Duration (hours)	2-Year Depth (inches)	PMP - Depth (inches)	Storm Distribution
24	0.94	10.30	I

Subbasin ID	Subbasin Area (ft^2)	Subbasin Area (acres)	Subbasin Area (sq mile)	CN = 81 Covered Landfill HSG C (acres)	Open Rangeland HSG C (acres)	CN = 84 Pond Area (acres)	Composite SCS Curve No.	$S = \frac{1000}{CN} - 10$	Unit Runoff Q (in)	Runoff Volume (ac-ft)	Runoff Volume (ft^3)
North 01	43,996	1.01	0.0016	1.01			CN = 81	2.35	7.94	0.67	29,100
North 02	175,111	4.02	0.0063	4.02			CN = 81	2.35	7.94	2.66	115,822
North 03	87,556	2.01	0.0031	2.01			CN = 81	2.35	7.94	1.33	57,911
North 04	133,294	3.06	0.0048	3.06			CN = 81	2.35	7.94	2.02	88,163
North 05	108,029	2.48	0.0039	2.48			CN = 81	2.35	7.94	1.64	71,453
North 06	159,430	3.66	0.0057	3.66			CN = 81	2.35	7.94	2.42	105,450
North 07	159,430	3.66	0.0057	3.66			CN = 81	2.35	7.94	2.42	105,450
North 08	157,252	3.61	0.0056	3.61			CN = 81	2.35	7.94	2.39	104,010
North 09	30,928	0.71	0.0011	0.71			CN = 81	2.35	7.94	0.47	20,456
North 10	212,137	4.87	0.0076	4.87			CN = 81	2.35	7.94	3.22	140,312
North 11	117,176	2.69	0.0042	2.69			CN = 81	2.35	7.94	1.78	77,503
North 12	59,242	1.36	0.0021	1.36			CN = 81	2.35	7.94	0.90	39,184
North 13	47,480	1.09	0.0017	1.09			CN = 81	2.35	7.94	0.72	31,405
North 14	59,677	1.37	0.0021	1.37			CN = 81	2.35	7.94	0.91	39,472
South 01	97,574	2.24	0.0035	2.24			CN = 81	2.35	7.94	1.48	64,538
South 02	99,317	2.28	0.0036	2.28			CN = 81	2.35	7.94	1.51	65,690
South 03	101,059	2.32	0.0036	2.32			CN = 81	2.35	7.94	1.53	66,843
South 04	82,328	1.89	0.0030	1.89			CN = 81	2.35	7.94	1.25	54,454
South 05	177,725	4.08	0.0064	4.08			CN = 81	2.35	7.94	2.70	117,551
South 06	68,389	1.57	0.0025	1.57			CN = 81	2.35	7.94	1.04	45,234
South 07	153,331	3.52	0.0055	3.52			CN = 81	2.35	7.94	2.33	101,417
South 08	158,558	3.64	0.0057	3.64			CN = 81	2.35	7.94	2.41	104,874
South 09	187,744	4.31	0.0067	4.31			CN = 81	2.35	7.94	2.85	124,178
South 10	117,612	2.70	0.0042	2.70			CN = 81	2.35	7.94	1.79	77,791
South 11	170,320	3.91	0.0061	3.91			CN = 81	2.35	7.94	2.59	112,653
South 12	230,868	5.30	0.0083	5.30			CN = 81	2.35	7.94	3.51	152,701
South 13	23,522	0.54	0.0008	0.54			CN = 81	2.35	7.94	0.36	15,558
OffSite 01	355,450	8.16	0.0128		8.16		CN = 74	3.51	7.03	4.78	208,097
OffSite 02	211,702	4.86	0.0076		4.86		CN = 81	2.35	7.94	3.21	140,024
OffSite 03	637,718	14.64	0.0229		14.64		CN = 81	2.35	7.94	9.68	421,800
OffSite 04	399,010	9.16	0.0143			9.16	CN = 84	1.90	8.32	6.35	276,684
OffSite 05	153,767	3.53	0.0055			3.53	CN = 84	1.90	8.32	2.45	106,626
Total:	4,976,730	114.25	0.18							75.35	3,282,402

TOTAL RUNOFF TO RESERVOIR 1 DURING PMP STORM: 31.8 AC-FT

TOTAL RUNOFF TO RESERVOIR 2 DURING PMP STORM: 34.1 AC-FT
STORAGE CAPACITY OF RESERVOIR 2: 32.5 AC-FT
EXCESS RUNOFF TO BE PUMPED OR DIVERTED OFF-SITE: 2.2 AC-FT

TABLE 2
BASIN TIME OF CONCENTRATION CALCULATIONS

Project Name : Kettleman Hydrology
Project Number: 083-91887

Date:	10/31/08
By:	PM
Chkd:	RH
Apprvd:	

					Flow Segment 1							Flow Segment 2						
Subbasin ID	Subbasin Area (sq mile)	Composite Curve Number	Total Lag (0.6*Tc) (min)	Total Travel Time (min)	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition[1]		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition[1]		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)
North 01	0.0016	81	0.0	0.0														
North 02	0.0063	81	0.0	0.0														
North 03	0.0031	81	0.0	0.0														
North 04	0.0048	81	0.0	0.0														
North 05	0.0039	81	0.0	0.0														
North 06	0.0057	81	0.0	0.0														
North 07	0.0057	81	0.0	0.0														
North 08	0.0056	81	0.0	0.0														
North 09	0.0011	81	0.0	0.0														
North 10	0.0076	81	0.0	0.0														
North 11	0.0042	81	0.0	0.0														
North 12	0.0021	81	0.0	0.0														
North 13	0.0017	81	0.0	0.0														
North 14	0.0021	81	0.0	0.0														
South 01	0.0035	81	0.0	0.0														
South 02	0.0036	81	0.0	0.0														
South 03	0.0036	81	0.0	0.0														
South 04	0.0030	81	0.0	0.0														
South 05	0.0064	81	0.0	0.0														
South 06	0.0025	81	0.0	0.0														
South 07	0.0055	81	0.0	0.0														
South 08	0.0057	81	0.0	0.0														
South 09	0.0067	81	0.0	0.0														
South 10	0.0042	81	0.0	0.0														
South 11	0.0061	81	0.0	0.0														
South 12	0.0083	81	0.0	0.0														
South 13	0.0008	81	0.0	0.0														
OffSite 01	0.0128	72	0.0	0.0														
OffSite 02	0.0076	81	0.0	0.0														
OffSite 03	0.0229	81	5.1	8.5	Sheet	100.0	0.420	H	Range		4.8	Shallow	1200.0	0.110	U	Unpaved		3.7
OffSite 04	0.0143	84	2.8	4.7	Sheet	65.0	0.500	H	Range		3.2	Shallow	600.0	0.160	U	Unpaved		1.5
OffSite 05	0.0055	84	2.8	4.7	Sheet	50.0	0.020	B	Fallow		4.3	Shallow	60.0	0.020	U	Unpaved		0.4

Notes:
(1) Refer to Attachment A for Roughness Condition descriptions and Tc Coefficients.

TABLE 3
FLOW RESULTS FROM HEC-HMS

Kettleman B-18 Hydrology
Project Number: 083-91887

Date:	10/31/08
By:	PM
Chkd:	RH
Apprvd:	

HEC-HMS Basin Model:	Kettleman B-18
HEC-HMS Met. Model:	Local PMP 6hr
HEC-HMS Control Specs:	15min 24hr

Hydrologic Element	Drainage Area (sq mile)	Peak Discharge (cfs)	Time of Peak	Total Volume (ac-ft)
North 01	0.002	9.1	2:45	0.4
North 02	0.006	35.9	2:45	1.5
North 03	0.003	17.6	2:45	0.7
North 04	0.005	27.4	2:45	1.1
North 05	0.004	22.2	2:45	0.9
North 06	0.006	32.4	2:45	1.3
North 07	0.006	32.5	2:45	1.3
North 08	0.006	31.8	2:45	1.3
North 09	0.001	6.3	2:45	0.3
North 10	0.008	43.3	2:45	1.8
North 11	0.004	23.9	2:45	1.0
North 12	0.002	12.0	2:45	0.5
North 13	0.002	9.7	2:45	0.4
North 14	0.002	11.9	2:45	0.5
South 01	0.004	19.9	2:45	0.8
South 02	0.004	20.5	2:45	0.8
South 03	0.004	20.6	2:45	0.8
South 04	0.003	17.1	2:45	0.7
South 05	0.006	36.4	2:45	1.5
South 06	0.003	14.3	2:45	0.6
South 07	0.006	31.4	2:45	1.3
South 08	0.006	32.5	2:45	1.3
South 09	0.007	38.2	2:45	1.6
South 10	0.004	23.9	2:45	1.0
South 11	0.006	34.8	2:45	1.4
South 12	0.008	47.3	2:45	1.9
South 13	0.001	4.6	2:45	0.2
Offsite 01	0.013	58.1	2:45	2.5
Offsite 02	0.008	43.3	2:45	1.8
Offsite 03	0.023	104.2	2:45	5.3
Offsite 04	0.014	69.6	2:45	3.6
Offsite 05	0.006	26.8	2:45	1.4
J N01-N02	0.008	45.0	2:45	1.8
J N03-N04	0.016	87.4	2:45	3.7
J N05	0.040	196.7	2:45	8.9
J N06-N14	0.048	228.0	2:45	10.8
J N08	0.011	59.3	2:45	2.6
J N09	0.012	65.6	2:45	2.9
J N11	0.012	63.3	2:45	2.8
J N12	0.026	136.9	2:45	6.2
J N13	0.028	142.8	2:45	6.6
J O1-O2	0.036	187.2	2:45	8.0
J S03	0.007	37.3	2:45	1.7
J S04	0.007	36.1	2:45	1.5
J S05	0.020	109.2	2:45	4.7
J S06	0.041	206.8	2:45	9.6
J S08	0.011	58.9	2:45	2.6
J S09	0.018	88.6	2:45	4.2
J S10	0.059	277.1	2:45	14.1
J S12	0.014	75.0	2:45	3.4
J S13	0.060	273.7	2:45	14.4
Reach-1	0.004	19.0	2:45	4.4
Reach-2	0.020	103.9	2:45	4.4
Reach-3	0.008	42.3	2:45	4.4
Reach-4	0.036	176.4	2:45	4.1
Reach-5	0.040	183.6	2:45	4.2
Reach-6	0.004	16.7	2:45	4.4
Reach-7	0.006	26.3	2:45	4.4
Reach-8	0.011	50.4	2:45	4.5
Reach-9	0.006	27.6	2:45	4.4
Reach-10	0.041	178.3	2:45	4.5
Reach-11	0.059	269.1	2:45	4.5
Reach-12	0.006	27.4	2:45	4.4
Reach-13	0.012	61.7	2:45	4.5
Reach-14	0.008	39.3	2:45	4.4
Reach-15	0.026	133.2	2:45	4.5
Reach-16	0.007	30.7	2:45	4.4
Reach-17	0.016	85.7	2:45	4.4

TABLE 3
FLOW RESULTS FROM HEC-HMS

Kettleman B-18 Hydrology
Project Number: 083-91887

Date:	10/31/08
By:	PM
Chkd:	KH
Apprvd:	

HEC-HMS Basin Model: Kettleman B-18
HEC-HMS Met. Model: Local PMP 6hr
HEC-HMS Control Specs: 15min 24hr

Hydrologic Element	Drainage Area (sq mile)	Peak Discharge (cfs)	Time of Peak	Total Volume (ac-ft)
North 01	0.002	9.1	2:45	0.4
North 02	0.006	35.9	2:45	1.5
North 03	0.003	17.6	2:45	0.7
North 04	0.005	27.4	2:45	1.1
North 05	0.004	22.2	2:45	0.9
North 06	0.006	32.4	2:45	1.3
North 07	0.006	32.5	2:45	1.3
North 08	0.006	31.9	2:45	1.3
North 09	0.001	6.3	2:45	0.3
North 10	0.008	43.3	2:45	1.8
North 11	0.004	23.9	2:45	1.0
North 12	0.002	12.0	2:45	0.5
North 13	0.002	9.7	2:45	0.4
North 14	0.002	11.9	2:45	0.5
South 01	0.004	19.9	2:45	0.8
South 02	0.004	20.5	2:45	0.8
South 03	0.004	20.8	2:45	0.8
South 04	0.003	17.1	2:45	0.7
South 05	0.006	36.4	2:45	1.5
South 06	0.003	14.3	2:45	0.6
South 07	0.006	31.4	2:45	1.3
South 08	0.006	32.5	2:45	1.3
South 09	0.007	38.2	2:45	1.6
South 10	0.004	23.9	2:45	1.0
South 11	0.006	34.8	2:45	1.4
South 12	0.008	47.3	2:45	1.9
South 13	0.001	4.6	2:45	0.2
Offsite 01	0.013	58.1	2:45	2.5
Offsite 02	0.008	43.3	2:45	1.8
Offsite 03	0.023	104.2	2:45	5.3
Offsite 04	0.014	69.6	2:45	3.6
Offsite 05	0.006	26.8	2:45	1.4
J N01-N02	0.008	45.0	2:45	1.8
J N03-N04	0.016	87.4	2:45	3.7
J N05	0.040	196.7	2:45	8.9
J N06-N14	0.048	228.0	2:45	10.8
J N08	0.011	59.3	2:45	2.6
J N09	0.012	65.6	2:45	2.9
J N11	0.012	63.3	2:45	2.8
J N12	0.026	136.9	2:45	6.2
J N13	0.028	142.6	2:45	6.6
J O1-O2	0.036	187.2	2:45	8.0
J S03	0.007	37.3	2:45	1.7
J S04	0.007	36.1	2:45	1.5
J S05	0.020	109.2	2:45	4.7
J S06	0.041	206.8	2:45	9.6
J S08	0.011	58.9	2:45	2.6
J S09	0.016	88.6	2:45	4.2
J S10	0.059	277.1	2:45	14.1
J S12	0.014	75.0	2:45	3.4
J S13	0.060	273.7	2:45	14.4
Reach-1	0.004	19.0	2:45	4.4
Reach-2	0.020	103.9	2:45	4.4
Reach-3	0.008	42.3	2:45	4.4
Reach-4	0.036	176.4	2:45	4.1
Reach-5	0.040	183.6	2:45	4.2
Reach-6	0.004	16.7	2:45	4.4
Reach-7	0.006	26.3	2:45	4.4
Reach-8	0.011	50.4	2:45	4.5
Reach-9	0.005	27.6	2:45	4.4
Reach-10	0.041	176.3	2:45	4.5
Reach-11	0.059	269.1	2:45	4.5
Reach-12	0.006	27.4	2:45	4.4
Reach-13	0.012	61.7	2:45	4.5
Reach-14	0.008	39.3	2:45	4.4
Reach-15	0.026	133.2	2:45	4.5
Reach-16	0.007	36.7	2:45	4.4
Reach-17	0.016	85.7	2:45	4.4

Table 4
Channel Hydraulic Calculations

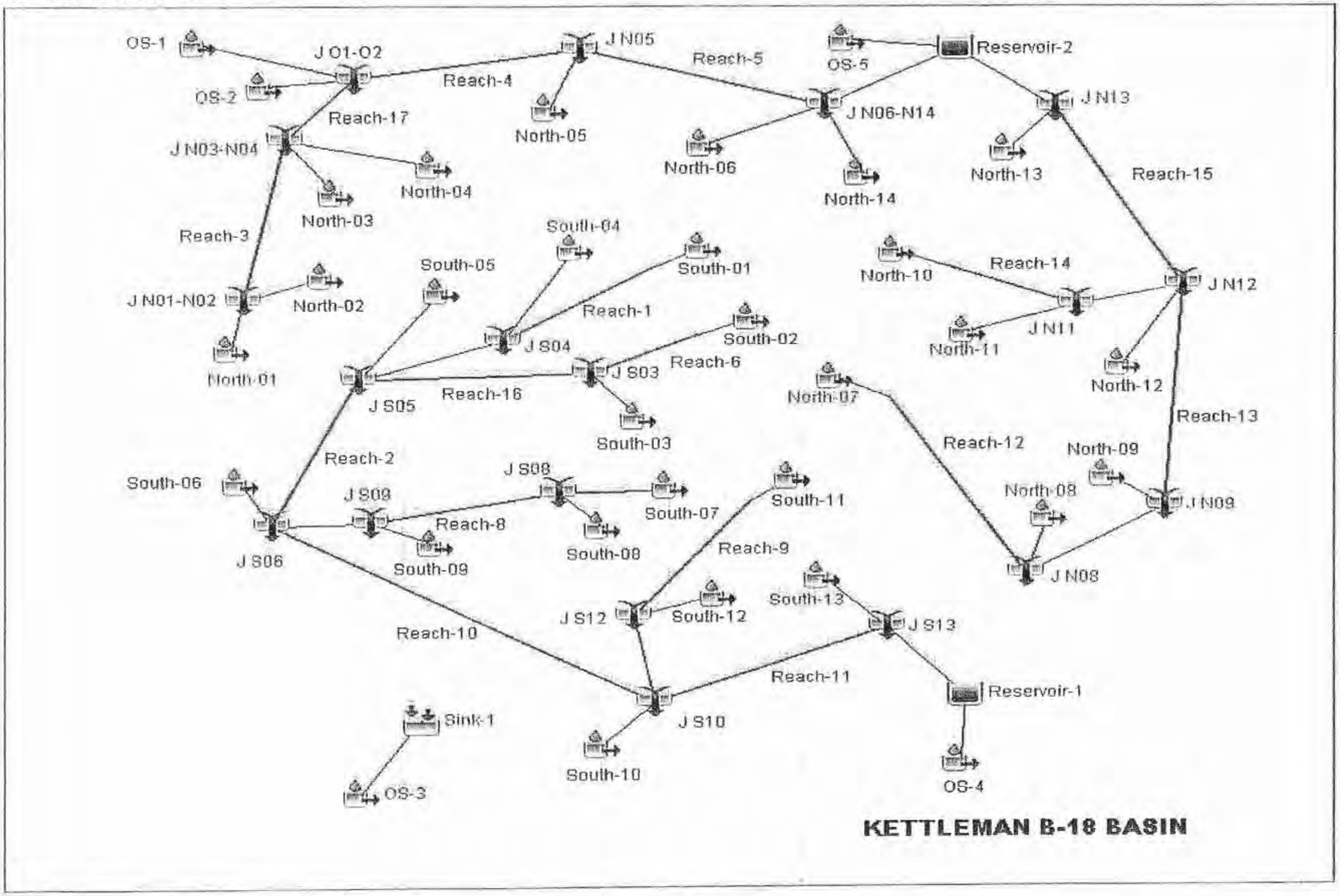
Kettleman B-18 Hydrology

Date:	10/31/08
By:	PM
Chkd:	RH
Apprvd:	

			Channel Design Geometry						Channel Roughness Parameters			Hydraulic Calculations			
Reach Designation	QPMP from HEC-HMS (cfs)	HEC HMS Element ID for Q	Approximate Channel Length (ft)	Channel Slope (ft/ft)	Left Side Slope (H:1V)	Right Side Slope (H:1V)	Bottom Width (ft)	Minimum Channel Depth (ft)	Design Channel Lining	Mannings 'n' for Capacity (Depth Calculation)	Mannings 'n' for Stability (Velocity Calculation)	Maximum Velocity (ft/sec)	Maximum Normal Flow Depth (ft)	Froude Number	Normal Depth Shear Stress (lb/ft^2)
Bench S01	19.9	South 01	770	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	3.7	0.45	1.04	0.56
Bench S02	20.5	South 02	910	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	3.7	0.46	1.05	0.57
Bench S03	16.7	Reach-6	1000	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	3.4	0.41	0.89	0.51
Cover S04	19.0	Reach-1	415	0.080	1.0	3.5	35	2.0	B Asphalt	0.016	0.016	12.9	0.73	3.75	3.64
Cover S03	36.7	Reach-16	175	0.080	1.0	3.5	35	2.0	G Grass-lined	0.035	0.030	7.1	0.04	1.80	0.21
Bench S05	36.4	South 05	1150	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	4.5	0.65	1.10	0.81
Cover S05	103.9	Reach-2	175	0.080	2.0	3.5	35	2.0	B Asphalt	0.016	0.016	8.5	1.64	2.81	8.19
Bench S07	31.4	South 07	990	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	4.3	0.59	1.08	0.74
Bench S08	26.3	Reach-7	980	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	4.4	0.53	0.93	0.66
Bench S09	50.4	Reach-8	1050	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	5.1	0.78	0.98	0.97
Perimeter S06	178.3	Reach-10	1290	0.014	2.0	3.5	25	3.0	G Grass-lined	0.035	0.030	5.3	2.84	0.93	2.48
Bench S11	34.8	South 11	975	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	4.5	0.63	1.09	0.79
Bench S12	27.6	Reach-9	1280	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	4.1	0.55	0.93	0.69
Bench S13	4.6	South 13	450	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	2.1	0.19	0.91	0.24
Perimeter S10	269.1	Reach-11	450	0.047	2.0	3.5	25	3.0	G Grass-lined	0.035	0.030	9.0	2.73	1.67	8.01
Bench N01	9.1	North 01	280	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	2.7	0.29	0.97	0.36
Bench N02	35.9	North 02	1050	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	4.5	0.64	1.10	0.80
Perimeter N02	42.3	Reach-3	610	0.060	2.0	3.5	25	3.0	B Asphalt	0.016	0.016	15.3	0.85	3.44	3.18
Bench N03	17.6	North 03	590	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	3.5	0.42	1.03	0.52
Bench N04	27.4	North 04	880	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	4.1	0.55	1.07	0.69
Perimeter N04	85.7	Reach-17	220	0.056	2.0	3.5	25	3.0	B Asphalt	0.016	0.016	18.0	1.24	3.46	4.33
Offsite-2 Ditch	43.3	Offsite 02	1010	0.075	3.0	3.0	0	2.0	B Asphalt	0.016	0.016	15.2	0.97	3.84	4.54
Perimeter O2	176.4	Reach-4	660	0.026	2.0	3.5	25	3.0	G Grass-lined	0.035	0.030	6.4	2.67	1.23	4.33
Bench N05	22.2	North 05	875	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	3.8	0.48	1.05	0.60
Perimeter N05	183.6	Reach-5	1175	0.060	2.0	3.5	25	3.0	G Grass-lined	0.035	0.030	8.7	2.51	1.80	9.40
Bench N06	32.4	North 06	1175	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	4.4	0.60	1.09	0.75
Bench N07	32.5	North 07	955	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	4.4	0.60	1.09	0.75
Bench N08	27.4	Reach-12	990	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	4.1	0.55	0.93	0.69
Perimeter N09	61.7	Reach-13	695	0.067	2.0	3.5	25	3.0	B Asphalt	0.016	0.016	17.6	1.00	3.70	4.18
Bench N10	43.3	North 10	1160	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	4.9	0.71	1.12	0.89
Bench N11	39.3	Reach-14	640	0.020	2.0	3.5	12	2.0	G Grass-lined	0.035	0.030	4.7	0.62	1.11	0.77
Perimeter N12	133.2	Reach-15	435	0.080	2.0	3.5	25	3.0	B Asphalt	0.016	0.016	23.0	1.41	4.20	7.04

Attachment B
HEC-HMS Screen Captures and Inputs

HEC-HMS Basin Model Schematic



Sub Basin Area	
Subbasin	Area (mi^2)
North 01	0.001600
North 02	0.006300
North 03	0.003100
North 04	0.004800
North 05	0.003900
North 06	0.005700
North 07	0.005700
North 08	0.005600
North 09	0.001100
North 10	0.007600
North 11	0.004200
North 12	0.002100
North 13	0.001700
North 14	0.002100
South 01	0.003500
South 02	0.003600
South 03	0.003600
South 04	0.003000
South 05	0.006400
South 06	0.002500
South 07	0.005500
South 08	0.005700
South 09	0.006700
South 10	0.004200
South 11	0.006100
South 12	0.008300
South 13	0.000800
Offsite 1	0.012800
Offsite 2	0.007600
Offsite 3	0.022900
Offsite 4	0.014300
Offsite 5	0.005500

Loss SCS Curve Number			
Subbasin	Initial Abstraction (in)	Curve Number	Impervious (%)
North 01		81	0
North 02		81	0
North 03		81	0
North 04		81	0
North 05		81	0
North 06		81	0
North 07		81	0
North 08		81	0
North 09		81	0
North 10		81	0
North 11		81	0
North 12		81	0
North 13		81	0
North 14		81	0
South 01		81	0
South 02		81	0
South 03		81	0
South 04		81	0
South 05		81	0
South 06		81	0
South 07		81	0
South 08		81	0
South 09		81	0
South 10		81	0
South 11		81	0
South 12		81	0
South 13		81	0
Offsite 1		74	0
Offsite 2		81	0
Offsite 3		81	0
Offsite 4		84	0
Offsite 5		84	0

Transform SCS Unit Hydrograph	
Subbasin	Lag Time (min)
Offsite 3	5.1
Offsite 4	2.8
Offsite 5	2.8

Kinematic Wave Transform (Main Channel)									
Subbsain	Route Upstream	Route Method	Length (ft)	Slope (ft/ft)	subreaches	Shape	Manning's n	Width (ft)	Side Slope (xH:1V)
North 01	No	Kinematic Wave	280	0.065	5	Trapezoid	0.035	12	3.25
North 02	No	Kinematic Wave	1050	0.020	5	Trapezoid	0.035	12	3.25
North 03	No	Kinematic Wave	590	0.056	5	Trapezoid	0.035	12	3.25
North 04	No	Kinematic Wave	880	0.020	5	Trapezoid	0.035	12	3.25
North 05	No	Kinematic Wave	875	0.026	5	Trapezoid	0.035	12	3.25
North 06	No	Kinematic Wave	1175	0.060	5	Trapezoid	0.035	12	3.25
North 07	No	Kinematic Wave	955	0.020	5	Trapezoid	0.035	12	3.25
North 08	No	Kinematic Wave	990	0.020	5	Trapezoid	0.035	12	3.25
North 09	No	Kinematic Wave	250	0.057	5	Trapezoid	0.035	12	3.25
North 10	No	Kinematic Wave	1160	0.022	5	Trapezoid	0.035	12	3.25
North 11	No	Kinematic Wave	640	0.020	5	Trapezoid	0.035	12	3.25
North 12	No	Kinematic Wave	695	0.057	5	Trapezoid	0.035	12	3.25
North 13	No	Kinematic Wave	435	0.080	5	Trapezoid	0.035	12	3.25
North 14	No	Kinematic Wave	290	0.042	5	Trapezoid	0.035	12	3.25
South 01	No	Kinematic Wave	770	0.020	5	Trapezoid	0.035	12	3.25
South 02	No	Kinematic Wave	910	0.020	5	Trapezoid	0.035	12	3.25
South 03	No	Kinematic Wave	1000	0.020	5	Trapezoid	0.035	12	3.25
South 04	No	Kinematic Wave	615	0.080	5	Trapezoid	0.035	12	3.25
South 05	No	Kinematic Wave	1150	0.020	5	Trapezoid	0.035	12	3.25
South 06	No	Kinematic Wave	650	0.007	5	Trapezoid	0.035	12	3.25
South 07	No	Kinematic Wave	990	0.020	5	Trapezoid	0.035	12	3.25
South 08	No	Kinematic Wave	980	0.020	5	Trapezoid	0.035	12	3.25
South 09	No	Kinematic Wave	1050	0.020	5	Trapezoid	0.035	12	3.25
South 10	No	Kinematic Wave	1290	0.014	5	Trapezoid	0.035	12	3.25
South 11	No	Kinematic Wave	975	0.020	5	Trapezoid	0.035	12	3.25
South 12	No	Kinematic Wave	1280	0.020	5	Trapezoid	0.035	12	3.25
South 13	No	Kinematic Wave	450	0.047	5	Trapezoid	0.035	12	3.25
Offsite 1	No	Kinematic Wave	1180	0.075	5	Trapezoid	0.035	12	3.25
Offsite 2	No	Kinematic Wave	1010	0.060	5	Trapezoid	0.035	12	3.25

Routing Kinematic Wave Channel								
Reach	Length (ft)	Slope (ft/ft)	Manning's n	subreaches	Shape	Diameter (ft)	Width (ft)	Side Slope (xH:1V)
Reach-1	415	0.060	0.035	2	Trapezoid		12	3.25
Reach-2	665	0.052	0.035	2	Trapezoid		12	3.25
Reach-3	610	0.060	0.035	2	Trapezoid		12	3.25
Reach-4	660	0.028	0.035	2	Trapezoid		12	3.25
Reach-5	1175	0.060	0.035	2	Trapezoid		12	3.25
Reach-6	1000	0.020	0.035	2	Trapezoid		12	3.25
Reach-7	980	0.020	0.035	2	Trapezoid		12	3.25
Reach-8	1050	0.020	0.035	2	Trapezoid		12	3.25
Reach-9	1280	0.020	0.035	2	Trapezoid		12	3.25
Reach-10	1290	0.014	0.035	2	Trapezoid		12	3.25
Reach-11	450	0.047	0.035	2	Trapezoid		12	3.25
Reach-12	990	0.020	0.035	2	Trapezoid		12	3.25
Reach-13	695	0.067	0.035	2	Trapezoid		12	3.25
Reach-14	640	0.020	0.035	2	Trapezoid		12	3.25
Reach-15	435	0.080	0.035	2	Trapezoid		12	3.25
Reach-16	175	0.080	0.035	2	Trapezoid		12	3.25
Reach-17	220	0.056	0.035	2	Trapezoid		12	3.25

Attachment C ~ Local Storm

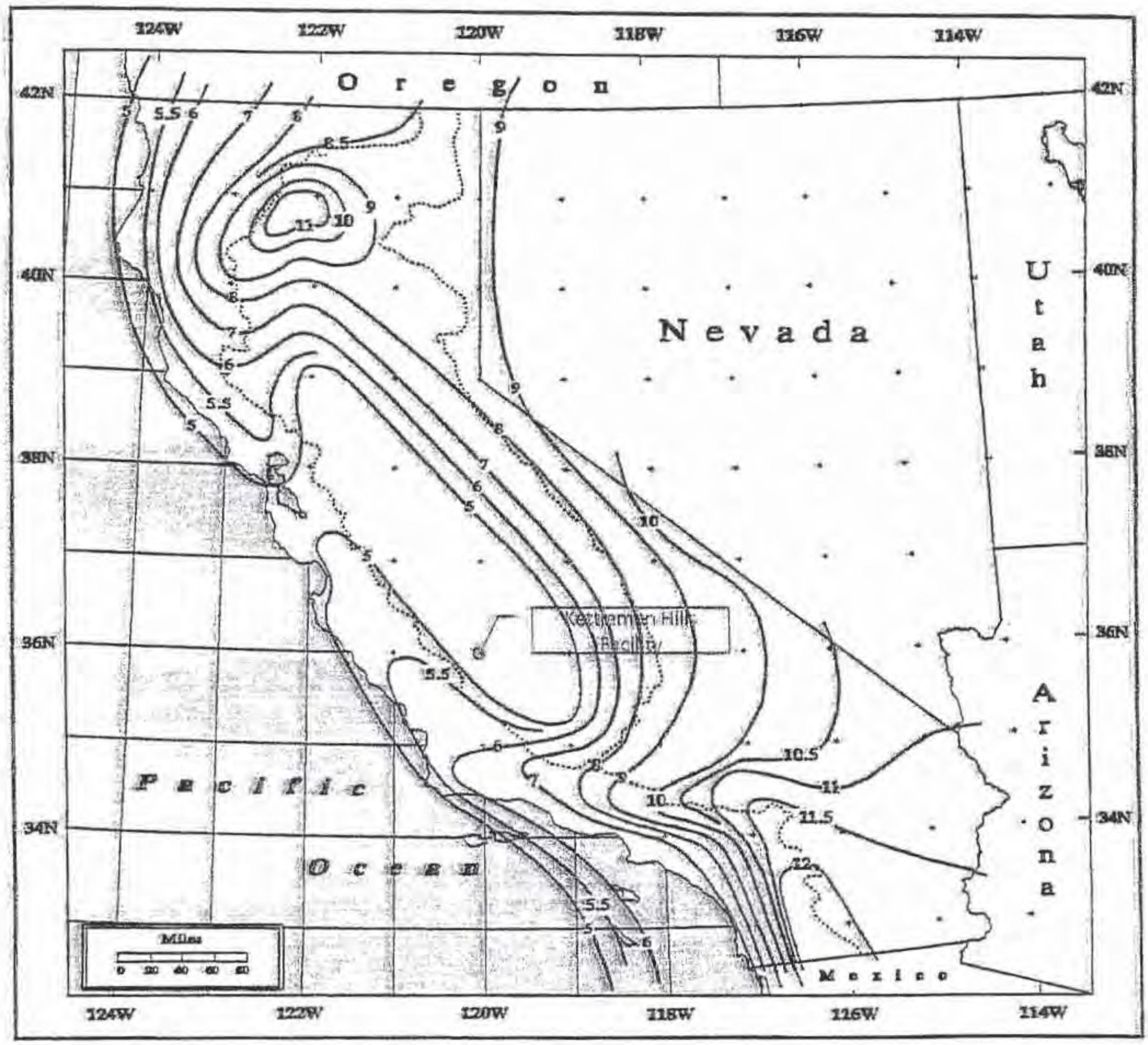
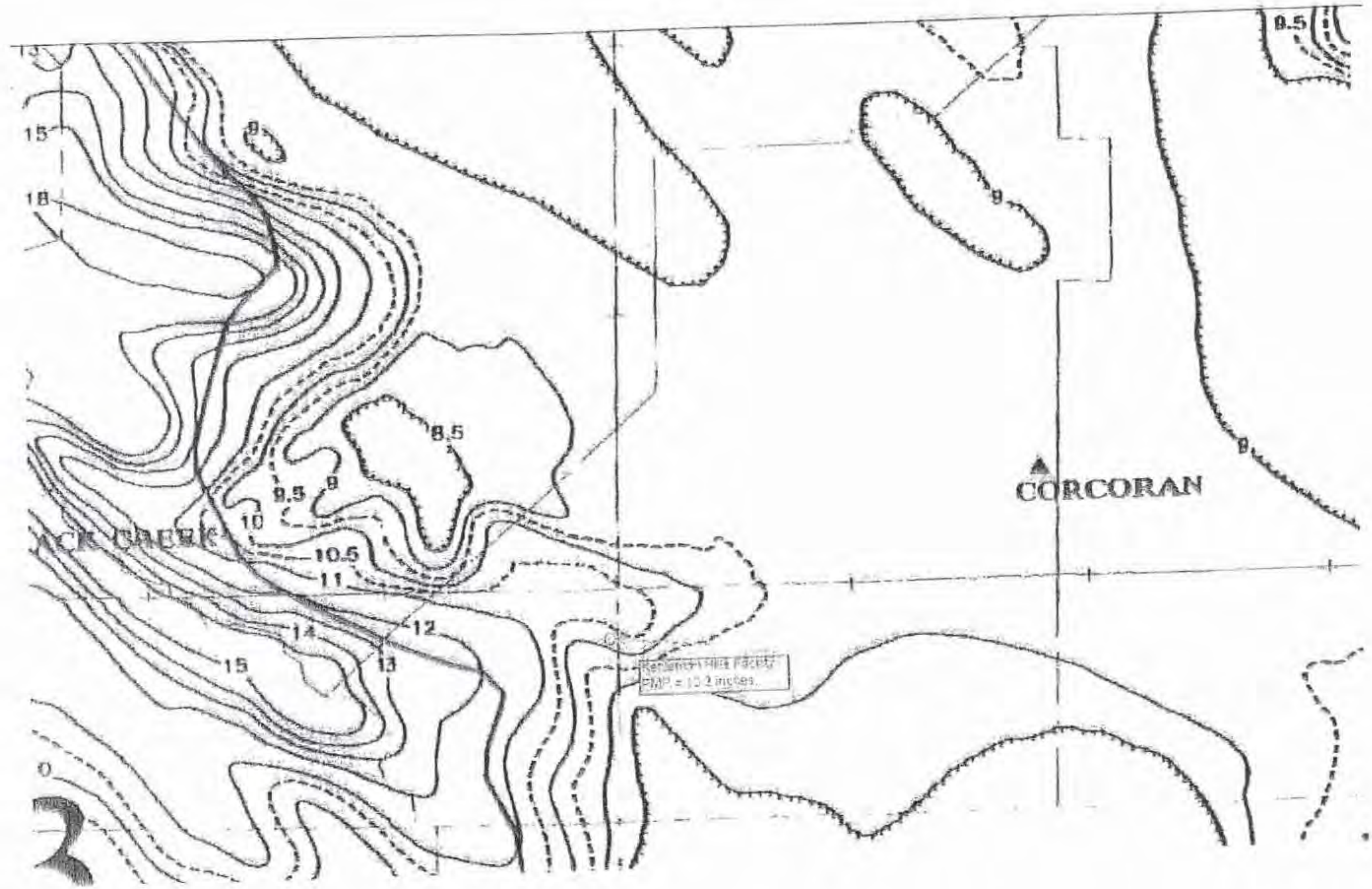


Figure 13.21. *California local-storm PMP precipitation estimates for 1 mi², 1 hour (inches). Dashed lines are drainage divides. Same as Figure 9.23.*

Figure 13.21 from HMR No. 59 is used to derive the 6 hour PMP.

Attachment C ~ General Storm



Project: Kettleman_Final_10-08
Simulation Run: PMP24hr Reservoir: Reservoir-1

Start of Run:	01Feb2020, 01:00	Basin Model:	Final Cover
End of Run:	02Feb2020, 13:00	Meteorologic Model:	LocalPMP24hr
Compute Time:	03Nov2008, 09:31:12	Control Specifications:	1Hr36Hr

Volume Units: AC-FT

Computed Results

Peak Inflow :	47.1 (CFS)	Date/Time of Peak Inflow :	01Feb2020, 08:00
Peak Outflow :	0.0 (CFS)	Date/Time of Peak Outflow :	01Feb2020, 01:00
Total Inflow :	31.8 (AC-FT)	Peak Storage (CAPACITY) 48	~~31.8~~ (AC-FT)
Total Outflow :	0.0 (AC-FT)	Peak Elevation : 857	~~849.3~~ (FT)

Project: Kettleman_Final_10-08
Simulation Run: PMP24hr Reservoir: Reservoir-2

Start of Run:	01Feb2020, 01:00	Basin Model:	Final Cover
End of Run:	02Feb2020, 13:00	Meteorologic Model:	LocalPMP24hr
Compute Time:	03Nov2008, 09:31:12	Control Specifications:	1Hr36Hr

Volume Units: AC-FT

Computed Results

Peak Inflow :	52.0 (CFS)	Date/Time of Peak Inflow :	01Feb2020, 08:00
Peak Outflow :	19.0 (CFS)	Date/Time of Peak Outflow :	02Feb2020, 01:00
Total Inflow :	34.1 (AC-FT)	Peak Storage :	32.5 (AC-FT)
Total Outflow :	2.2 (AC-FT)	Peak Elevation :	771.0 (FT)

Worksheet for Cover Road Reach 1

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.08000	ft/ft
Discharge	19.0	ft^3/s

Section Definitions

Station (ft)	Elevation (ft)
0+00.0	820.00
0+02.0	818.00
0+37.0	818.00
0+40.0	816.50
0+45.3	818.00
0+52.3	820.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.0, 820.00)	(0+02.0, 818.00)	0.030
(0+02.0, 818.00)	(0+37.0, 818.00)	0.030
(0+37.0, 818.00)	(0+45.3, 818.00)	0.016
(0+45.3, 818.00)	(0+52.3, 820.00)	0.030

Results

Normal Depth	0.73	ft
Elevation Range	816.50 to 820.00 ft	
Flow Area	1.48	ft^2
Wetted Perimeter	4.32	ft
Top Width	4.04	ft
Normal Depth	0.73	ft
Critical Depth	1.54	ft
Critical Slope	0.00671	ft/ft
Velocity	12.85	ft/s

Worksheet for Cover Road Reach 1

Results

Velocity Head		2.57	ft
Specific Energy		3.30	ft
Froude Number		3.75	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.73	ft
Critical Depth	1.54	ft
Channel Slope	0.08000	ft/ft
Critical Slope	0.00671	ft/ft

Cross Section for Cover Road Reach 1

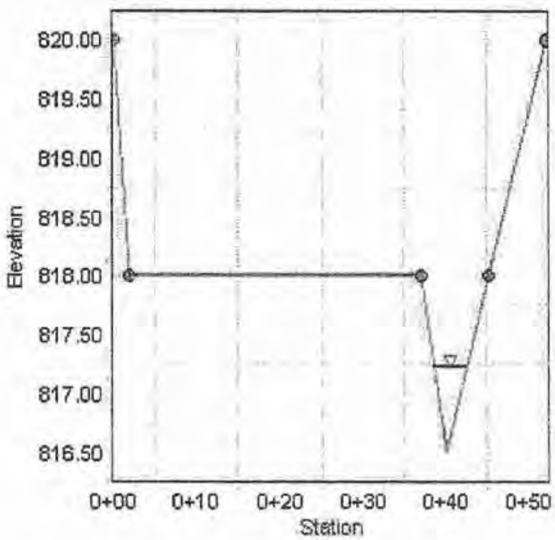
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.08000	ft/ft
Normal Depth	0.73	ft
Discharge	19.0	ft³/s

Cross Section Image



Worksheet for Cover Road Reach 2

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.08000	ft/ft
Discharge	103.9	ft^3/s
Section Definitions		

Station (ft)	Elevation (ft)
0+00.0	820.00
0+02.0	818.00
0+37.0	818.00
0+40.0	816.50
0+45.3	818.00
0+52.3	820.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.0, 820.00)	(0+02.0, 818.00)	0.030
(0+02.0, 818.00)	(0+37.0, 818.00)	0.022
(0+37.0, 818.00)	(0+45.3, 818.00)	0.016
(0+45.3, 818.00)	(0+52.3, 820.00)	0.030

Results

Normal Depth	1.64	ft
Elevation Range	816.50 to 820.00 ft	
Flow Area	12.30	ft^2
Wetted Perimeter	44.57	ft
Top Width	43.93	ft
Normal Depth	1.64	ft
Critical Depth	1.92	ft
Critical Slope	0.00812	ft/ft
Velocity	8.45	ft/s

Worksheet for Cover Road Reach 2

Results

Velocity Head		1.11	ft
Specific Energy		2.75	ft
Froude Number		2.81	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.64	ft
Critical Depth	1.92	ft
Channel Slope	0.08000	ft/ft
Critical Slope	0.00812	ft/ft

Cross Section for Cover Road Reach 2

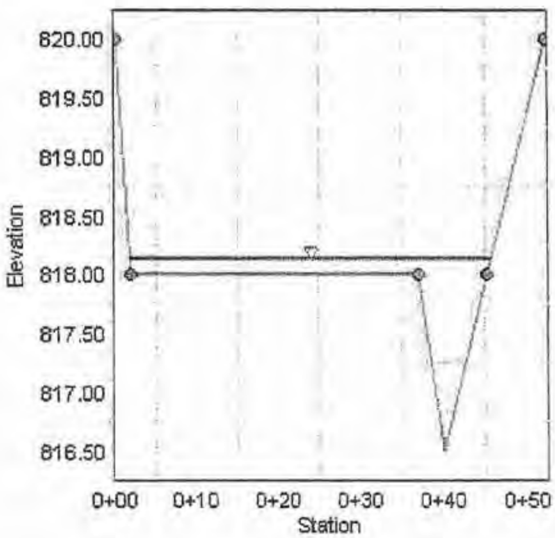
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.08000	ft/ft
Normal Depth	1.64	ft
Discharge	103.9	ft³/s

Cross Section Image



Worksheet for Perimeter Road Reach 3

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.06000	ft/ft
Discharge	42.3	ft^3/s

Section Definitions

Station (ft)	Elevation (ft)
0+00.0	820.00
0+08.0	816.00
0+10.0	817.00
0+35.0	816.50
0+38.0	814.50
0+40.0	814.50
0+43.0	816.50
0+50.0	818.50

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.0, 820.00)	(0+10.0, 817.00)	0.035
(0+10.0, 817.00)	(0+35.0, 816.50)	0.035
(0+35.0, 816.50)	(0+43.0, 816.50)	0.016
(0+43.0, 816.50)	(0+50.0, 818.50)	0.035

Results

Normal Depth	0.85	ft
Elevation Range	814.50 to 820.00 ft	
Flow Area	2.77	ft^2
Wetted Perimeter	5.06	ft
Top Width	4.54	ft
Normal Depth	0.85	ft
Critical Depth	1.66	ft

Worksheet for Perimeter Road Reach 3

Results

Critical Slope		0.00447	ft/ft
Velocity		15.25	ft/s
Velocity Head		3.61	ft
Specific Energy		4.46	ft
Froude Number		3.44	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.85	ft
Critical Depth	1.66	ft
Channel Slope	0.06000	ft/ft
Critical Slope	0.00447	ft/ft

Cross Section for Perimeter Road Reach 3

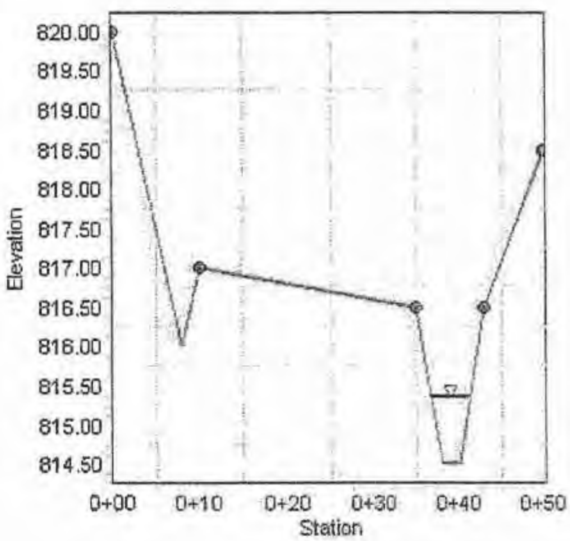
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.06000	ft/ft
Normal Depth	0.85	ft
Discharge	42.3	ft³/s

Cross Section Image



Worksheet for Perimeter Road Reach 4

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02600	ft/ft
Discharge	176.4	ft^3/s
Section Definitions		

Station (ft)	Elevation (ft)
0+00.0	820.00
0+08.0	816.00
0+10.0	817.00
0+35.0	816.50
0+38.0	814.50
0+40.0	814.50
0+43.0	816.50
0+50.0	818.50

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.0, 820.00)	(0+10.0, 817.00)	0.035
(0+10.0, 817.00)	(0+35.0, 816.50)	0.035
(0+35.0, 816.50)	(0+43.0, 816.50)	0.016
(0+43.0, 816.50)	(0+50.0, 818.50)	0.035

Results

Normal Depth		2.67	ft
Elevation Range	814.50 to 820.00 ft		
Flow Area		29.43	ft^2
Wetted Perimeter		41.52	ft
Top Width		39.70	ft
Normal Depth		2.67	ft
Critical Depth		2.78	ft

Worksheet for Perimeter Road Reach 4

Results

Critical Slope		0.01656	ft/ft
Velocity		5.99	ft/s
Velocity Head		0.56	ft
Specific Energy		3.23	ft
Froude Number		1.23	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	2.67	ft
Critical Depth	2.78	ft
Channel Slope	0.02600	ft/ft
Critical Slope	0.01656	ft/ft

Cross Section for Perimeter Road Reach 4

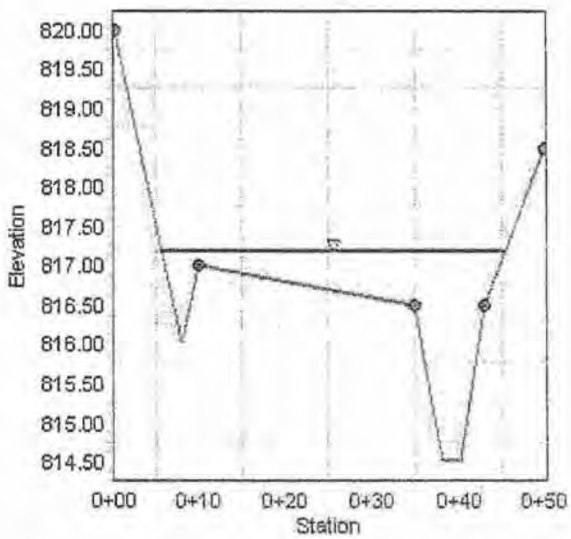
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.02600	ft/ft
Normal Depth	2.67	ft
Discharge	176.4	ft³/s

Cross Section Image



Worksheet for Perimeter Road Reach 5

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.06000	ft/ft
Discharge	183.6	ft³/s
Section Definitions		

Station (ft)	Elevation (ft)
0+00.0	820.00
0+08.0	816.00
0+10.0	817.00
0+35.0	816.50
0+38.0	814.50
0+40.0	814.50
0+43.0	816.50
0+50.0	818.50

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.0, 820.00)	(0+10.0, 817.00)	0.035
(0+10.0, 817.00)	(0+35.0, 816.50)	0.035
(0+35.0, 816.50)	(0+43.0, 816.50)	0.016
(0+43.0, 816.50)	(0+50.0, 818.50)	0.035

Results

Normal Depth		2.51	ft
Elevation Range	814.50 to 820.00 ft		
Flow Area		23.21	ft²
Wetted Perimeter		40.59	ft
Top Width		38.82	ft
Normal Depth		2.51	ft
Critical Depth		2.81	ft

Worksheet for Perimeter Road Reach 5

Results

Critical Slope		0.01635	ft/ft
Velocity		7.91	ft/s
Velocity Head		0.97	ft
Specific Energy		3.49	ft
Froude Number		1.80	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	2.51	ft
Critical Depth	2.81	ft
Channel Slope	0.06000	ft/ft
Critical Slope	0.01635	ft/ft

Cross Section for Perimeter Road Reach 5

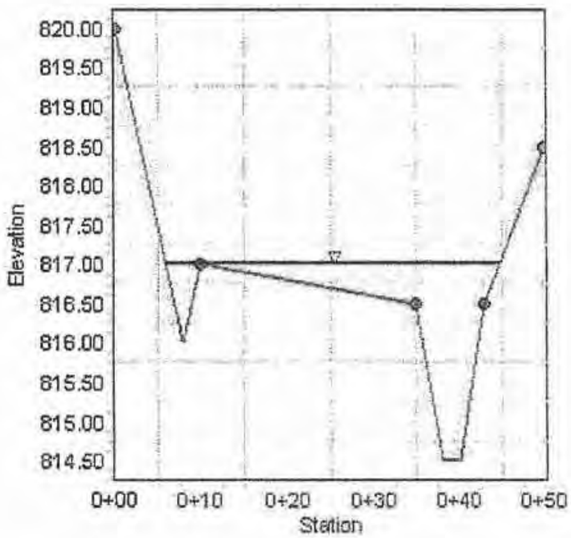
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.06000	ft/ft
Normal Depth	2.51	ft
Discharge	183.6	ft³/s

Cross Section Image



Worksheet for Bench Reach 6

Project Description

Friction Method	Manning Formula	
Solve For	Normal Depth	

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.02000	ft/ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	16.7	ft³/s ✓

Results

Normal Depth	0.41	ft
Flow Area	5.37	ft²
Wetted Perimeter	14.41	ft
Top Width	14.25	ft
Critical Depth	0.38	ft
Critical Slope	0.02562	ft/ft
Velocity	3.11	ft/s ✓
Velocity Head	0.15	ft
Specific Energy	0.56	ft
Froude Number	0.89	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.41	ft
Critical Depth	0.38	ft
Channel Slope	0.02000	ft/ft
Critical Slope	0.02562	ft/ft

Cross Section for Bench Reach 6

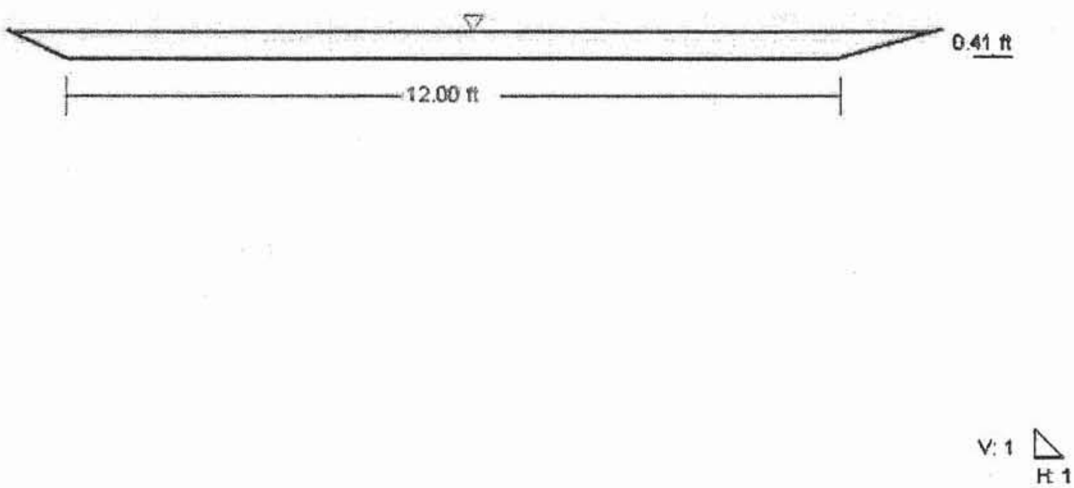
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.02000	ft/ft
Normal Depth	0.41	ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	16.7	ft³/s

Cross Section Image



Worksheet for Bench Reach 7

Project Description

Friction Method	Manning Formula	
Solve For	Normal Depth	

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.02000	ft/ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	26.3	ft³/s

Results

Normal Depth	0.53	ft
Flow Area	7.20	ft²
Wetted Perimeter	15.14	ft
Top Width	14.94	ft
Critical Depth	0.51	ft
Critical Slope	0.02351	ft/ft
Velocity	3.66	ft/s
Velocity Head	0.21	ft
Specific Energy	0.74	ft
Froude Number	0.93	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.53	ft
Critical Depth	0.51	ft
Channel Slope	0.02000	ft/ft
Critical Slope	0.02351	ft/ft

Cross Section for Bench Reach 7

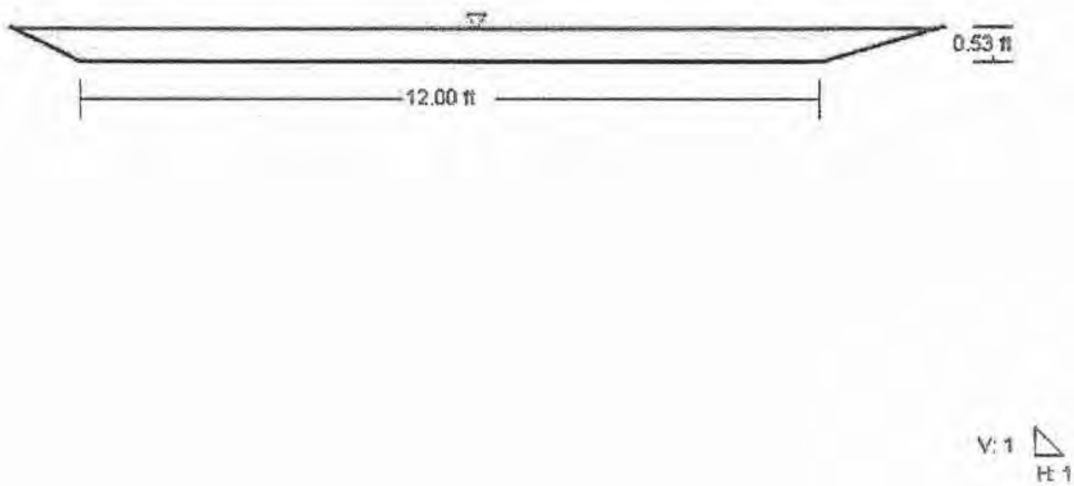
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.02000	ft/ft
Normal Depth	0.53	ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	26.3	ft³/s

Cross Section Image



Worksheet for Bench Reach 8

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.02000	ft/ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	50.4	ft³/s

Results

Normal Depth	0.78	ft
Flow Area	11.03	ft²
Wetted Perimeter	16.58	ft
Top Width	16.29	ft
Critical Depth	0.77	ft
Critical Slope	0.02089	ft/ft
Velocity	4.57	ft/s
Velocity Head	0.32	ft
Specific Energy	1.10	ft
Froude Number	0.98	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.78	ft
Critical Depth	0.77	ft
Channel Slope	0.02000	ft/ft
Critical Slope	0.02089	ft/ft

Cross Section for Bench Reach 8

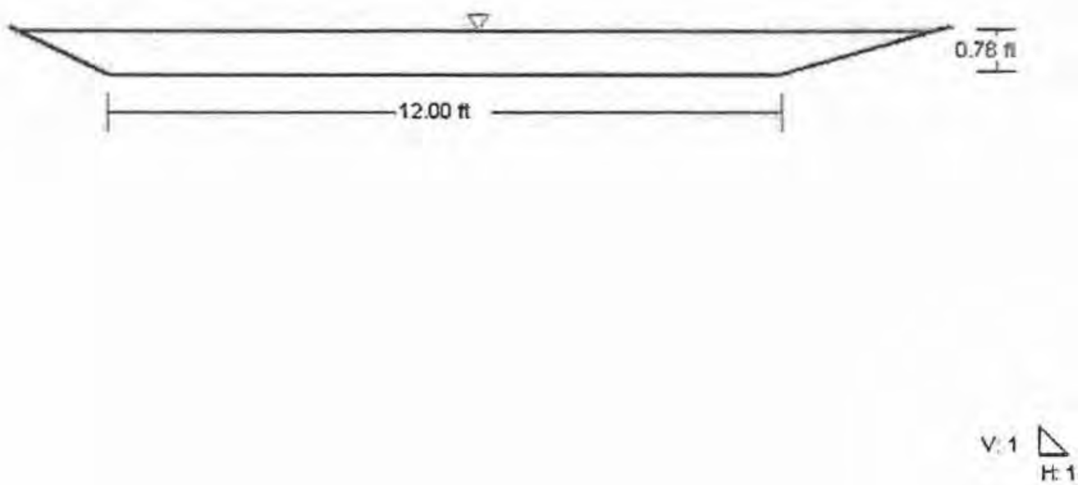
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.02000	ft/ft
Normal Depth	0.78	ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	50.4	ft³/s

Cross Section Image



Worksheet for Bench Reach 9

Project Description

Friction Method	Manning Formula	
Solve For	Normal Depth	

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.02000	ft/ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	27.6	ft^3/s

Results

Normal Depth	0.55	ft
Flow Area	7.42	ft^2
Wetted Perimeter	15.23	ft
Top Width	15.02	ft
Critical Depth	0.53	ft
Critical Slope	0.02329	ft/ft
Velocity	3.72	ft/s
Velocity Head	0.21	ft
Specific Energy	0.76	ft
Froude Number	0.93	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.55	ft
Critical Depth	0.53	ft
Channel Slope	0.02000	ft/ft
Critical Slope	0.02329	ft/ft

Cross Section for Bench Reach 9

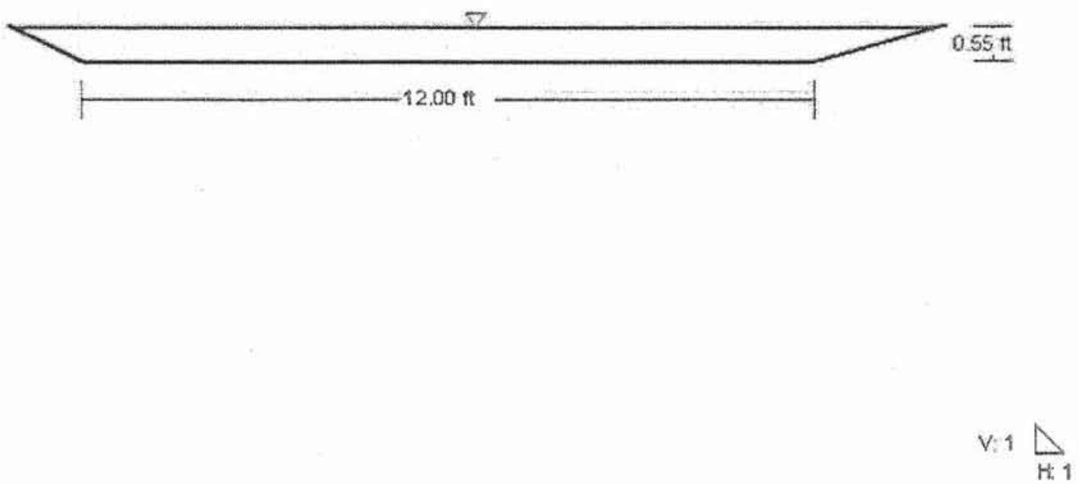
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.02000	ft/ft
Normal Depth	0.55	ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	27.6	ft³/s

Cross Section Image



Worksheet for Perimeter Road Reach 10

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.01400	ft/ft
Discharge	178.3	ft^3/s

Section Definitions

Station (ft)	Elevation (ft)
0+00.0	820.00
0+08.0	816.00
0+10.0	817.00
0+35.0	816.50
0+38.0	814.50
0+40.0	814.50
0+43.0	816.50
0+50.0	818.50

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.0, 820.00)	(0+10.0, 817.00)	0.035
(0+10.0, 817.00)	(0+35.0, 816.50)	0.035
(0+35.0, 816.50)	(0+43.0, 816.50)	0.016
(0+43.0, 816.50)	(0+50.0, 818.50)	0.035

Results

Normal Depth	2.84	ft
Elevation Range	814.50 to 820.00 ft	
Flow Area	36.05	ft^2
Wetted Perimeter	42.49	ft
Top Width	40.60	ft
Normal Depth	2.84	ft
Critical Depth	2.79	ft

Worksheet for Perimeter Road Reach 10

Results

Critical Slope		0.01660	ft/ft
Velocity		4.95	ft/s
Velocity Head		0.38	ft
Specific Energy		3.22	ft
Froude Number		0.93	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	2.84	ft
Critical Depth	2.79	ft
Channel Slope	0.01400	ft/ft
Critical Slope	0.01660	ft/ft

Cross Section for Perimeter Road Reach 10

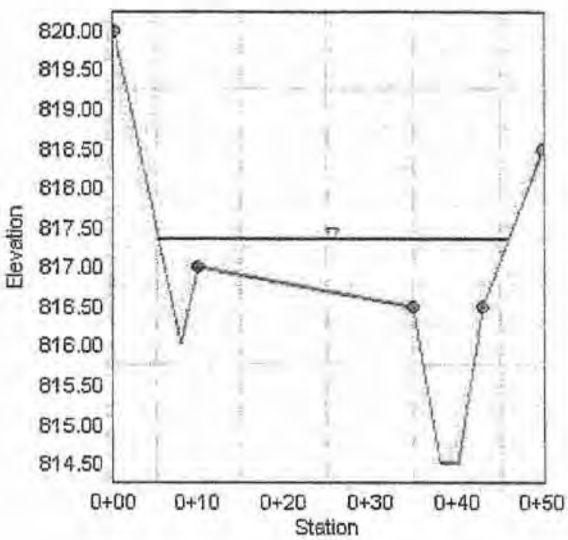
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.01400	ft/ft
Normal Depth	2.84	ft
Discharge	178.3	ft³/s

Cross Section Image



Worksheet for Perimeter Road Reach 11

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.04700	ft/ft
Discharge	269.1	ft³/s
Section Definitions		

Station (ft)	Elevation (ft)
0+00.0	820.00
0+08.0	816.00
0+10.0	817.00
0+35.0	816.50
0+38.0	814.50
0+40.0	814.50
0+43.0	816.50
0+50.0	818.50

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.0, 820.00)	(0+10.0, 817.00)	0.035
(0+10.0, 817.00)	(0+35.0, 816.50)	0.035
(0+35.0, 816.50)	(0+43.0, 816.50)	0.016
(0+43.0, 816.50)	(0+50.0, 818.50)	0.035

Results

Normal Depth		2.73	ft
Elevation Range	814.50 to 820.00 ft		
Flow Area		31.87	ft²
Wetted Perimeter		41.88	ft
Top Width		40.03	ft
Normal Depth		2.73	ft
Critical Depth		3.07	ft

Worksheet for Perimeter Road Reach 11

Results

Critical Slope		0.01525	ft/ft
Velocity		8.44	ft/s
Velocity Head		1.11	ft
Specific Energy		3.84	ft
Froude Number		1.67	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	2.73	ft
Critical Depth	3.07	ft
Channel Slope	0.04700	ft/ft
Critical Slope	0.01525	ft/ft

Cross Section for Perimeter Road Reach 11

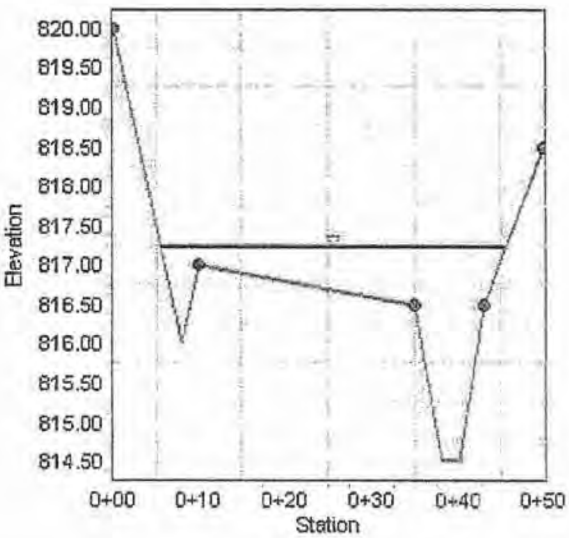
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.04700	ft/ft
Normal Depth	2.73	ft
Discharge	269.1	ft³/s

Cross Section Image



Worksheet for Bench Reach 12

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.02000	ft/ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	27.4	ft³/s

Results

Normal Depth	0.55	ft
Flow Area	7.39	ft²
Wetted Perimeter	15.21	ft
Top Width	15.01	ft
Critical Depth	0.52	ft
Critical Slope	0.02332	ft/ft
Velocity	3.71	ft/s
Velocity Head	0.21	ft
Specific Energy	0.76	ft
Froude Number	0.93	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.55	ft
Critical Depth	0.52	ft
Channel Slope	0.02000	ft/ft
Critical Slope	0.02332	ft/ft

Cross Section for Bench Reach 12

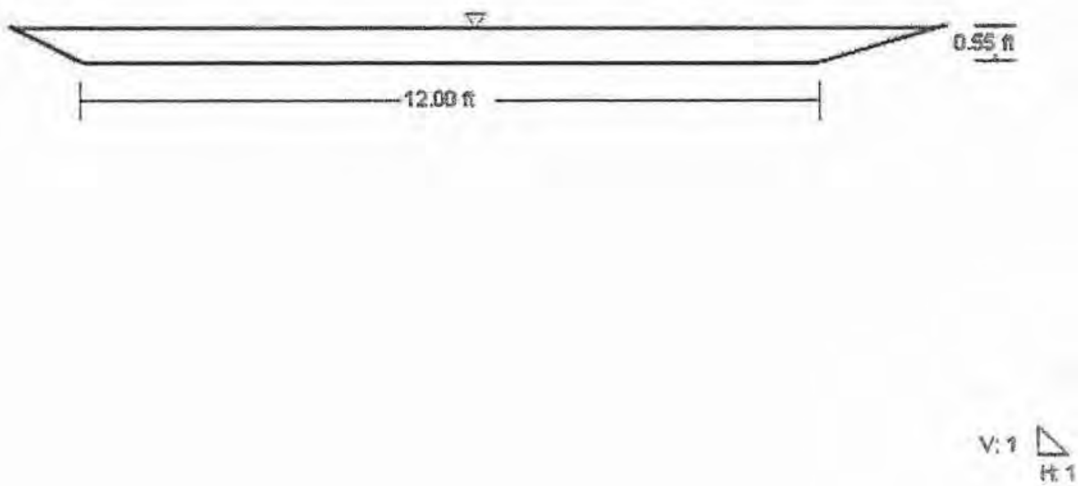
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.02000	ft/ft
Normal Depth	0.55	ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	27.4	ft³/s

Cross Section Image



Worksheet for Perimeter Road Reach 13

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.06700	ft/ft
Discharge	61.7	ft^3/s ✓

Section Definitions

Station (ft)	Elevation (ft)
0+00.0	820.00
0+08.0	816.00
0+10.0	817.00
0+35.0	816.50
0+38.0	814.50
0+40.0	814.50
0+43.0	816.50
0+50.0	818.50

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.0, 820.00)	(0+10.0, 817.00)	0.035
(0+10.0, 817.00)	(0+35.0, 816.50)	0.036
(0+35.0, 816.50)	(0+43.0, 816.50)	0.016
(0+43.0, 816.50)	(0+50.0, 818.50)	0.035

Results

Normal Depth	1.00	ft
Elevation Range	814.50 to 820.00 ft	
Flow Area	3.51	ft^2
Wetted Perimeter	5.61	ft
Top Width	5.01	ft
Normal Depth	1.00	ft
Critical Depth	2.20	ft

Worksheet for Perimeter Road Reach 13

Results

Critical Slope		0.00477	ft/ft
Velocity		17.58	ft/s
Velocity Head		4.80	ft
Specific Energy		5.80	ft
Froude Number		3.70	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.00	ft
Critical Depth	2.20	ft
Channel Slope	0.06700	ft/ft
Critical Slope	0.00477	ft/ft

Cross Section for Perimeter Road Reach 13

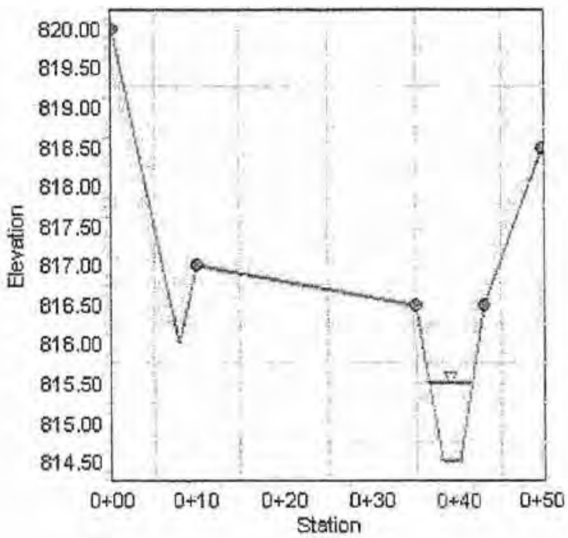
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.06700	ft/ft
Normal Depth	1.00	ft
Discharge	61.7	ft³/s

Cross Section Image



Worksheet for Bench Reach 14

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.02000	ft/ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	39.3	ft³/s

Results

Normal Depth	0.68	ft
Flow Area	9.35	ft²
Wetted Perimeter	15.97	ft
Top Width	15.71	ft
Critical Depth	0.66	ft
Critical Slope	0.02184	ft/ft
Velocity	4.20	ft/s
Velocity Head	0.27	ft
Specific Energy	0.95	ft
Froude Number	0.96	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.68	ft
Critical Depth	0.66	ft
Channel Slope	0.02000	ft/ft
Critical Slope	0.02184	ft/ft

Cross Section for Bench Reach 14

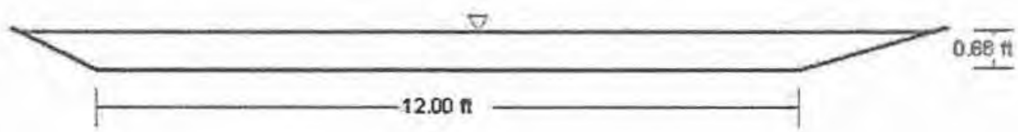
Project Description

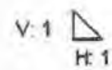
Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.02000	ft/ft
Normal Depth	0.68	ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	39.3	ft³/s

Cross Section Image





Worksheet for Perimeter Road Reach 15

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.08000	ft/ft
Discharge	133.2	ft³/s
Section Definitions		

Station (ft)	Elevation (ft)
0+00.0	820.00
0+08.0	816.00
0+10.0	817.00
0+35.0	816.50
0+38.0	814.50
0+40.0	814.50
0+43.0	816.50
0+50.0	818.50

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.0, 820.00)	(0+10.0, 817.00)	0.035
(0+10.0, 817.00)	(0+35.0, 816.50)	0.035
(0+35.0, 816.50)	(0+43.0, 816.50)	0.016
(0+43.0, 816.50)	(0+50.0, 818.50)	0.035

Results

Normal Depth	1.41	ft
Elevation Range	814.50 to 820.00 ft	
Flow Area	5.80	ft²
Wetted Perimeter	7.08	ft
Top Width	6.23	ft
Normal Depth	1.41	ft
Critical Depth	2.63	ft

Worksheet for Perimeter Road Reach 15

Results

Critical Slope		0.00444	ft/ft
Velocity		22.98	ft/s
Velocity Head		8.21	ft
Specific Energy		9.62	ft
Froude Number		4.20	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth		0.00	ft
Length		0.00	ft
Number Of Steps		0	

GVF Output Data

Upstream Depth		0.00	ft
Profile Description			
Profile Headloss		0.00	ft
Downstream Velocity		Infinity	ft/s
Upstream Velocity		Infinity	ft/s
Normal Depth		1.41	ft
Critical Depth		2.63	ft
Channel Slope		0.08000	ft/ft
Critical Slope		0.00444	ft/ft

Cross Section for Perimeter Road Reach 15

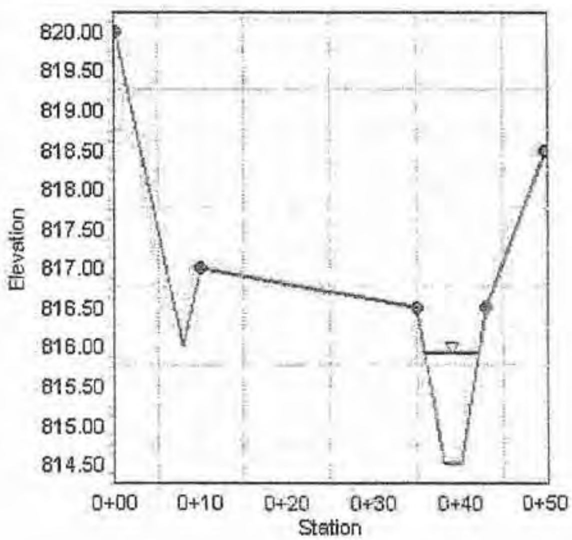
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.08000	ft/ft
Normal Depth	1.41	ft
Discharge	133.2	ft^3/s

Cross Section Image



Worksheet for Bench Reach 16

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.08000	ft/ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	36.7	ft³/s

Results

Normal Depth	0.43	ft
Flow Area	5.70	ft²
Wetted Perimeter	14.54	ft
Top Width	14.38	ft
Critical Depth	0.63	ft
Critical Slope	0.02211	ft/ft
Velocity	6.43	ft/s
Velocity Head	0.64	ft
Specific Energy	1.08	ft
Froude Number	1.80	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.43	ft
Critical Depth	0.63	ft
Channel Slope	0.08000	ft/ft
Critical Slope	0.02211	ft/ft

Cross Section for Bench Reach 16

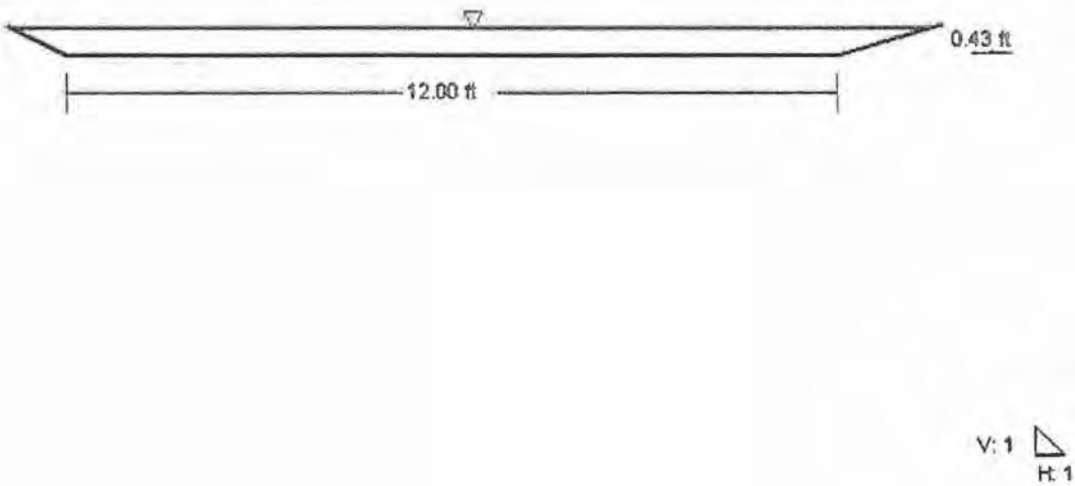
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.08000	ft/ft
Normal Depth	0.43	ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	3.50	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	36.7	ft^3/s

Cross Section Image



Worksheet for Perimeter Road Reach 17

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.05600	ft/ft
Discharge	85.7	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
0+00.0	820.00
0+08.0	816.00
0+10.0	817.00
0+35.0	816.50
0+38.0	814.50
0+40.0	814.50
0+43.0	816.50
0+50.0	818.50

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.0, 820.00)	(0+10.0, 817.00)	0.035
(0+10.0, 817.00)	(0+35.0, 816.50)	0.035
(0+35.0, 816.50)	(0+43.0, 816.50)	0.016
(0+43.0, 816.50)	(0+50.0, 818.50)	0.035

Results

Normal Depth	1.24	ft
Elevation Range	814.50 to 820.00 ft	
Flow Area	4.77	ft²
Wetted Perimeter	6.46	ft
Top Width	5.71	ft
Normal Depth	1.24	ft
Critical Depth	2.42	ft

Worksheet for Perimeter Road Reach 17

Results

Critical Slope		0.00478	ft/ft
Velocity		17.96	ft/s
Velocity Head		5.01	ft
Specific Energy		6.25	ft
Froude Number		3.46	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.24	ft
Critical Depth	2.42	ft
Channel Slope	0.05600	ft/ft
Critical Slope	0.00478	ft/ft

Cross Section for Perimeter Road Reach 17

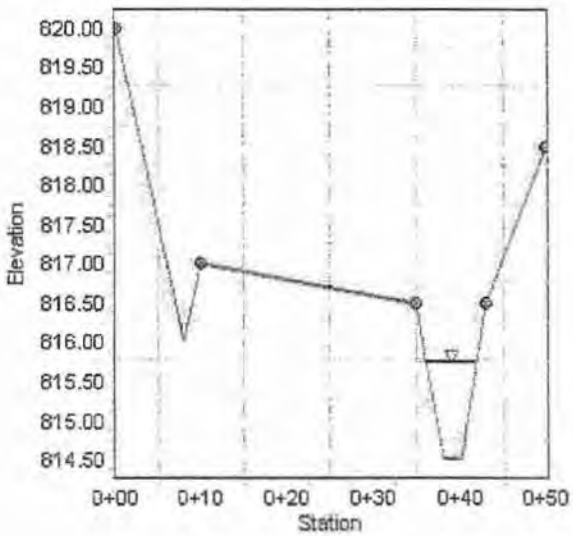
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

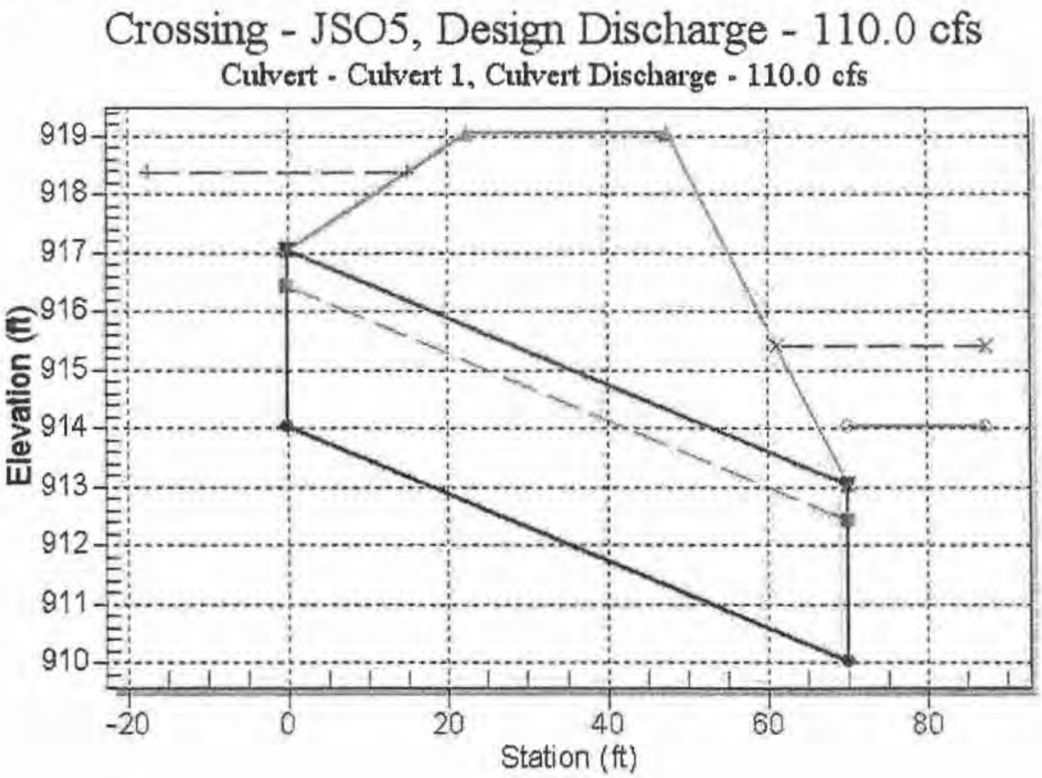
Channel Slope	0.05600	ft/ft
Normal Depth	1.24	ft
Discharge	85.7	ft³/s

Cross Section Image



HY-8 Culvert Analysis Report

Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 914.00 ft

Outlet Station: 70.00 ft

Outlet Elevation: 910.00 ft

Number of Barrels: 2

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 3.00 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: None

Table 1 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
110.00	110.00	918.32	4.320	3.173	4-FFf	1.159	2.405	1.159	1.365	21.793	19.665
110.00	110.00	918.32	4.320	3.173	4-FFf	1.159	2.405	1.159	1.365	21.793	19.665
110.00	110.00	918.32	4.320	3.173	4-FFf	1.159	2.405	1.159	1.365	21.793	19.665
110.00	110.00	918.32	4.320	3.173	4-FFf	1.159	2.405	1.159	1.365	21.793	19.665
110.00	110.00	918.32	4.320	3.173	4-FFf	1.159	2.405	1.159	1.365	21.793	19.665
110.00	110.00	918.32	4.320	3.173	4-FFf	1.159	2.405	1.159	1.365	21.793	19.665
110.00	110.00	918.32	4.320	3.173	4-FFf	1.159	2.405	1.159	1.365	21.793	19.665
110.00	110.00	918.32	4.320	3.173	4-FFf	1.159	2.405	1.159	1.365	21.793	19.665
110.00	110.00	918.32	4.320	3.173	4-FFf	1.159	2.405	1.159	1.365	21.793	19.665
110.00	110.00	918.32	4.320	3.173	4-FFf	1.159	2.405	1.159	1.365	21.793	19.665
110.00	110.00	918.32	4.320	3.173	4-FFf	1.159	2.405	1.159	1.365	21.793	19.665

****

Inlet Elevation (invert): 914.00 ft, Outlet Elevation (invert): 910.00 ft

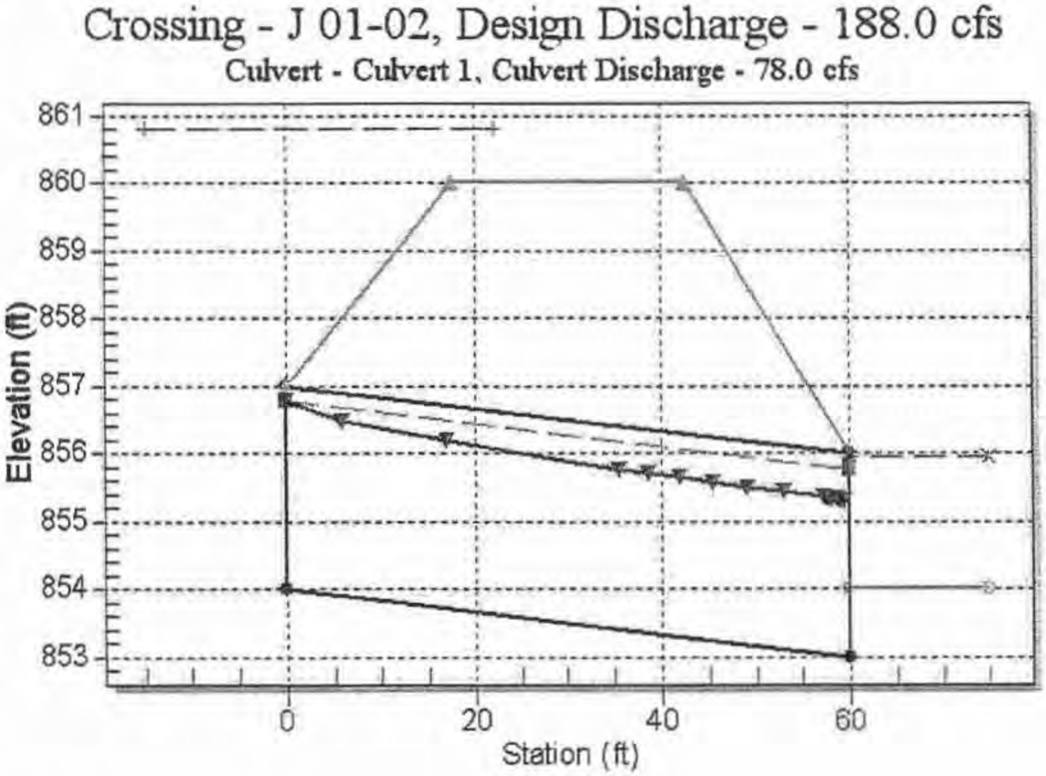
Culvert Length: 70.11 ft, Culvert Slope: 0.0571

****

Table 2 - Summary of Culvert Flows at Crossing: JSO5

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
918.32	110.00	110.00	0.00	1
918.32	110.00	110.00	0.00	1
918.32	110.00	110.00	0.00	1
918.32	110.00	110.00	0.00	1
918.32	110.00	110.00	0.00	1
918.32	110.00	110.00	0.00	1
918.32	110.00	110.00	0.00	1
918.32	110.00	110.00	0.00	1
918.32	110.00	110.00	0.00	1
918.32	110.00	110.00	0.00	1
918.32	110.00	110.00	0.00	1
919.00	124.86	124.86	0.00	Overtopping

Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 854.00 ft

Outlet Station: 60.00 ft

Outlet Elevation: 853.00 ft

Number of Barrels: 1

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 3.00 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: None

Table 3 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
188.00	78.04	860.81	6.807	6.807	5-S2n	2.096	2.775	2.310	1.959	13.387	12.185
188.00	78.04	860.81	6.807	6.807	5-S2n	2.096	2.775	2.310	1.959	13.387	12.185
188.00	78.04	860.81	6.807	6.807	5-S2n	2.096	2.775	2.310	1.959	13.387	12.185
188.00	78.04	860.81	6.807	6.807	5-S2n	2.096	2.775	2.310	1.959	13.387	12.185
188.00	78.04	860.81	6.807	6.807	5-S2n	2.096	2.775	2.310	1.959	13.387	12.185
188.00	78.04	860.81	6.807	6.807	5-S2n	2.096	2.775	2.310	1.959	13.387	12.185
188.00	78.04	860.81	6.807	6.807	5-S2n	2.096	2.775	2.310	1.959	13.387	12.185
188.00	78.04	860.81	6.807	6.807	5-S2n	2.096	2.775	2.310	1.959	13.387	12.185
188.00	78.04	860.81	6.807	6.807	5-S2n	2.096	2.775	2.310	1.959	13.387	12.185
188.00	78.04	860.81	6.807	6.807	5-S2n	2.096	2.775	2.310	1.959	13.387	12.185
188.00	78.04	860.81	6.807	6.807	5-S2n	2.096	2.775	2.310	1.959	13.387	12.185

****

Inlet Elevation (invert): 854.00 ft, Outlet Elevation (invert): 853.00 ft

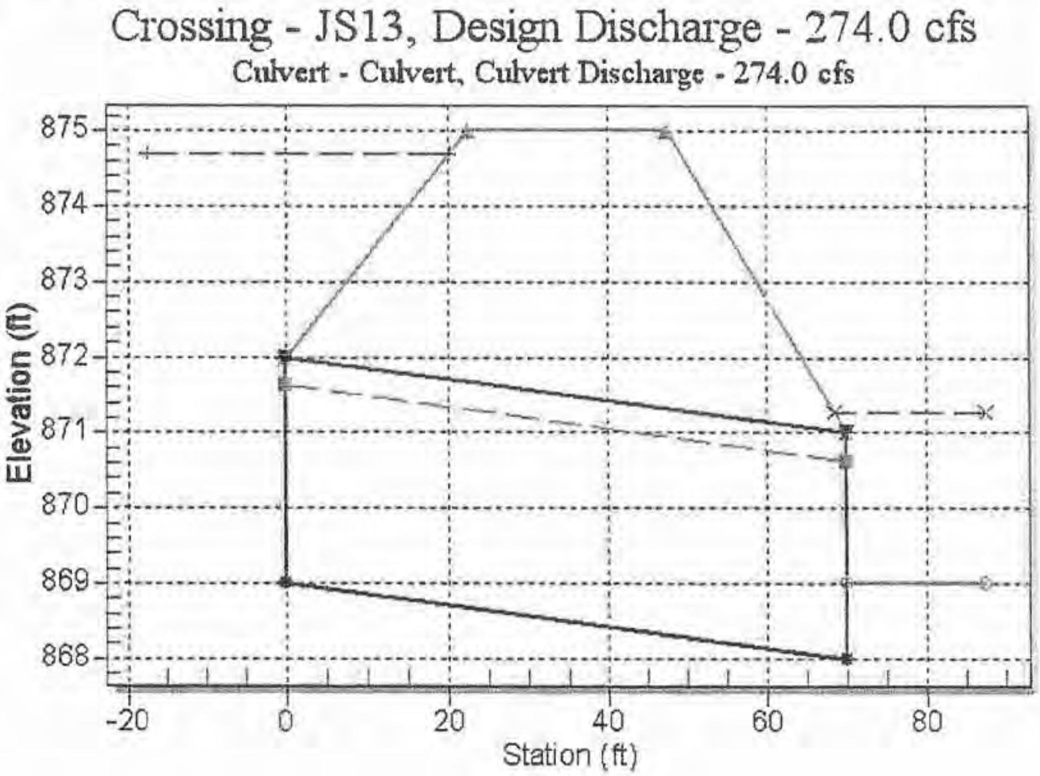
Culvert Length: 60.01 ft, Culvert Slope: 0.0167

****

Table 4 - Summary of Culvert Flows at Crossing: J 01-02

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
860.81	188.00	78.04	109.94	8
860.81	188.00	78.04	109.94	2
860.81	188.00	78.04	109.94	2
860.81	188.00	78.04	109.94	2
860.81	188.00	78.04	109.94	2
860.81	188.00	78.04	109.94	2
860.81	188.00	78.04	109.94	2
860.81	188.00	78.04	109.94	2
860.81	188.00	78.04	109.94	2
860.81	188.00	78.04	109.94	2
860.81	188.00	78.04	109.94	2
860.00	71.29	71.29	0.00	Overtopping

Water Surface Profile Plot for Culvert: Culvert



Site Data - Culvert

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 869.00 ft

Outlet Station: 70.00 ft

Outlet Elevation: 868.00 ft

Number of Barrels: 4

Culvert Data Summary - Culvert

Barrel Shape: Circular

Barrel Diameter: 3.00 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: None

Table 5 - Culvert Summary Table: Culvert

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
274.00	274.00	874.69	5.692	5.084	4-FFf	2.017	2.622	2.017	2.271	13.568	22.317
274.00	274.00	874.69	5.692	5.084	4-FFf	2.017	2.622	2.017	2.271	13.568	22.317
274.00	274.00	874.69	5.692	5.084	4-FFf	2.017	2.622	2.017	2.271	13.568	22.317
274.00	274.00	874.69	5.692	5.084	4-FFf	2.017	2.622	2.017	2.271	13.568	22.317
274.00	274.00	874.69	5.692	5.084	4-FFf	2.017	2.622	2.017	2.271	13.568	22.317
274.00	274.00	874.69	5.692	5.084	4-FFf	2.017	2.622	2.017	2.271	13.568	22.317
274.00	274.00	874.69	5.692	5.084	4-FFf	2.017	2.622	2.017	2.271	13.568	22.317
274.00	274.00	874.69	5.692	5.084	4-FFf	2.017	2.622	2.017	2.271	13.568	22.317
274.00	274.00	874.69	5.692	5.084	4-FFf	2.017	2.622	2.017	2.271	13.568	22.317
274.00	274.00	874.69	5.692	5.084	4-FFf	2.017	2.622	2.017	2.271	13.568	22.317
274.00	274.00	874.69	5.692	5.084	4-FFf	2.017	2.622	2.017	2.271	13.568	22.317

****

Inlet Elevation (invert): 869.00 ft,   Outlet Elevation (invert): 868.00 ft

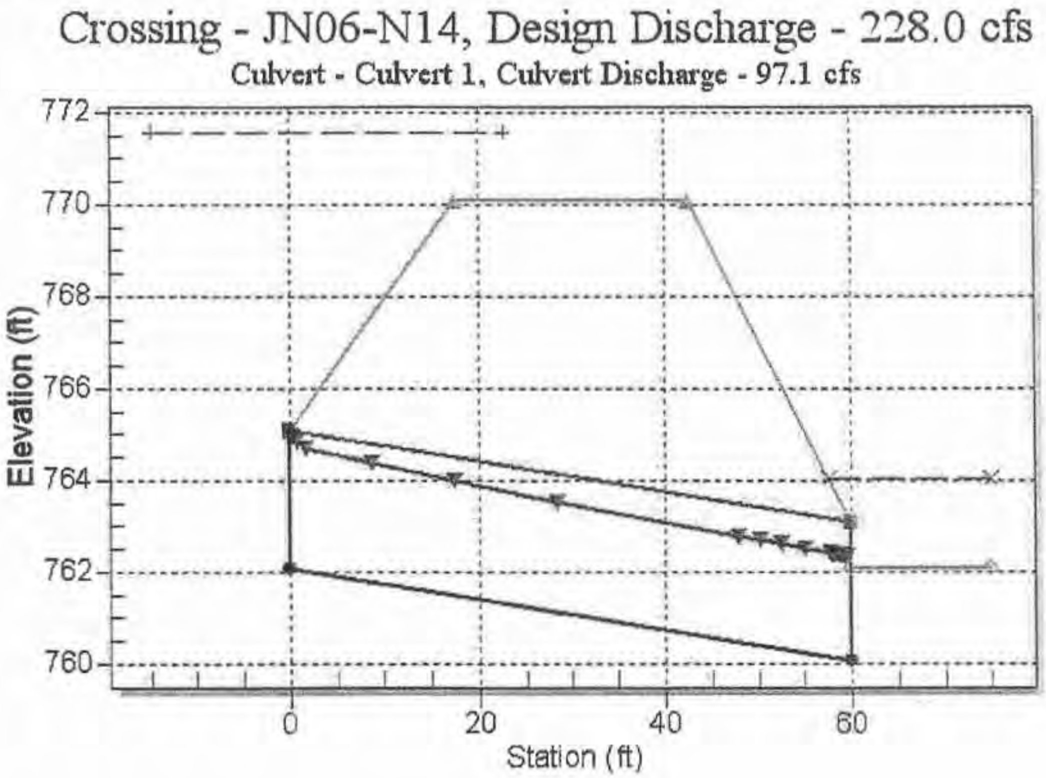
Culvert Length: 70.01 ft,   Culvert Slope: 0.0143

****

Table 6 - Summary of Culvert Flows at Crossing: JS13

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert Discharge (cfs)	Roadway Discharge (cfs)	Iterations
874.69	274.00	274.00	0.00	1
874.69	274.00	274.00	0.00	1
874.69	274.00	274.00	0.00	1
874.69	274.00	274.00	0.00	1
874.69	274.00	274.00	0.00	1
874.69	274.00	274.00	0.00	1
874.69	274.00	274.00	0.00	1
874.69	274.00	274.00	0.00	1
874.69	274.00	274.00	0.00	1
874.69	274.00	274.00	0.00	1
874.69	274.00	274.00	0.00	1
875.00	285.05	285.05	0.00	Overtopping

Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 762.00 ft

Outlet Station: 60.00 ft

Outlet Elevation: 760.00 ft

Number of Barrels: 1

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 3.00 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: None

Table 7 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
228.00	97.07	771.48	9.480	7.420	5-S2n	1.913	3.000	2.253	1.952	17.085	23.710
228.00	97.07	771.48	9.480	7.420	5-S2n	1.913	3.000	2.253	1.952	17.085	23.710
228.00	97.07	771.48	9.480	7.420	5-S2n	1.913	3.000	2.253	1.952	17.085	23.710
228.00	97.07	771.48	9.480	7.420	5-S2n	1.913	3.000	2.253	1.952	17.085	23.710
228.00	97.07	771.48	9.480	7.420	5-S2n	1.913	3.000	2.253	1.952	17.085	23.710
228.00	97.07	771.48	9.480	7.420	5-S2n	1.913	3.000	2.253	1.952	17.085	23.710
228.00	97.07	771.48	9.480	7.420	5-S2n	1.913	3.000	2.253	1.952	17.085	23.710
228.00	97.07	771.48	9.480	7.420	5-S2n	1.913	3.000	2.253	1.952	17.085	23.710
228.00	97.07	771.48	9.480	7.420	5-S2n	1.913	3.000	2.253	1.952	17.085	23.710
228.00	97.07	771.48	9.480	7.420	5-S2n	1.913	3.000	2.253	1.952	17.085	23.710
228.00	97.07	771.48	9.480	7.420	5-S2n	1.913	3.000	2.253	1.952	17.085	23.710

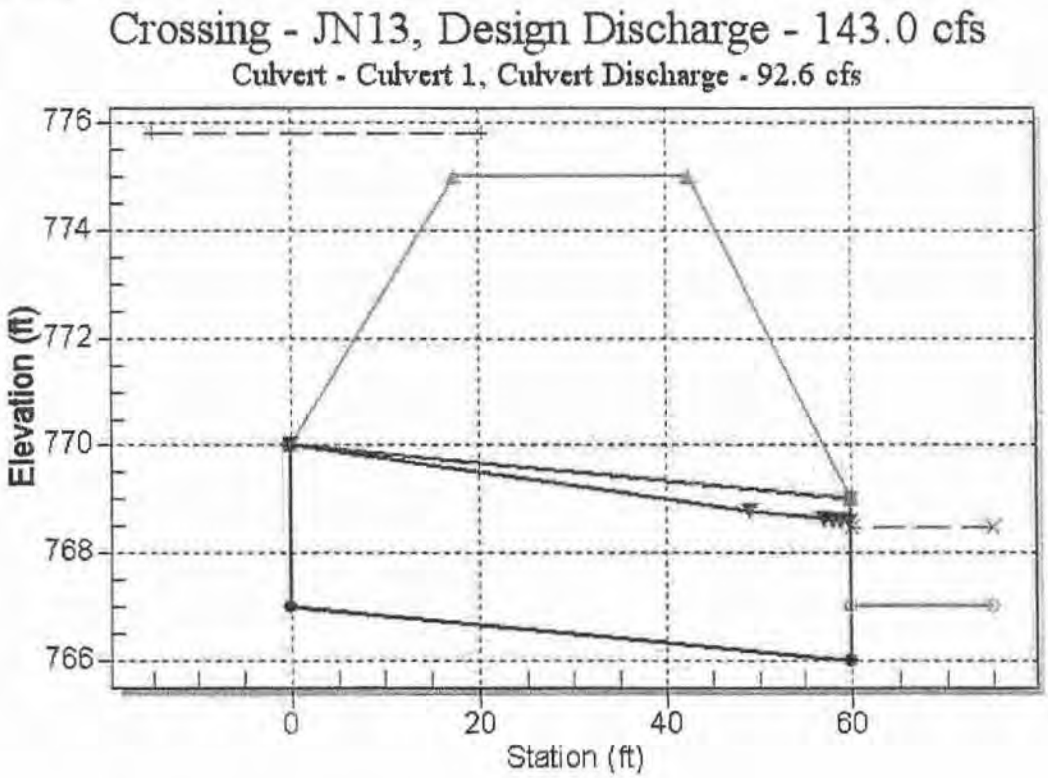
Inlet Elevation (invert): 762.00 ft, Outlet Elevation (invert): 760.00 ft

Culvert Length: 60.03 ft, Culvert Slope: 0.0333

Table 8 - Summary of Culvert Flows at Crossing: JN06-N14

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
771.48	228.00	97.07	130.89	11
771.48	228.00	97.07	130.89	2
771.48	228.00	97.07	130.89	2
771.48	228.00	97.07	130.89	2
771.48	228.00	97.07	130.89	2
771.48	228.00	97.07	130.89	2
771.48	228.00	97.07	130.89	2
771.48	228.00	97.07	130.89	2
771.48	228.00	97.07	130.89	2
771.48	228.00	97.07	130.89	2
771.48	228.00	97.07	130.89	2
770.00	87.19	87.19	0.00	Overtopping

Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 767.00 ft

Outlet Station: 60.00 ft

Outlet Elevation: 766.00 ft

Number of Barrels: 1

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 3.00 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: None

Table 9 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
143.00	92.62	775.81	8.807	6.979	5-S2n	2.446	3.000	2.587	1.458	14.337	23.424
143.00	92.62	775.81	8.807	6.979	5-S2n	2.446	3.000	2.587	1.458	14.337	23.424
143.00	92.62	775.81	8.807	6.979	5-S2n	2.446	3.000	2.587	1.458	14.337	23.424
143.00	92.62	775.81	8.807	6.979	5-S2n	2.446	3.000	2.587	1.458	14.337	23.424
143.00	92.62	775.81	8.807	6.979	5-S2n	2.446	3.000	2.587	1.458	14.337	23.424
143.00	92.62	775.81	8.807	6.979	5-S2n	2.446	3.000	2.587	1.458	14.337	23.424
143.00	92.62	775.81	8.807	6.979	5-S2n	2.446	3.000	2.587	1.458	14.337	23.424
143.00	92.62	775.81	8.807	6.979	5-S2n	2.446	3.000	2.587	1.458	14.337	23.424
143.00	92.62	775.81	8.807	6.979	5-S2n	2.446	3.000	2.587	1.458	14.337	23.424
143.00	92.62	775.81	8.807	6.979	5-S2n	2.446	3.000	2.587	1.458	14.337	23.424
143.00	92.62	775.81	8.807	6.979	5-S2n	2.446	3.000	2.587	1.458	14.337	23.424

Inlet Elevation (invert): 767.00 ft, Outlet Elevation (invert): 766.00 ft

Culvert Length: 60.01 ft, Culvert Slope: 0.0167

Table 10 - Summary of Culvert Flows at Crossing: JN13

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
775.81	143.00	92.62	50.35	12
775.81	143.00	92.62	50.35	2
775.81	143.00	92.62	50.35	2
775.81	143.00	92.62	50.35	2
775.81	143.00	92.62	50.35	2
775.81	143.00	92.62	50.35	2
775.81	143.00	92.62	50.35	2
775.81	143.00	92.62	50.35	2
775.81	143.00	92.62	50.35	2
775.81	143.00	92.62	50.35	2
775.81	143.00	92.62	50.35	2
775.00	87.02	87.02	0.00	Overtopping

APPENDIX K
LCRS ANALYSES

APPENDIX K.1	LCRS CAPACITY
APPENDIX K.2	GEOTEXTILE FILTER CAPACITY

APPENDIX K.1
LCRS CAPACITY

Golder Associates	Subject: Kettleman Hills Facility Landfill Unit B-18 LCRS Calculations	Made by: RJS	Job No.: 083-91887
		Checked by: RH	Date: 04/23/2008
		Reviewed by: SS	Sheet No.: 1 of 1

OBJECTIVE:

Evaluate if the existing Landfill B-18 Leachate Collection and Removal System (LCRS) will be sufficient to support the proposed Phase III expansion. Also confirm that the maximum head on the base liner will not exceed 12 inches at any point.

Compare capacity calculations with measured leachate volumes.

METHOD:

The original LCRS calculations performed by ESI (1990) assumed that 100% of the rainwater falling on B-18 would infiltrate into the LCRS. This assumption is very conservative and no longer valid as the current B-18 waste mass has significant absorptive capacity and given the climatic conditions much of the rainfall will either runoff and be collected or evaporate.

Perform calculations similar to the Phase I and II LCRS calculations to confirm the transmissivity of the LCRS geocomposite is adequate to convey the potential leachate (equal to rainfall volume) to the sump. Compute the capacity of the LCRS gravel around the sump to convey leachate and compare to potential maximum leachate volumes and historical recorded leachate generation rates.

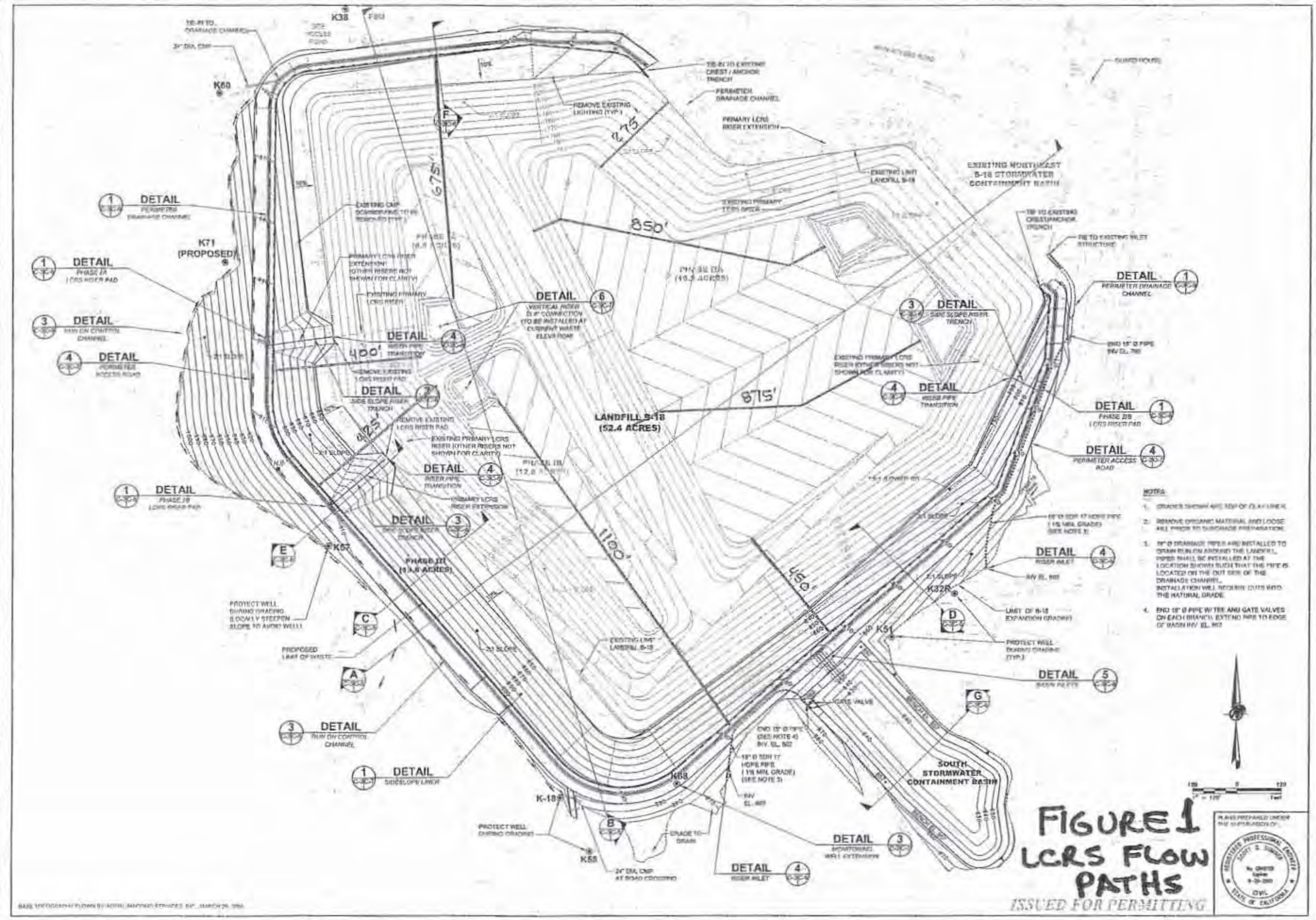
CALCULATIONS:

Following the calculation approach used by ESI (see Pages 3 to 5 in Attachment 1), confirm the increased slope length (i.e. greater capture area) is able to convey the annualized average leachate volume (assuming all rainfall becomes leachate). The flow length for the base considers the contributory flow from the upstream slope. Flow paths are shown on Figure 1 in Attachment 2. Based on the calculations the geocomposite is capable of conveying the annual rainfall to the sump.

Sump capacity was determined by Darcy's Law where the capacity (Q) is equal to the permeability (k) multiplied by the cross sectional area of the LCRS gravel (A) multiplied by the gradient of the floor (i). The capacity of the B-18 sump perimeter is approximately 9,000 gallons per day for each sump. Historic records (see Figure 2) indicate that the average flow is approximately 200 gallons per day maximum (Sump IB) with a maximum measured generation rate of approximately 6,000 gallons per day. The maximum flow rate resulted from exposing an area of geocomposite during a storm event allowing runoff to enter the LCRS system. The geocomposite is typically covered by protective geomembrane, operations layer and waste which limit the flow. Based on the observation the LCRS system is capable of conveying much larger volumes than typically encountered.

CONCLUSIONS/RESULTS:

The LCRS system is capable of conveying the expected leachate volumes. The geocomposite is shown to be capable of conveying the leachate to the sump without exceeding 12 inches of head on the liner. The original calculations are shown in Attachment 1, and the updated calculations are shown in Attachment 2.



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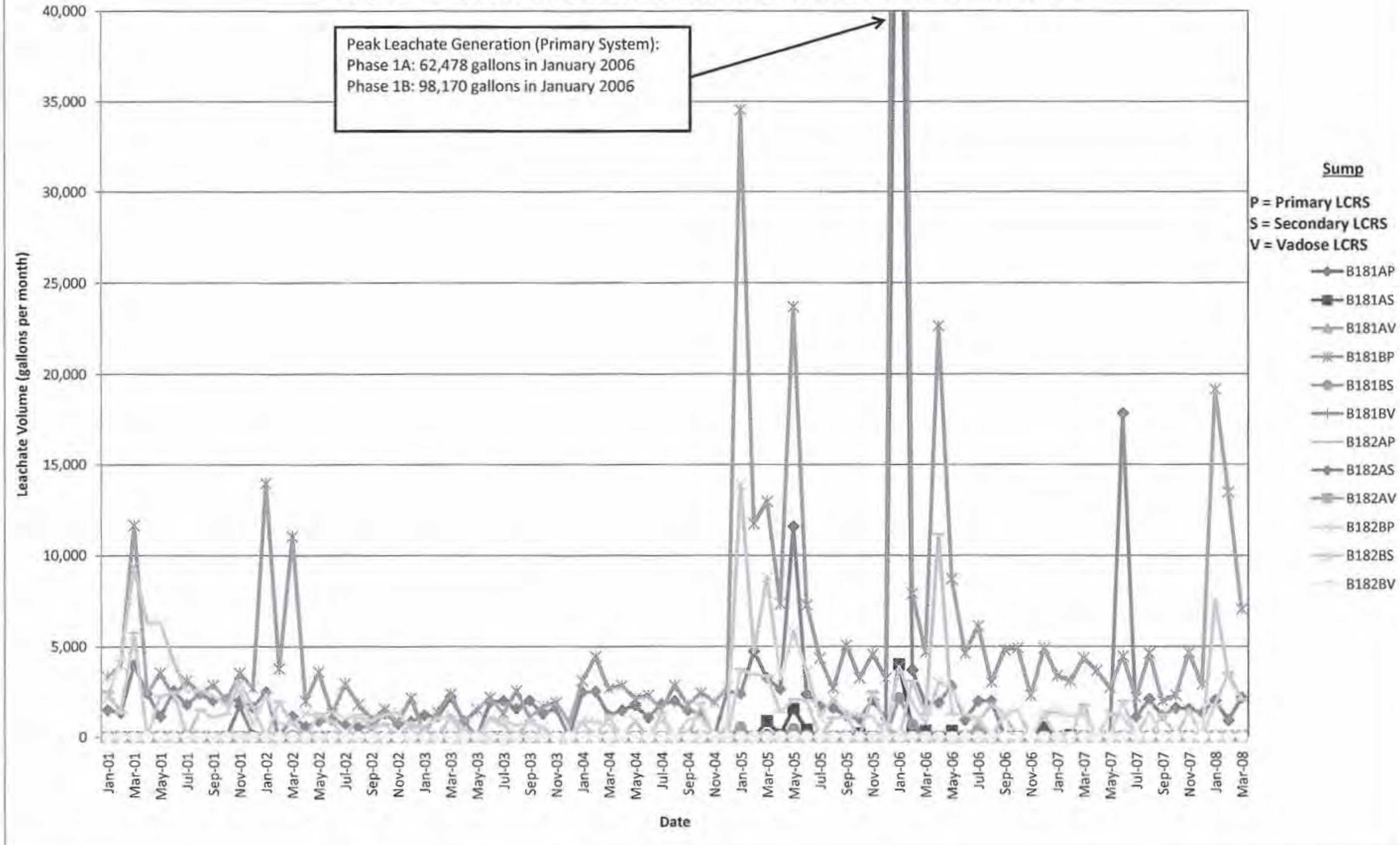
CHEMICAL WASTE MANAGEMENT
KETTLEMAN HILLS FACILITY
35251 OLD SKYLINE ROAD
KETTLEMAN CITY, CALIFORNIA 93239
(559) 386-9711

B-18 CLASS I LANDFILL
PHASE III EXPANSION AND FINAL CLOSURE
BASE LINER PLAN

Figure 2 - Historical Leachate Removal Rates from Landfill B-18



Attachment 1

Original LCRS Calculations

ENVIRONMENTAL SOLUTIONS, INC.

By DPL Date 8-13-90 Subject LANDFILL B-18 LCRS EVALUATION Sheet No. 1 of 22
Chkd. By GSC Date 8/13/90 Proj. No. 89-977

TABLE OF CONTENT

ENVIRONMENTAL SOLUTIONS, INC.

By Dpic Date 8-10-90 Subject LANDFILL B+B LCRS EVALUATION Sheet No. 2 of 33

Chkd. By GSC Date 8/13/90 Proj. No. 89-977

PURPOSE : TO EVALUATE THE CAPACITY OF THE LCR SYSTEM AND THE GEOTEXTILE DESIGN IN THE PRIMARY LCRS.

CRITERIA : THE LCR SYSTEM CAPACITY SHOULD BE SUFFICIENT TO HANDLE THE FLOW OF

1. LEACHATE GENERATED BY ANNUAL PRECIPITATION.
2. SURFACE WATER OF A 100-YR STORM IN 24-HOUR PERIOD.

REFERENCE :

1. GEOSYNTHETIC DESIGN GUIDANCE FOR HAZARDOUS WASTE LANDFILL CELL AND SURFACE IMPOUNDMENTS. EPA/600/2-87/097 BY SOIL & MATERIAL ENGINEERS INC. DEC. 1987
2. ENGINEERING REPORT, LANDFILL B-19, PHASE II & III BY DONOHUE & ASSOCIATES, INC. OCT 1988
3. SOIL MECHANICS LAMBE & WHITMAN
4. DESIGN WITH GEOSYNTHETICS KOERNER, 1986

ENVIRONMENTAL SOLUTIONS, INC.

By JPL Date 7-30-90 Subject LANDFILL B-18 . LCRS Sheet No. 3 of 23
Chkd. By GSC Date 8/13/90 EVALUATION Proj. No. 89-977

LCR SYSTEM CAPACITY FOR ANNUAL LEACHATE GENERATION

BASED ON KETTLEMAN STATION RECORD (EXHIBIT 1), THE AVERAGE ANNUAL PRECIPITATION RATE IS 6.48 IN. TO BE CONSERVATIVE, ASSUME 100 % OF THE RAINFALL WILL PERCOLATE THROUGH THE LANDFILL, THE LEACHATE GENERATION RATE IS THEREFORE

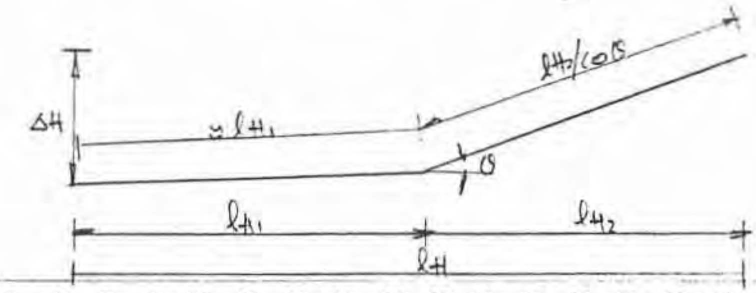
$$q = \frac{6.48}{365 \times 24 \times 60 \times 60 \times 12} = 1.71 \times 10^{-8} \text{ ft/sec}$$ ✓

ASSUME EACH LEACHATE SUMP WILL BE OPERATED INDEPENDENTLY, THE REQUIRED TRANSMISSIVITY OF A STRIP ALONG THE LONGEST FLOW PATH IN EACH CELL MAY BE ESTIMATED USING THE FOLLOWING FORMULA:

$$\boxed{\theta_{REQ} = \frac{q\,l}{i}}$$

WHERE

l = LENGTH OF LONGEST FLOW PATH
i = HYDRAULIC GRADIENT
θ_{REQ} = TRANSMISSIVITY



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Chkd. By SSC Date 8/13/90 EVALUATION Proj. No. 89.077

BASED ON THE ANNUAL LEACHATE GENERATION RATE, THE REQUIRED TRANSMISSIVITY FOR EACH CELL ARE CALCULATED AS FOLLOWS:

SUMP	l_H	ΔH	l	i	θ_{REQ} TRANSMISSIVITY (m^2/sec)
IA SLOPE	220	113	247	0.51	7.7×10^{-7}
IA BOTTOM	420	10	420	0.024	2.8×10^{-5}
IB SLOPE	230	92	248	0.4	1.0×10^{-6}
IB BOTTOM	800	20	800	0.025	5×10^{-5}
IIA SLOPE	90	44	100	0.49	3.2×10^{-7}
IIA BOTTOM	600	16	600	0.027	3.5×10^{-5}
IIB SLOPE	90	46	101	0.51	3.1×10^{-7}
IIB BOTTOM	710	16	710	0.023	4.9×10^{-5}

TEST RESULTS FOR A GEOCOMPOSITE (GEOTEXTILE/GEONET/GEOTEXTILE) PLACED BETWEEN TWO LAYERS OF SOILS ARE AS FOLLOW

i	θ_{TEST}	
0.25	5×10^{-5} m^2/sec	SEE EXHIBIT Z AT PRESSURE OF 7500 psf
0.5	4×10^{-5} m^2/sec	

BY COMPARING THE TEST RESULTS USING HIGHER GRADIENT FOR THE SLOPES AND LOWER GRADIENT FOR THE BOTTOMS OF THE LANDFILL WITH THE REQUIRED TRANSMISSIVITY,

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THE TEST RESULTS ARE MUCH HIGHER FOR THE SLOPE PORTION BUT ONLY SLIGHTLY HIGER FOR THE BOTTOM PORTION HOWEVER, THE ASSUMPTION OF 100 % OF THE PRECIPITATION WILL PERCOLATE THROUGH THE WASTE IS VERY CONSERVATIVE. FURTHERMORE, THE TRANSMISSIVITY FOR THE B-18 GEOCOMPOSITE SYSTEM IS EXPECTED TO BE HIGHER THAN THE REPORTED VALUES BECAUSE THE GEOCOMPOSITE SYSTEM FOR THE B-18 LANDFILL WILL BE PLACED BELOW A GRANUL.. LAYER AND ATOP A HDPE LAYER ; WHICH WILL BE LESS RESTRICTIVE FOR FLUID FLOW THAN THE SOIL LAYERS USED IN THE TEST. THEREFORE, THE CAPACITY OF THE LCRS IS CONSIDERED ADEQUATE.

(RESULTS ARE COMPARED TO TDEVIRA 1125)

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PER DARCY'S LAW, THE REQUIRED TRANSMISSIVITY ALONG THE PERIMETER OF THE LEACHATE SUMP MAY BE EXPRESSED BY

$$Q = k\,i\,A$$
$$= k\,i\,wt$$

$$\theta_{REQ} = \frac{Q}{w\,i}$$

WHERE w IS THE PERIMETER OF THE LEACHATE SUMP.

BASED ON THE AVERAGE ANNUAL PRECIPITATION OF 6.48 IN THE RUN-OFF VOLUME FOR EACH SUMP IS

SUMP	C ft/sec	RUN-OFF AREA (ACRES)	VOLUME (1) (cfs)
IA	1.71×10^{-8}	90	6.7×10^{-3}
IB	↓	125	9.2×10^{-3}
IIA	↓	16.9	1.3×10^{-2}
IIB	↓	15.1	1.1×10^{-2}

(1) VOLUME = $C\,i\,A$ WHERE $i = \frac{6.48}{365\times24} = 7.4\times10^{-4}$ IN/HR

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BASED ON THE ANNUAL LEACHATE GENERATION RATE, THE REQUIRED TRANSMISSIVITY FOR EACH SUMP IS CALCULATED AS FOLLOW:

PHASE	W (ft)	i	Q (cfs)	θ_{REQ} (m²/sec)	OVERBURDEN (psf)	θ_{TEST} (1) (m²/sec)
IA	550	0.02	6.7×10^{-3}	5.7×10^{-5}	21275	6.5×10^{-5}
IB	540	0.02	9.2×10^{-3}	8.2×10^{-6}	21275	6.5×10^{-5}
IIA	600	0.02	1.3×10^{-2}	1×10^{-4}	13225	1.4×10^{-4}
IIB	650	0.02	1.1×10^{-2}	7.9×10^{-5}	13225	1.4×10^{-4}

$\theta_{TEST} > \theta_{REQ}$ IN ALL C[illegible]ES

(1) EXHIBIT 2

BASED ON THE ABOVE CALCULATION, THE GEOCOMPOSITE CAPACITY ALONE IN PHASE IB SUMP WILL NOT BE SUFFICIENT TO HANDLE THE FLOW. THEREFORE, HYDRAULIC HEAD IS EXPECTED TO BUILD UP IN THE GRANULAR DRAINAGE LAYER. BASED ON MINIMUM TECHNOLOGY GUIDANCE, THE HYDRAULIC HEAD BUILD-UP ATOP THE LCRS CANNOT EXCEED 12" AT ALL TIME. ?

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Chkd. By GCC Date 8/13/90 EVALUATION Proj. No. 89 077

GRANULAR DRAINAGE LAYER

THE MAXIMUM HYDRAULIC HEAD IN THE DRAINAGE LAYER CAN BE ESTIMATED USING THE FORMULA:

$$h_{max} = \frac{L}{2n}\left[\sqrt{\frac{e}{K_S} + \tan \alpha^2} - \tan \alpha\right]$$ (REF. 1) EQ (3.6)

WHERE

L = EFFECTIVE FLOW LENGTH OF THE LCR

n = POROSITY OF DRAINAGE LAYER = 0.4 (REF 2)

e = INFILTRATION RATE

α = HYDRAULIC GRADIENT

K_S = DRAINAGE LAYER PERMEABILITY = 0.5 cm/sec (REF 2)
= 0.19 in/sec

BASED ON AVERAGE ANNUAL PRECIPITATION OF 6.48 IN, THE INFILTRATION RATE e IS

$$e = \frac{6.48}{365 \times 24 \times 3600} = 2.1 \times 10^{-7} \text{ in/sec}$$

$L = 110 \qquad d = 0.02$

$$\therefore \quad h = \frac{110 \times 12}{2 \times 0.4}\left[\sqrt{\frac{2.1 \times 10^{-7}}{0.19} + 0.02^2} - 0.02\right]$$

$$= 0.04'' \ll 12'' \quad \text{OK.}$$

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By ~~~~ Date 7-30-90 Subject LANDFILL B-18 LCRS EVALUATION Sheet No. 10 of 23

Chkd. By ~~~~ Date 8/13/90 Proj. No. 89-977

LCR SYSTEM CAPACITY FOR SURFACE WATER FLOW

BASED ON KETTLEMAN STATION RECORD, THE 24-HOUR RAINFALL OF A 100-YEAR STORM IS 2.34 IN (EXHIBIT 1).

THE SURFACE WATER RUN-OFF IN EACH CELL IS CALCULATED AS FOLLOW:

SUMP	RUNOFF AREA (ACRES)	VOLUME(1) (cfs)
I A	9.0	0.88
I B	12.5	1.22
II A	16.9	1.65
II B	15.1	1.47

(1) VOLUME = CiA $\qquad i = \frac{2.34}{24} = 0.09$ IN/HR

BASED ON THE 24-HR 100-YR STORM RUN-OFF VOLUME, THE REQUIRED TRANSMISSIVITY FOR EACH SUMP IS CALCULATED AS FOLLOW:

PHASE	W (ft)	i	Q (cfs)	θ_{REQ} (m²/sec)	OVERBURDEN (psf)	θ_{TEST}(1) (m²/sec)
I A	540	0.02	0.88	7.6×10^{-3}	21275	6.5×10^{-5}
I B	520	0.02	1.22	1.1×10^{-2}	21275	6.5×10^{-5}
II A	600	0.02	1.65	1.3×10^{-2}	13225	1.4×10^{-4}
II B	650	0.02	1.47	1.1×10^{-2}	13225	1.4×10^{-4}

(1) EXHIBIT 2

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By DPL Date 8-10-90 Subject LANDFILL B-18 Sheet No. 11 of 73

Chkd. By ESC Date 8/13/90 LCPS EVALUATION Proj. No. 89-977

BASED ON THE ABOVE CALCULATION, THE GEOCOMPOSITE ALONE IS NOT CAPABLE TO HANDLE THE SURFACE WATER FROM A 100-YEAR STORM. THEREFORE, HYDRAULIC HEAD IS EXPECTED TO BUILD UP IN THE LEACHATE SUMP

THE INFILTRATION RATE FOR A 24-HR 100-YR STORM IS

$$e = \frac{2.34}{24 \times 3600} = 2.71 \times 10^{-5} \text{ IN/SEC}$$

ENVIRONMENTAL SOLUTIONS, INC.

By 7Pi Date 7-30-90 Subject LANDFILL B-18, LCRS Sheet No. 12 of 23
Chkd. By GSC Date 8/15/90 EVALUATION Proj. No. 89 977

BASED ON A 100-YR STORM IN A 24-HOUR PERIOD, THE HYDRAULIC HEAD BUILT-UP IN EACH SOU IS CALCULATED AS FOLLOW =

PHASE	L (ft)	i	$h_{(MAX)}$ (IN)
I A	100	0.02	4.9
I B	110	0.02	5.4
II A	100	0.02	4.9
II B	130	0.02	6.4

(1) L = EF

THE HYDRAULIC HEAD BUILT UP IN THE GRANULAR LAYER WILL BE LESS THAN 12". THEREFORE, THE HYDRAULIC HEAD BUILT-UP WILL BE LESS THAN THE MTG (MINIMUM TECNOLOGY GUIDANCE) CRITERIA OF NO MORE THAN 1 FOOT OF LEACHATE ACTING ON THE LINER AT ALL TIME.

Attachment 2

Updated LCRS Calculations

LCRS Calculations

by:	RJS
Checked:	RH
Reviewed:	SS

Avg. Annual Rainfall = 6.48 in

Average Leachate gen. Rate = 1.71E-08 ft/sec/ft^2

Area	ΔL	i	Required Transmissivity (m^2/sec)	Geocomposite transmisivity (m2/sec)	Drainage layer Transmissivity (m2/sec)	Total Transmisivity (m2/sec)	Required Capacity met?
IA Slope	400	0.411	1.55E-06	4.00E-05		4.00E-05	yes
IA Bottom	675	0.024	4.51E-05	5.00E-05	3.05E-05	8.05E-05	yes
IB Slope	425	0.363	1.86E-06	4.00E-05		4.00E-05	yes
IB Bottom	1100	0.026	6.78E-05	5.00E-05	3.05E-05	8.05E-05	yes
IIA Slope	275	0.354	1.24E-06	4.00E-05		4.00E-05	yes
IIA Bottom	850	0.026	5.18E-05	5.00E-05	3.05E-05	8.05E-05	yes
IIB Slope	450	0.406	1.76E-06	4.00E-05		4.00E-05	yes
IIB Bottom	875	0.028	5.06E-05	5.00E-05	3.05E-05	8.05E-05	yes

Adequacy of system along perimeter of leachate sump

Sump	Leachate generation	Run-off Area (acres)	w (ft)	i	Potential Leachate Volume (cfs)	Drainage Layer Hydraulic Conductivity (in^2/sec)	Sump Capacity (cfs) Q = kiA	Sump Capacity (gallons/day)	Adequate Sump Capacity
IA	1.71E-08	12.9	550	0.02	9.6E-03	0.19	1.5E-02	9,380	yes
IB	1.71E-08	19.7	540	0.02	1.5E-02	0.19	1.4E-02	9,209	no
IIA	1.71E-08	17.8	600	0.02	1.3E-02	0.19	1.6E-02	10,233	yes
IIB	1.71E-08	17.5	650	0.02	1.3E-02	0.19	1.7E-02	11,085	yes

Leachate generation assumes all rainfall will be collected as leachate.

Note: the leachate flow capacity for each sump is in excess of 9,000 gallons per day. Based on historic measurements the maximum leachate generation rate has been approximatley 6,000 gallons per day. This peak generation was measured during a one month period and can be attributed to an exposure of the geocomposite. Leachate generation has typically been less than 200 gallons per day, on average (January 2001 to December 2007). Given the dry nature of the facility and that the existing waste is below field capacity, the leachate generation rate is expected to remain in the 200 to 300 gallons per day per sump (maximum). The construction of Phase III is not expected to result in significantly larger volumes of leachate.

APPENDIX K.2
GEOTEXTILE FILTER CAPACITY

ENVIRONMENTAL SOLUTIONS, INC.

By Vpi Date 8-10-90 Subject LANDFILL B-18 LCRS EVALUATION Sheet No. 13 of 23

Chkd. By GSC Date 8/13/90 Proj. No. 89-971

FILTER - RETENTION CAPACITY

THE FILTER-RETENTION CAPACITY COMPUTATION IS NOT NECESSARY FOR THE GEOTEXTILE PLACED IN THE SECONDARY LCRS BECAUSE INSUFFICIENT WATER FLOW IS EXPECTED IN THE LCRS TO CAUSE SOIL PARTICLES MOVEMENT. FOR THE PRIMARY LCRS, A LAYER OF GEOTEXTILE IS PLACED BETWEEN THE OPERATION LAYER AND THE DRAINAGE LAYER; SINCE THE OPERATION LAYER MATERIAL WILL BE NON-COHESIVE, IT IS ASSUMED THAT THE GRAIN SIZE CHARACTERISTICS ARE SIMILAR TO THE ON-SITE SANDSTONE MATERIALS

THE CALCULATION IS NOT REQUIRED FOR THE SLOPE PORTION OF THE LANDFILL BECAUSE THE POTENTIAL FOR HYDRAULIC HEAD BUILT-UP ON SLOPE IS UNLIKELY.

CRITERION 1 - RETENTION CAPACITY

FOR SOIL LESS THAN 50 % PASSING #200 SIEVE
AOS OF FABRIC > NO 30 SIEVE
(SEE EXHIBIT 3 - SANDSTONE GRAIN SIZE CHARACTERISTIC)

		SIEVE #		
AOS OF FABRIC	(TREVIRA 1155) :	120 - 170	> #30	OK
	(TREVIRA 1120) :	70 - 100	> 30	OK

REF = EXHIBIT 4

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Chkd. By GCC Date 8/15/9. Proj. No. 89-977

CRITERION 2 - FLOWING THROUGH & CLOGGING

<i> $\frac{O_{95}}{d_{85}} < 2$ FLOWING THROUGH CRITERION (REF 1 EQ 3.11)

<ii> $\frac{O_{95}}{d_{15}} > 2$ CLOGGING CRITERION (REF 1 EQ. 3.12)

FOR THE SANDSTONE MATERIALS, (EXHIBIT 3), THE AVERAGE d_{85} & d_{15} IS 0.46 mm AND 0.075 OR LESS, RESPECTIVELY; AND O_{95} FOR TREVIRA 1155 RANGES FROM 0.125 ~ 0.088 (EXHIBIT ~~4~~) 5 MP

THEREFORE

$$\frac{O_{95}}{d_{85}} = \frac{0.125}{0.46} = 0.27 < 2 \quad OK$$

$$\frac{O_{95}}{d_{15}} = \frac{0.088}{<0.075} = 1.17 \text{ OR GREATER}$$ (SHOULD BE > 2)

FOR TREVIRA 1125 O_{95} RANGES FROM 0.21 TO 0.149 (EXHIBIT ~~4~~) 5 MP

THEREFORE

$$\frac{O_{95}}{d_{85}} = \frac{0.21}{0.46} = 0.46 < 2 \quad OK$$

$$\frac{O_{95}}{d_{15}} = \frac{0.149}{0.06} = 2.5 > 2 \quad OK$$

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CRITERION 3. - EXCESSIVE LOSS OF FINES

ASSUME THE SANDSTONE MATERIAL WILL BECOME DENSE ($D_r > 80\%$) AFTER WATER PERCOLATING THROUGH THE LAYER (JETTING EFFECT).

COEFFICIENT OF UNIFORMITY OF THE MATERIAL

$$CU: \frac{d_{60}}{d_{10}} > 3 \qquad \begin{array}{l} d_{60} = 0.25 \sim 0.28\,mm \\ d_{10} \approx < 0.06 \end{array} \qquad C_U = \frac{0.28}{0.06} = 4.7$$

THEREFORE :

TREVIRA 1155 TREVIRA 1125

$$\frac{18 d_{50}}{CU} = \frac{18 \times 0.2}{4.7} = 0.77 > O_{95} \; (0.125 \sim 0.056),\ (0.21 - 0.149) \quad OK$$

USING A MORE RESTRICTIVE CRITERION BY ASSUMING THE SANDSTONE IS LOOSE, THUS

$$\frac{9 d_{50}}{CU} = \frac{9 \times 0.21}{4.7} = 0.38 > O_{95} \qquad OK$$

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PERMEABILITY

THE PERMEABILITY OF THE GEOTEXTILE SHOULD BE GREATER THAN THE PERMEABILITY OF THE OPERATION LAYER BASED ON GRAIN SIZE CHARACTERISTICS OF THE OPERATION LAYER MATERIAL, THE PERMEABILITY OF THE MATERIAL MAY BE ESTIMATED AS:

$k_{SOIL} = C\,D_{10}^2$ $\qquad C = 100$ (REF 3 EQ 19.9)

$= 100 \times 0.006^2$ $\qquad D_{10} < 0.006$ cm EXHIBIT 3
$= 0.0036$ cm/sec

$\therefore\ k_{soil} < 0.0036$ cm/sec $< k_{fabric}$ (TREVIRA 1125 = 0.59 cm/sec)

PUNCTURE RESISTANCE

BASED ON REFERENCE 4, THE TENSILE FORCE IN THE GEOTEXTILE T MAY BE ESTIMATED AS

$$T = \pi\,(d_i\ d_a)\,p'\,s'$$

WHERE

d_i : INITIAL AVERAGE VOID DIAMETER OF THE GEOTEXTILE. = 0.21 mm = 0.008 in (TREVIRA 1120) (EXHIBIT

(EXHIBIT 3)

d_a = AVERAGE DIAMETER OF THE MATERIAL ; USE d_{50} = 0.2 mm
= 0.008 in

ENVIRONMENTAL SOLUTIONS, INC.

By WPL Date 8-13-90 Subject LANDFILL B-18 Sheet No. 17 of 23

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P' = OVERBURDERN PRESSURE = 210 X 115 = 168 psi

S' = SHAPE FACTOR USE 1 FOR SHARP OBJECT (CONSERVATIVE)

$\therefore$ T = π X 0.008 X 0.008 X 168 X 1 = 0.03 lb

PUNCTURE STRENGTH FOR TREVIRA 1120 = 100 lb $\gg$ 0.03 lb O.K

CONSIDER THE PUNCTURE POTENTIAL FROM THE GRANULAR DRAINAGE LAYER

S' = 0.6 CRUSHED STONE (REF 2)

d_a = 0.25 CRUSHED STONE (REF 2)

$\therefore$ T = π X 0.008 X 0.25 X 168 X 0.6 = 0.6 lb $\ll$ 100 lb

RETENTION CAPACITY AND POTENTIAL FOR CLOGGING AND EXCESSIVE LOSS OF FINES ARE NO CONCERN FOR THE GEOTEXTILE UNDER THE GRANULAR DRAINAGE LAYER BECAUSE OF LACK OF FINES IN THE GRADATION OF THE GRANULAR MATERIALS (SEE SPECIFICATION 2.03 , LESS THAN 5 % OF THE MATERIAL WILL PASS # 100 SIEVE)

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By DPK Date 8-13-90 Subject LANDFILL B-18 Sheet No. 18 of 23
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CONCLUSION

THE CAPACITY OF THE LCRS AND THE GEOTEXTILE DESIGN IN THE PRIMARY LCRS HAVE BEEN EVALUATED. BASED ON THE CALCULATION PERFORMED, THE LCRS CAPACITY IS CONSIDERED ADEQUATE TO HANDLE LEACHATE FLOW THAT MAY GENERATED DURING LANDFILL OPERATION. THE CALCULATION ALSO INDICATED THAT THE GEOTEXTILE (TREVIRA 1125) USED IN THE PRIMARY LCRS HAS MET THE CRITERIA FOR RETENTION, CLOGGING, AND PREVENTION OF EXCESSIVE LOSS OF FINES.

19/27

EXHIBIT 1

PRECIPITATION DEPTH-DURATION-FREQUENCY TABLE

STATION NO. BSN ORDER SUB	STATION NAME	ELEV	SEC	TWP	RNG LOT	BWM	LATITUDE	LONGITUDE	COUNTY CODE
C00 4536 00	KETTLEMAN STATION	508	25	21S	17E L	M	36.075	120.085	16

MAXIMUM PRECIPITATION FOR INDICATED DURATION D-DAYS H-HOURS

RETURN PERIOD IN YEARS	1D	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	365D
2	.98	1.21	1.31	1.38	1.44	1.50	1.60	1.76	2.00	2.14	2.58	3.49	6.08
5	1.35	1.71	1.89	2.00	2.09	2.18	2.33	2.53	2.93	3.10	3.65	4.98	8.35
10	1.59	2.05	2.28	2.42	2.53	2.64	2.83	3.04	3.56	3.74	4.34	5.96	9.79
20	1.83	2.38	2.65	2.82	2.95	3.08	3.30	3.52	4.17	4.35	4.98	6.88	11.12
25	1.90	2.48	2.77	2.94	3.08	3.21	3.45	3.67	4.36	4.54	5.17	7.16	11.53
40	2.05	2.70	3.01	3.20	3.36	3.50	3.76	3.99	4.75	4.93	5.27	7.75	12.37
50	2.12	2.80	3.13	3.33	3.48	3.64	3.90	4.14	4.94	5.12	5.77	8.03	12.76
100	2.34	3.10	3.48	3.70	3.88	4.05	4.34	4.59	5.50	5.68	6.35	8.87	13.95
200	2.55	3.41	3.82	4.07	4.27	4.45	4.78	5.03	6.06	6.23	6.91	9.69	15.10
1000	3.03	4.09	4.60	4.91	5.15	5.37	5.78	6.03	7.33	7.49	8.18	11.54	17.67
10000	3.71	5.05	5.70	6.09	6.38	6.66	7.17	7.42	9.10	9.23	9.92	14.11	21.19
PMP	7.04	9.47	10.72	11.49	12.05	12.58	13.55	14.32	17.25	17.88	20.09	27.91	43.52
MEAN	1.060	1.329	1.454	1.530	1.598	1.666	1.779	1.934	2.226	2.358	2.792	3.807	6.481
CLOCK HR. COR.	1.140	1.070	1.040	1.020	1.010	1.010	1.000	1.000	1.000	1.000	1.000	1.000	1.000
CALCULATED SKEW	.663	.605	.815	1.008	1.037	1.133	1.237	1.084	1.499	1.444	1.302	1.936	1.578
REGIONAL SKEW	1.300	1.400	1.400	1.400	1.400	1.400	1.400	1.300	1.400	1.300	1.100	1.200	1.000
SKEW USED	1.300	1.400	1.400	1.400	1.400	1.400	1.400	1.300	1.400	1.300	1.100	1.200	1.000
KURTOSIS	3.212	3.150	2.880	3.426	3.574	3.701	4.394	4.095	6.012	5.746	4.328	7.584	5.922
N	24	24	24	24	24	24	24	24	24	24	24	24	23
RECORD YEAR	1956	1956	1962	1974	1974	1974	1969	1969	1969	1969	1969	1969	1969
RECORD MAXIMUM	1.680	2.170	2.600	3.120	3.360	3.360	3.860	4.190	5.650	5.660	6.390	10.370	14.710
NORMALIZED MAX	1.557	1.768	1.869	2.284	2.369	2.351	2.638	2.631	3.092	3.033	2.712	3.328	2.748
CALC. COEF. VAR	.376	.358	.422	.455	.465	.433	.444	.443	.498	.462	.475	.518	.462
REGN. COEF. VAR	.376	.408	.425	.434	.436	.437	.441	.427	.450	.439	.413	.422	.381
USED COEF. VAR	.376	.408	.425	.434	.436	.437	.441	.427	.450	.439	.413	.422	.381
MEAN/A	.1635	.2051	.2243	.2361	.2466	.2570	.2744	.2984	.3434	.3638	.4307	.5874	1.0000
RP10/A	.2458	.3170	.3517	.3731	.3903	.4071	.4362	.4690	.5500	.5777	.6693	.9197	1.5107
RP25/A	.2931	.3832	.4271	.4542	.4754	.4959	.5320	.5669	.6722	.7005	.7982	1.1049	1.7783
RP50/A	.3274	.4316	.4822	.5134	.5375	.5608	.6019	.6381	.7615	.7897	.8905	1.2384	1.9685
RP100/A	.3609	.4789	.5362	.5714	.5983	.6244	.6704	.7074	.8490	.8766	.9798	1.3681	2.1516
RP1000/A	.4682	.6315	.7100	.7583	.7944	.8292	.8911	.9297	1.1308	1.1552	1.2621	1.7809	2.7264
RP10000/A	.5717	.7798	.8789	.9398	.9849	1.0282	1.1055	1.1443	1.4046	1.4242	1.5309	2.1769	3.2696
PMP/A	1.0857	1.4605	1.6543	1.7733	1.8594	1.9416	2.0899	2.2093	2.6615	2.7594	3.0991	4.3056	6.7150

PEARSON TYPE III DISTRIBUTION USED
PROBABLE MAXIMUM PRECIPITATION ESTIMATE BASED ON 15 STANDARD DEVIATIONS
WHERE N IS SMALL RESULTS ARE NOT DEPENDABLE

B

EXHIBIT 2

20/23

TABLE 3. HYDRAULIC TRANSMISSIVITY (M^2/Sec X 10^{-3}) SOIL/TREVIRA 1120/PN3000 GEONET/TREVIRA 1120/SOIL

BY FLUID SYSTEM, INC, CINCINNATI, OHIO.

GRADIENT = 0.25

Specimen	5,000 psf	10,000 psf	15,000 psf	20,000 psf	25,000 psf
1.	0.41	0.16	0.07	0.04	0.02
2.	0.42	0.17	0.08	0.05	0.04
3.	0.48	0.23	0.14	0.11	0.09
Avg:	0.44	0.19	0.10	0.07	0.05
SD:	0.04	0.04	0.04	0.04	0.04

GRADIENT = 0.50

Specimen	5,000 psf	10,000 psf	15,000 psf	20,000 psf	25,000 psf
1.	0.36	0.14	0.08	0.03	0.02
2.	0.37	0.14	0.08	0.04	0.03
3.	0.40	0.18	0.11	0.07	0.06
Avg:	0.38	0.15	0.09	0.05	0.04
SD:	0.02	0.02	0.02	0.02	0.02

GRADIENT = 0.75

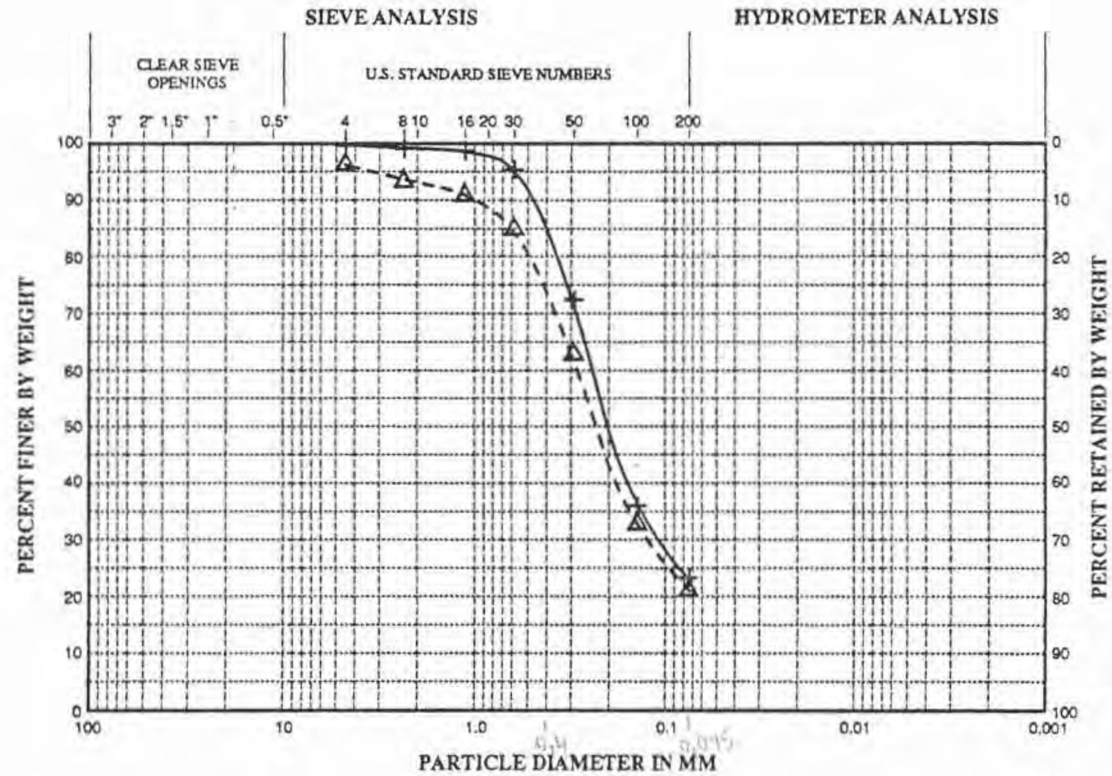
Specimen	5,000 psf	10,000 psf	15,000 psf	20,000 psf	25,000 psf
1.	0.32	0.14	0.06	0.04	0.03
2.	0.32	0.14	0.06	0.05	0.03
3.	0.35	0.16	0.08	0.06	0.05
Avg:	0.33	0.15	0.07	0.05	0.04
SD:	0.02	0.01	0.01	0.01	0.01

GRADIENT = 1.0

Specimen	5,000 psf	10,000 psf	15,000 psf	20,000 psf	25,000 psf
1.	0.26	0.12	0.05	0.03	0.03
2.	0.26	0.12	0.06	0.04	0.03
3.	0.29	0.14	0.07	0.05	0.04
Avg:	0.27	0.13	0.06	0.04	0.03
SD:	0.02	0.01	0.01	0.01	0.01

89-977/Fig D.2.7/G/SID/U18-9 REV. 8/7/90

SYMBOL	TEST PIT TYPE	DEPTH (ft.)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	STRATIGRAPHIC UNIT	MATERIAL TYPE	USCS
+———+	TP-1, B-1	7.0	--	--	18-9	Sandstone	SM
Δ----Δ	TP-42, B-1	6.0	--	--	18-9	Sandstone	SM



COBBLES	GRAVEL		SAND			SILT AND CLAY FRACTION
	coarse	fine	coarse	medium	fine	

FIGURE D.2.7

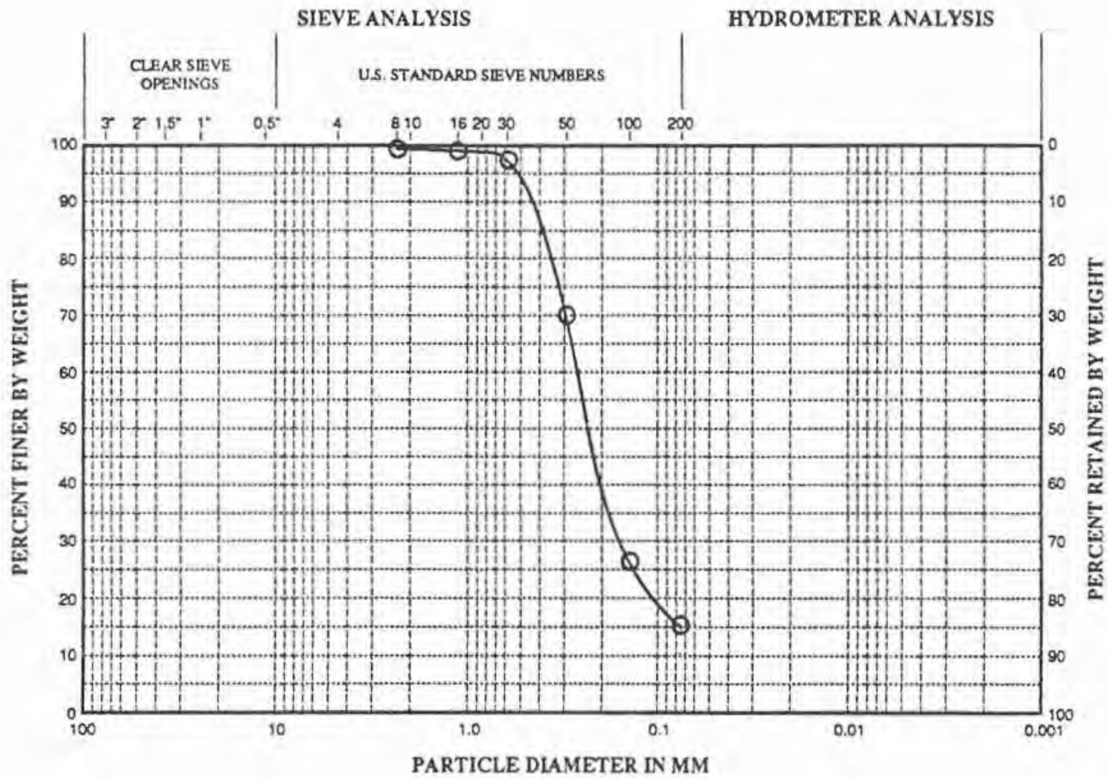
GRAIN SIZE DISTRIBUTION STRATIGRAPHIC UNIT 18-9

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS, INC.

89-977/Fig D.2.8/GrSiD/U18-11 REV. 8/7/90

SYMBOL	BORING	DEPTH (ft.)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	STRATIGRAPHIC UNIT	MATERIAL TYPE	USCS
o——o	L18-B	37.0-39.5	--	--	18-11	Sandstone	SM



COBBLES	GRAVEL		SAND			SILT AND CLAY FRACTION
	coarse	fine	coarse	medium	fine	

FIGURE D.2.8

GRAIN SIZE DISTRIBUTION
STRATIGRAPHIC UNIT 18-11

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS, INC.

TRANSMISSIVITY DATA IS BASED ON TREVIRA 1120

Trevira® Spunbond nonwoven engineering products are highly needled fabrics with excellent tensile properties, high filtration potential and outstanding permeability.

Trevira® Spunbond Type 11 products are 100% continuous filament polyester nonwoven needlepunched engineering fabrics. They deliver a combination of advantages unmatched by any other spunbonded geotextiles. They're resistant to freeze-thaw, soil chemicals and ultraviolet light exposure.

Trevira® Spunbond nonwoven engineering fabrics offer excellent performance where the requirement is tensile reinforcement, planar flow, filtration, or separation. They are ideal for roadways, railbeds, drainage systems, pondliners, retaining walls. And much more.

TYPICAL PHYSICAL PROPERTIES OF TREVIRA® TYPE 11 PRODUCTS

Fabric Property	Unit	Test Method	1112	1114	1120	1125	1135	1145	1155
Fabric Weight	oz/yd²	ASTM D-3776	3.5	4.2	6.0	7.5	10.5	13.5	16.5
Thickness, t	mils	ASTM D-1777	60	70	95	115	150	175	215
Grab Strength (MD/CD)[1]	lbs	ASTM D-4632	120/95	150/115	230/180	305/235	420/350	500/425	650/750
Grab Elongation (MD/CD)[1]	%	ASTM D-4632	65/75	65/70	65/75	65/75	65/75	70/75	70/75
Trapezoid Tear Strength (MD/CD)[1]	lbs	ASTM D-4533	50/40	55/50	80/75	105/90	145/130	185/170	215/190
Puncture Resistance	lbs	ASTM D-4833	55	65	100	115	160	180	230
Mullen Burst Strength	psi	ASTM D-3786	195	230	345	400	590	750	900
Water Flow Rate	gpm/ft²	ASTM D-4491	200	200	180	150	120	90	75
Permittivity, Ψ	sec⁻¹	ASTM D-4491	2.71	2.71	2.44	2.04	1.63	1.22	1.02
Permeability, k	cm/sec	k = Ψt	.41	.48	.59	.59	.62	.54	.56
AOS	Sieve Size mm	ASTM D-4751	70–100 .210–.149	70–100 .210–.149	70–100 .210–.149	70–100 .210–.149	70–120 .210–.125	100–120 .149–.125	120–170 .125–.088
Standard Roll Widths[2]	ft		←——— 12.5 and 15.0 ———→						
Standard Roll Length[2]	ft		400	400	300	300	300	300	300

[1]MD = Machine Direction, CD = Cross Machine Direction.

[2]Other width and length rolls are available upon request.

MINIMUM AVERAGE ROLL VALUES (WEAKEST PRINCIPAL DIRECTION) OF TREVIRA® TYPE 11 PRODUCTS

Fabric Property	Unit	Test Method	1112	1114	1120	1125	1135	1145	1155
Fabric Weight	oz/yd²	ASTM D-3776	3.3	4.0	5.7	7.1	10.0	13.0	16.0
Thickness, t	mils	ASTM D-1777	50	55	80	95	130	155	200
Grab Strength	lbs	ASTM D-4632	80	100	160	210	300	375	500
Grab Elongation	%	ASTM D-4632	50	50	50	50	50	50	50
Trapezoid Tear Strength	lbs	ASTM D-4533	30	40	60	75	100	130	160
Puncture Resistance	lbs	ASTM D-4833	40	45	80	95	130	155	195
Mullen Burst Strength	psi	ASTM D-3786	170	190	305	360	530	700	825
Water Flow Rate	gpm/ft²	ASTM D-4491	150	150	130	100	80	60	40
Permittivity, Ψ	sec⁻¹	ASTM D-4491	2.03	2.03	1.76	1.36	1.08	0.81	0.54
Permeability, k	cm/sec	k = Ψt	.26	.28	.36	.33	.36	.32	.28
AOS[3]	Sieve Size mm	ASTM D-4751							

EX 1817

5

23/23

The information contained herein is offered free of charge, and is, to our best knowledge, true and accurate; however, all recommendations or suggestions are made without guarantee, since the conditions of use are beyond our control. There is no expressed warranty and no implied warranty of merchantability or of fitness for purpose of the product or products described herein. In submitting this information, no liability is assumed or license or other rights implied given with

APPENDIX L
RISER PIPE ANALYSES

Objectives:

1. Evaluate the ability of the existing vertical LCRS riser pipes and underlying clay liner to withstand loading from an additional 90 vertical feet of waste placed above the original top of waste grade to the proposed Phase III top of waste grade.
2. Evaluate the ability of the existing sideslope riser pipes to withstand loading from an additional 90 vertical feet of waste placed above the original top of waste grade to the proposed Phase III top of waste grade.

Given:

The currently-permitted maximum waste height in the vicinity of the vertical and sideslope riser pipes is approximately 210 feet. The proposed Phase III expansion will increase this maximum waste height to approximately 300 feet (i.e., an approximately 90-foot increase). The as-built locations and configurations of the existing piping as well as the pipe materials and properties are shown on the Phases I and II construction drawings in Appendix A.1. The construction drawings in Appendix A.2 show the proposed final waste configuration.

Assumptions and Methodology:

The assumptions and methodology used to evaluate the existing vertical and sideslope riser pipes follows that of ESI (1990) for the original design of B-18. Golder has updated ESI's previous calculations to reflect the increased waste height (the methods utilized in the calculations are taken directly from the 1990 ESI calculations).

Summary of Results:

The calculations for the riser pipes are presented in Attachment A. The vertical riser pipe calculations are shown on pages 1 thru 11 while the sideslope riser pipe calculations are shown on pages 12 thru 17. The calculations indicate the following:

1. Vertical riser pipes: the existing vertical riser pipes and underlying clay liner are anticipated to have sufficient strength to resist the additional pressures from 90 extra feet of waste.

2. Sideslope riser pipes: the existing 8-inch-diameter steel riser pipes are anticipated to deflect a maximum of approximately 0.9% of their diameter under the full height of waste (300 feet), which is an acceptable deflection. As in the original design (ESI, 1990), the 8-inch-diameter HDPE riser pipes are anticipated to deflect more than 20% of their diameter (i.e., 21% to 34%). This amount of deflection exceeds the manufacturer's recommended maximum, but since the HDPE pipe is a backup to the steel pipe, it is considered acceptable for the sideslope riser application.

3. The proposed design provides for a transition from carbon steel pipe to a HDPE pipe. During the B-18 Phases I and II construction in the early 1990s, steel pipes were used due to the anticipated high loads and relatively new use of HPDE pipe. Since this time, however,

HDPE pipes are commonly used for LCRS riser pipes, including Landfill B-17 and Landfill B-19 Phase 1A at the Kettleman Hills Facility. Based on Golder's experience, there is little movement of the LCRS riser pipes once these pipes have been confined by soil cover/operations layer. This would be particularly true for the vadose and secondary riser pipes which are placed within excavated trenches and are below the weakest liner interface. For the primary riser pipe, movement of the waste could result in deflection of the LCRS riser. However, the movement of the waste (due to settlement) would primarily be down slope. Down slope movement would not result in significant shear on the LCRS riser; in fact the slip connection between the HDPE and steel pipes would allow for stress release if compression forces develop due to waste settlement. Additionally, it should be noted that the magnitude of settlement for the Class I waste in B-18 is relatively small compared with that of Class III municipal solid wastes. In summary, Golder believes that the riser pipes will not be subjected to shear forces that could damage the pipes and, therefore, the pipes will perform as designed. In the unlikely event of a failure at the steel/HDPE transition, the design includes redundant primary and secondary riser pipe systems which do not include the transition from steel to HDPE.

<u>Reference:</u>

Environmental Solutions Inc. (ESI), "Engineering and Design Report, Landfill Unit B-18, Phases I and II and Final Closure, Kettleman Hills Facility," August 1990.

Attachment A

Riser Pipe Calculations

Attachment A

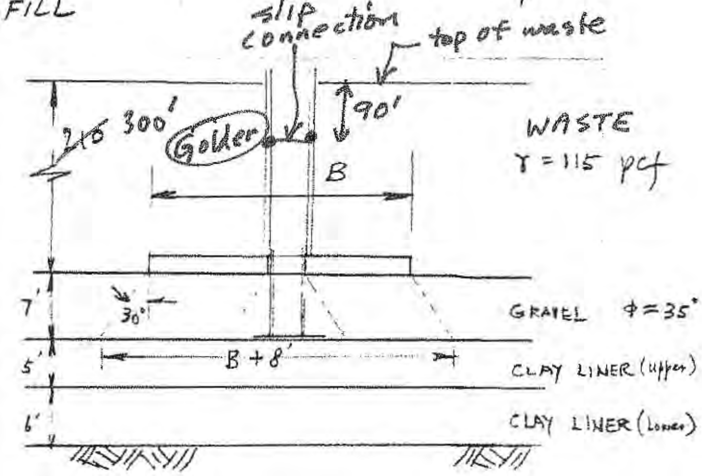
ENVIRONMENTAL SOLUTIONS, INC.

By GSC Date 7/27/90 Subject BEARING CAPACITY Sheet No. ~~3~~ 1 of ~~29~~ 26

Chkd. By [illegible] Date 8/15/90 VERTICAL RISER FOR Proj. No. 89-771

Modified By: Golder 5/18/08 KETTLEMAN LANDFILL

Objective : To calculate the available bearing capacity of the clay liner and design the riser base dimension



RESULTS FROM UU TEST OF MODIFIED PROCTOR FOR CLAYSTONE

INDICATE $\phi = 8°$ $c = 3600$ psf ----- CLAY LINER

USE $C = 3600$ psf for design

— Check squeezing effect to clay liner

Ref : "Foundation design & Construction" by Hamlington P.211

See ~~attachment 1~~ page 23 (Golder)

$$\frac{B + 8}{\text{Thickness of clay layer}}$$

Assuming max $B = 10'$

$$\frac{10+8}{5} = \frac{18}{5} = 3.6 < 6$$ Therefore, no squeezing effect is considered.

— Check allowable bearing pressure for clay liner

$q_u = cN_c + \gamma' d$ where C = undrained shear strength

$N_c = 6\,(1 + 0.2\ D/B)$ or limit to 9.0

D = Depth of footing below ground

B = width of foot

γ' = effective unit weight of soil

d = Thickness of footing

$= 9c + \gamma' d$

$= 9 \times 3600$ psf $+ 115 \times 2$ psf

$= 32400 + 230 = 32630$ psf or 32.6 ksf

Attachment A

ENVIRONMENTAL SOLUTIONS, INC.

By GSC Date 7/27/90 Subject BEARING CAPACITY Sheet No. ~~4~~ 2 of ~~29~~ 26

Chkd. By [illegible] Date 8/15/90 VERTICAL RISER BASE Proj. No. 89-977

Modified By: Golder 5/18/08

Use factor of safety of 2.0

Allowable bearing pressure at top of upper liner

$$= \frac{32.6}{2} = 16.3 \text{ ksf}$$

Assume use 8 × 8 riser base, and 30° tranfer of load from the base at top of gravel to top of clay liner

Area at top of clay liner $= (8+8)^2 - \left(\frac{2+8}{2}\right)^2 \pi$

$$= 256 - 78.5$$
$$= 177.5$$

Top load at base include weight of base

= ~~184.4~~ 302.5 (Golder) Kips (Ref. to ~~Leachate Sump Riser Calculations P. 3 & 4 of 14~~ pages 5–7)

Pressure exerted on top of upper clay liner

$$= \frac{\cancel{184.4}\ 302.5}{177.5} = \cancel{1.0}\ 1.7 \text{ KSF} < 16.3 \text{ KSF}$$

(Golder)

O.K.

For worst case if clay strength be c = 2000 psf

$$q_u = 9 \times 2000 + (115 - 62.4)\,2$$
$$= 18{,}000 + 105$$
$$= 18105 \text{ psf or } 18.1 \text{ ksf}$$

$$q_{allowable} = \frac{18.1}{2} = 9.1 \text{ ksf}$$

(Golder)

Applied pressure $= \cancel{1.0}\ 1.7 \text{ ksf} < 9.1 \text{ ksf}$ O.K.

Attachment A

ENVIRONMENTAL SOLUTIONS, INC.

By GSC Date 7/27/90 Subject BEARING CAPACITY Sheet No. ~~8~~ 3 of ~~29~~ 26

Chkd. By TPL Date 8/15/90 Proj. No. 89-977

Modified By: Golder 5/18/08

For 6 x 6 riser base

Area at Top clay liner $= (6+8)^2 - \left(\frac{2+8}{2}\right)^2 \pi$

$= 196 - 78.5$

$= 117.5 \text{ ft}^2$ Golder

Pressure exerted on clay liner $= \frac{\cancel{184.4}\ 302.5}{117.5} = \cancel{1.57}\ 2.57 \text{ KSF} < 9.1 \text{ KSF}$

O.K.

– Check consolidation settlement

The weight of fill above The clay liner ~~= 237 x 115~~ ~~= 27255 psf~~ ~~= 27.3 KSF~~

$= 300 \times 115 + 7 \times 130$

$= 35410$ PSF (35.4 KSF)

Golder

Additional pressure due to riser = 1.56 KSF

Based on consolidation test results (see attached test results) pages 24-25

An additional settlement of 0.5 percent may be expected

$\therefore\ 0.5\% \times 8' = 0.04$ inch (at base of riser)

- Check settlement in the gravel layer below the base of vertical riser

Elastic settlement :

Vertical stress at bottom of riser base

$$\sigma_v = \cancel{237}^{300} \times 0.115 + 184.4 / \left[6 \times 6 - \left(\tfrac{2}{2}\right)^2 \pi \right] \text{ ksf}$$

Golder $= \cancel{27.3}^{34.5} + 5.6$

$= \cancel{32.9}^{40.1}$ ksf

Typical value of E_s for sand and gravel (see ~~attachment~~ p. 26)

Assume $E_s = 14$ ksi

$= 2016$ ksf

$$\epsilon = \frac{\sigma_v}{E} = \frac{\cancel{32.9}^{40.1}}{2016} = \cancel{0.016}^{0.02}$$

Golder

$\therefore$ Elastic settlement $= \cancel{0.016}^{0.02} \times 7'$

1.67" (0.139')

$= 1.37$ inches

CONCLUSIONS :

1). WITH A 6'x6' VERTICAL RISER BASE, THE PRESSURE EXERTED ON CLAY LINER BENEATH THE BASE IS ABOUT ~~1.6~~ 3.0 KSF. THIS COMPARES WITH A ULTIMATE BEARING CAPACITY OF AT LEAST 18.1 KSF FOR THE CLAY LINER. IT PROVIDES A FACTOR OF SAFETY OF AT LEAST ~~11~~ 7 AGAINST BEARING FAILURE FOR THE RISER BASE. Golder

Golder

2.) IT IS ANTICIPATED THAT THERE WILL BE A DIFFERENTAL MOVEMENT BETWEEN THE RISER BASE AND THE BOTTOM STEEL PIPE INSERT. THE MOVEMENT IS EXPECTED TO BE ABOUT ~~1.3~~ 1.67 INCHES DUE TO THE ELASTIC DEFORMATION OF THE GARVEL LAYER BENEATH THE BASE.

Golder

Golder Associates	SUBJECT Attachment A — B-18 Riser Pipes		
	Job No. Ref. 083-91887	Made by EH Checked RH Reviewed	Date 04/11/08 Sheet 5 of 26

TO BE CONSERVATIVE, IT IS ASSUMED THAT DOWNDRAG FORCE DUE TO WASTE SETTLEMENT WILL BE TRANSFERRED TO THE STEEL PIPES THROUGH CONTACT BETWEEN THE TWO PIPES. USING A AREA OF 10% OF THE STEEL PIPE, THE DOWNDRAG FORCE IS:

$$F_D = \int_{H_1}^{H_2} (p K_0 \gamma \tan\delta) z \, dz$$

$$= \frac{K_0 \gamma (H_1 + H_2)}{2} \cdot (H_2 - H_1) \cdot \pi \cdot \frac{24}{12} \cdot \tan\delta \cdot 10\%$$

$$= \frac{(1 - \sin 27^\circ) \cdot 115 \cdot (90 + 300)}{2} \cdot (300 - 90) \cdot \pi \cdot 2 \cdot \tan\delta \cdot 10\%$$

$$= 255.8 \text{ kips}$$

WEIGHT OF STEEL PILE SCH 40

$$W_p = 171 \cdot 210 = 35.9 \text{ kips}$$

WEIGHT OF HDPE PIPE

$$W_H = 15.5 \cdot 210 = 3.3 \text{ kips}$$

TOTAL FORCE

$$F = 255.8 + 35.9 + 3.3 = 295 \text{ kips}$$

A 6x6x1.5 CONCRETE PAD USED

Golder Associates	SUBJECT Attachment A – B-18 Riser Pipes		
	Job No. Ref. 083-91887	Made by EH Checked RH Reviewed	Date 04/11/08 Sheet 6 of 26

CHECK ON ORIGINAL DESIGN OF THE RISER FOUNDATION

BEARING PLATE FOR THE STEEL PIPE

CIRCUMFERENCE OF VERTICAL RISER

$C = (24 - 0.687) \times \pi = 73.24$ in

CONTACT AREA $= 73.24 \cdot 0.687 = 50.3\ in^2$

WEIGHT OF PIPE + DOWNDRAG $= 35.9 + 255.8 = 291.7$ kips

BEARING PRESSURE AT STEEL PLATE $= \frac{291.7}{73.24} = 3.98$ kips/in

BEARING PRESSURE @ CONCRETE $= \frac{295}{\frac{\pi}{4}(36^2 - 19^2)} = 0.401$ ksi

3000 psi CONCRETE USED

ALLOWABLE BEARING PRESSURE $= 0.5 \cdot 3000 = 1500$ psi > 401 psi OK

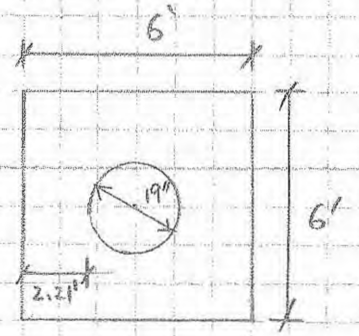
ALLOWABLE SHEAR STRESS IN BEARING PLATE $= 0.4 F_y$

USE A36 STEEL $F_y = 36$ ksi

$\therefore F_v = 0.4 \cdot 36 = 14.4$ ksi $> \frac{291.7}{50.3} = 5.8$ ksi OK

Golder Associates	SUBJECT Attachment A - B-18 Riser Pipes		
	Job No. Ref. 083-91887	Made by EH Checked RH Reviewed	Date 04/11/08 Sheet 7 of 26

A 6×6×1.5 CONCRETE PAD USED



$W = 6 \cdot 6 \cdot 1.5 \cdot 144 - \frac{\pi}{4}(19/12)^2 \cdot 1.5 \cdot 144$

$= 7.35$ kips

TOTAL DEAD : $295 + 7.35 = 302.35$ kips

$f'_c = 3000$ psi, $f_y = 60{,}000$ psi, $d = 1.5 - 0.25 - \left(\frac{0.5}{2}\right)/12 = 1.23' = 14.76''$

$q_u = \frac{1.4 \cdot 302.35}{34.03} = 12.44$ ksf

CRITICAL SHEAR :

ONE-WAY SHEAR

$V_u = q_u \cdot b \cdot (2.21 - d)$

$= 12.44 \cdot 6 \cdot (2.21 - 1.23)$

$= 73.15$ Kips

$V_c = \phi \cdot 2\sqrt{f'_c}\; bd$

$= 0.85 \cdot 2 \cdot \sqrt{3000} \cdot (6 \cdot 12) \cdot (1.23 \times 12)$

$= 98.9$ kips $> V_u$ OK

TWO-WAY SHEAR

$V_u = q_u \cdot \left(6^2 - \left(\frac{19+d}{12}\right)^2\right)$

$= 12.44 \cdot \left(6^2 - \left(\frac{19 + 14.76}{12}\right)^2\right)$

$= 349.4$ Kips

$V_c = \phi \cdot 4 \sqrt{f_c'}\ b_o d$

$b_o = 4 \cdot (19 + d) = 4 \cdot (19 + 14.76) = 135.04$

$V_c = 0.85 \cdot 4 \cdot \sqrt{3000} \cdot 135.04 \cdot 14.76$

$= 371 \text{ kips} > 349.4 \text{ kips}$

BENDING

CRITICAL BENDING MOMENT :

$$M_u = q_u \cdot \frac{(2.21 - d)^2}{2}$$

$$= 12.44 \cdot \frac{(2.21 - 1.23)^2}{2}$$

$$= 5.97 \text{ ft - kips}$$

PERCENTAGE OF STEEL REQUIRED

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 m R_u}{f_y}}\right)$$

WHERE $m = \dfrac{f_y}{0.85 f_c'} = \dfrac{60000}{0.85 \cdot 3000} = 23.5$

$R_u = \dfrac{M_u}{\phi b d^2} = \dfrac{5.97 \cdot 12 \cdot 1000}{0.9 \cdot 12 \cdot 14.76} = 30.45$

$$\therefore \rho = \frac{1}{23.5}\left(1 - \sqrt{1 - \frac{2 \cdot 23.5 \cdot 30.45}{60000}}\right)$$

$$= 0.00051 = 0.051\ \%$$

#5 RE-BAR @ 6" c/c BW USED

$A_s = 0.62 \, in^2$

$$\rho = \frac{0.62}{bd} = \frac{0.62}{12 \cdot 14.76} = 0.35\% > 0.051\% \quad ok$$

LOWER VERTICAL RISER

PRESSURE AT TOP OF PIPE

OVERBURDEN = 300 · 115 = 34500 psf

BEARING PRESSURE = 4950 psf

PRESSURE AT BOTTOM OF PIPE

OVERBURDEN = 34500 + 7 · 130 = 35410 psf

BEARING PRESSURE = 1017 psf

LATERAL PRESSURE UNDER AT-REST CONDITION

$$K_0 = (1-\sin\phi) = (1-\sin 40^\circ) = 0.35$$

$\therefore$

$$P_T = 0.35 \cdot (34500 + 4950) = 13808 \text{ psf}$$

$$P_B = 0.35 \cdot (35410 + 1017) = 12749 \text{ psf}$$

RING COMPRESSION AT TOP OF PIPE

$$P = \frac{13808}{2} \cdot D$$

$$= 6904 D$$

Golder Associates	SUBJECT Attachment A – B-18 Riser Pipes		
	Job No. Ref. 083-91997	Made by EH Checked RH Reviewed	Date 04/11/08 Sheet 11 of 26

18" STAINLESS STEEL PIPE ($E = 28 \times 10^6$ psi) USED

NOMINAL DIAMETER	OD	ID	t	r	I
18	18	16.5	0.75	8.625	0.035

$$\therefore P = 6904 \cdot \left(\frac{18}{12}\right) = 10356 \text{ lb/ft} = 863 \text{ lb/in}$$

STRESS IN PIPE WALL = $863/0.75 = 1150.7$ psi

A FS of 3 IS REQUIRED TO AVOID COLLAPSING OF THE PIPE

THE MAX. CRITICAL PRESSURE IS:

$$P_{CR} = \frac{3EI}{R^3} = \frac{24EI}{R^3}$$

$$FS = \frac{P_{CR}}{P} = \frac{24 \cdot 28 \times 10^6 \cdot 0.035}{(8.625 \cdot 2)^3 \cdot 1150.7} = 3.98 > 3. \quad \underline{\underline{OK}}$$

PIPE DEFLECTION MAY BE DETERMINED BY THE IOWA'S EQUATION:

$$\Delta_x = \frac{KWr^3}{EI + 0.061E'r^3} \cdot D_e = \text{pipe deflection}$$

WHERE:

D_e = DEFLECTION FACTOR (= 1.5)

K = BENDING COEFFICIENT (= 0.1)

W = DESIGN LOAD

r = MEAN RADIUS OF PIPE ($= \frac{OD - t}{2}$)

E = MODULUS OF PIPE (= 20,000 psi)

I = MOMENT INERTIA OF PIPE ($= \frac{t^3}{12}$)

E' = MODULUS OF SOIL REACTION (= 1000 psi)

(COARSE GRAVEL MODERATELY COMPACTED)

DEFLECTION OF THE SLOPE RISER

(1) 8" ϕ DUCTILE IRON PIPE (SCH80)

NOMINAL DIAMETER	OD	ID	t	r	I
8"	8.625	7.625	0.5	4.063	0.010

$E = 24 \times 10^6$ FOR DUCTILE IRON PIPE

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TRENCH CONDITION

WASTE FILL OVERBURDEN

$$q_f = 115 \times 300 = 34500 \text{ psf}$$

PRESSURE AT TOP OF PIPES DUE TO WASTE FILL

$$\sigma_{v1} = q_f \, C_{\mu s}$$

$$= q_f \cdot e^{-2K_{\mu}'(H/B_d)}$$

$$= 34500 \cdot e^{-2 \cdot 0.165 \cdot \left(\frac{18-3-8.625}{18}\right)}$$

$$= 30694 \text{ psf}$$

PRESSURE AT TOP OF PIPE DUE TO TRENCH BACKFILL

$$\sigma_{v2} = B_d \cdot \gamma \cdot C_d$$

$$= 1.5 \cdot 130 \cdot \frac{1 - e^{-2 \cdot 0.165 \cdot (6.375/18)}}{2 \cdot 0.165}$$

$$= 65 \text{ psf}$$

TOTAL PRESSURE AT TOP OF PIPE

$$\sigma_{v0} = 30694 + 65 = 30759 \text{ psf}$$

INCREASE OF STRESS DUE TO VERTICAL RISER :

$$\Delta\sigma = \frac{1837}{13^2 - \left(\frac{7+19}{12}\right)^2 \frac{\pi}{4}} = 1111 \text{ psf}$$

$$\sigma_v = 30759 + 1111 = 31870 \text{ psf}$$

FORCE PER UNIT LENGTH OF PIPE

$w = \sigma_v \cdot B_c$

$= 31870 \cdot 8.625/12$

$= 22906$ lb/ft

$= 1909$ lb/in

$\therefore \Delta y = \dfrac{K w r^3}{EI + 0.061 E' r^3} \cdot D_e$

$= \dfrac{0.1 \times 1909 \times 4.063^3}{24 \cdot 10^6 \times \frac{0.5^3}{12} + 0.061 \times 1000 \times 4.063^3} \cdot 1.5$

$= 0.079$

$\therefore \dfrac{\Delta y}{D} = \dfrac{0.079}{8.625} = 0.9\ \% \ll 5\ \%$ OK

POSITIVE PROJECTION

TOTAL VERTICAL STRESS AT TOP OF PIPE

$\sigma_v = \sigma_{v1} + \Delta\sigma = 34500 + 1017 = 35517$

FORCE PER UNIT LENGTH OF PIPE

$w = \sigma_v B_c$

$= 35517 \cdot \dfrac{8.625}{12}$

$= 25527$ lb/ft

$= 2127$ lb/in

Golder Associates	SUBJECT Attachment A — B-18 Riser Pipes		
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$$\therefore \Delta y = \frac{0.1 \times 2127 \cdot 4.063^3}{24 \times 10^6 \times \frac{0.5^3}{12} + 0.061 \cdot 1000 \cdot 4.063^3} = 0.08''$$

$$\frac{\Delta y}{D} = \frac{0.08}{8.625} = 0.7\% \ll 5\% \quad OK \#$$

(2)

CHECK DEFLECTION OF THE 8"φ HDPE PIPE (SDR = 8.3), THE HDPE PIPE HAS THE FOLLOWING PROPERTIES:

NOMINAL DIAMETER	OD	ID	t	r	I	E
8"	8.625	6.549	1.039	3.793	0.093	20,000 psi

IN TRENCH CONDITION, THE NOMINAL PRESSURE AT TOP OF PIPE IS 30759 psf (= 213.6 psi)

$\therefore$ FORCE PER UNIT LENGTH OF PIPE

$$W = 30759 \cdot \frac{8.625}{12} = 22108 \ lb/ft$$

$$= 1842.3 \ lb/in$$

$$\therefore \Delta y = \frac{0.1 \times 1842.3 \times 3.793^3}{20000 \times 0.093 + 0.061 \times 1000 \times 3.793^3} \cdot 1.5 = 2.91''$$

$$\frac{\Delta y}{D} = \frac{2.91}{8.625} = 34\%$$

THE FOLLOWING IS BASED ON THE PROCEDURES SUGGESTED BY THE HDPE PIPE MANUFACTURER (pages 18-22)

Golder Associates	SUBJECT Attachment A – B-18 Riser Pipes		
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TOTAL EXTERNAL PRESSURE AT TOP OF PIPE:

$P_t = 213.6$ psi (see page 16)

EXAMINE SHORT-TERM WALL CRUSHING:

$$S_A = \frac{(SDR-1)P_t}{2} = \frac{(8.3-1)\times 213.6}{2} = 780 \text{ psi} < 1500 \text{ psi}$$

CALCULATE THE CRITICAL COLLAPSE PRESSURE:

$$P_c = \frac{2.32 \cdot 20000}{8.3^3} = 81$$

EXAMINE WALL-BUCKLING OF THE PIPE SOIL SYSTEM

ASSUME $P_{CB} = P_t$

THE REQUIRED SOIL MODULUS E' TO RESIST BUCKLING IS:

$$E' = \frac{213.6^2}{0.64 \cdot 81} \doteq 880 \text{ psi}$$

SINCE THE PIPE IS SURROUNDED BY GRAVEL, TO BE CONSERVATIVE, A SOIL MODULUS OF 1000 psi IS USED:

$$\text{PIPE DEFLECTION} = \% \text{ SOIL STRAIN} = \frac{213.6}{1000} \cdot 100 = 21\% > 2\%$$

THE DEFLECTION EXCEEDS THE MANUFATURER'S RECOMMENED ALLOWABLE DEFLECTION. SINCE THE HDPE PIPE SYSTEM IS A REDUNDANT SYSTEM. ~~IT CAN BE REPLACED BY THE STEEL PIPE ALONGSIDE THE HDPE PIPE.~~

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DRISCOPIPE® 8600

ULTRA-HIGH MOLECULAR WEIGHT HIGH DENSITY POLYETHYLENE

Dimensions and Pressure Ratings

ORDERING INFORMATION

PRICE	Prices subject to change without notice.
FREIGHT	1. All prices F.O.B. shipping point. 2. Less than truckload shipments will be shipped freight collect, or freight prepaid and charged, from point of origin.
TERMS AND CONDITIONS	Net 30 days. All orders are subject to Phillips Driscopipe, Inc.'s acceptance and to the terms and conditions contained in its then current Sales Order Acknowledgement form.
SPECIAL QUOTATIONS	Pricing for special sizes, dimensions, lengths, special packaging, special handling, and special "shop" alterations available upon request.
HOW TO ORDER	When ordering, please provide the following information: 1. Name and address of your organization 2. Your Purchase Order number 3. Nominal size 4. Footage or number required 5. Preferred method of shipment 6. Shipping destination 7. Desired delivery date

NOTE: Standard Length is 40 feet for all sizes.

FOR ADDITIONAL INFORMATION ON PRICES, SIZES OR FUSION EQUIPMENT CONTACT:

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LOW PRESSURE

NOMINAL SIZE	DIMENSIONS-INCHES NOMINAL OD	APPROX. ID	MINIMUM WALL	SDR	NOMINAL WEIGHT LBS/100'	JOINT LENGTH FT.	DESIGN PRESSURE PSI-73.4°F
3"	3.500	3.300	0.100	35	46	40	47
4"	4.500	4.200	0.150	30	88	40	55
*4"	4.500	4.026	0.237	19.0	135	40	89
5"	5.250	4.926	0.162	32.5	111	40	51
*5"	5.563	5.047	0.258	21.5	183	40	78
6"	6.625	6.217	0.204	32.5	176	40	51
7"	7.125	6.687	0.219	32.5	203	40	51
*7"	7.125	6.333	0.396	18	357	40	94
8"	8.625	8.095	0.265	32.5	297	40	51
10"	10.750	10.088	0.331	32.5	463	40	51
*10"	10.750	10.022	0.364	29.5	507	40	56
*12"	12.750	11.940	0.405	31.5	671	40	52
*14"	14.000	13.138	0.431	32.5	784	40	51
16"	16.000	15.016	0.492	32.5	1023	40	51
*16"	16.000	15.000	0.500	32.0	1039	40	52
18"	18.000	16.892	0.554	32.5	1296	40	51
*20"	20.000	18.806	0.597	33.5	1554	40	49
20"	20.000	18.770	0.615	32.5	1599	40	51
22"	21.500	20.176	0.662	32.5	1850	40	51
24"	24.000	22.524	0.738	32.5	2303	40	51
28"	27.953(1)	26.233	0.860	32.5	3125	40	51
32"	31.496(2)	29.558	0.969	32.5	3967	40	51
36"	36.000	33.784	1.108	32.5	5185	40	51
42"	42.000	39.416	1.292	32.5	7054	40	51
48"	47.244(3)	44.336	1.454	32.5	8930	40	51

(1) 710 MM (2) 800 MM (3) 1200 MM

65 psi

NOMINAL SIZE	DIMENSIONS NOMINAL OD	APPROX. ID	MINIMUM WALL	SDR	NOMINAL WEIGHT LBS/100'	JOINT LENGTH FT.
*6"	6.625	6.065	0.280	23.5 (71 psi)	238	40
*8"	8.625	7.981	0.322	27 (62 psi)	359	40
10"	10.750	9.900	0.425	25.3	588	40
20"	20.000	18.418	0.791	25.3	2037	40
24"	24.000	22.102	0.949	25.3	2933	40
28"	27.953(1)	25.743	1.105	25.3	3978	40
36"	36.000	32.572	1.714	21	7877	40
48"	47.244(3)	43.610	1.817	26	11068	40

(1) 710 mm (3) 1200 mm

110 psi

NOMINAL SIZE	DIMENSIONS-INCHES NOMINAL OD	APPROX. ID	MINIMUM WALL	SDR	NOMINAL WEIGHT LBS/100'	COIL OR JOINT LENGTH FT.
*1½"	1.900	1.610	0.145	13.1	34	500
*2"	2.375	2.069	0.153	15.5	46	300
*3"	3.500	3.068	0.216	16 (107 psi)	95	40
3"	3.500	3.048	0.226	15.5	99	40
4"	4.500	3.920	0.290	15.5	164	40
6"	6.625	5.771	0.427	15.5	355	40
8"	8.625	7.513	0.556	15.5	601	40
10"	10.750	9.362	0.694	15.5	935	40
12"	12.750	11.104	0.823	15.5	1315	40
14"	14.000	12.194	0.903	15.5	1584	40
16"	16.000	13.936	1.032	15.5	2069	40
18"	18.000	15.678	1.161	15.5	2619	40
22"	21.500	18.726	1.387	15.5	3737	40
24"	24.000	20.904	1.548	15.5	4656	40

130 psi

NOMINAL SIZE	DIMENSIONS-INCHES NOMINAL OD	APPROX. ID	MINIMUM WALL	SDR	NOMINAL WEIGHT LBS/100'	JOINT LENGTH FT.	DESIGN PRESSURE PSI-73.4°F
3"	3.500	2.982	0.259	13.5	112	40	130
4"	4.500	3.834	0.333	13.5	186	40	130
6"	6.625	5.643	0.491	13.5	403	40	130
8"	8.625	7.347	0.639	13.5	683	40	130

160 psi

NOMINAL SIZE	DIMENSIONS-INCHES NOMINAL OD	APPROX. ID	MINIMUM WALL	SDR	NOMINAL WEIGHT LBS/100'	COIL OR JOINT LENGTH FT.
¾"	1.050	0.860	0.095	11	12	500
1"	1.315	1.075	0.120	11	19	500
1¼"	1.660	1.348	0.151	11	31	500
1½"	1.900	1.554	0.173	11	40	500
2"	2.375	1.943	0.216	11	62	350
3"	3.500	2.864	0.318	11	135	40
4"	4.500	3.682	0.409	11	224	40
5"	5.563	4.551	0.506	11	342	40
6"	6.625	5.421	0.602	11	485	40
8"	8.625	7.057	0.784	11	823	40
10"	10.750	8.796	0.977	11	1278	40
12"	12.750	10.432	1.159	11	1798	40
14"	14.000	11.454	1.273	11	2168	40
16"	16.000	13.090	1.455	11	2833	40
18"	18.000	14.728	1.636	11	3583	40
22"	21.500	17.590	1.955	11	5114	40
24"	24.000	19.636	2.182	11	6372	40

190 psi

NOMINAL SIZE	DIMENSIONS-INCHES NOMINAL OD	APPROX. ID	MINIMUM WALL	SDR	NOMINAL WEIGHT LBS/100'	COIL OR JOINT LENGTH FT.
¾"	1.050	0.824	0.113	9.33	14	300
1"	1.315	1.033	0.141	9.33	22	300
1¼"	1.660	1.304	0.178	9.33	35	500
2"	2.375	1.865	0.255	9.33	72	350
3"	3.500	2.750	0.375	9.33	157	40
4"	4.500	3.536	0.482	9.33	259	40
6"	6.625	5.205	0.710	9.33	562	40

220 psi

NOMINAL SIZE	DIMENSIONS-INCHES NOMINAL OD	APPROX. ID	MINIMUM WALL	SDR	NOMINAL WEIGHT LBS/100'	JOINT LENGTH FT.
8"	8.625	6.547	1.039	8.3	1054	40
14"	14.000	10.164	1.918	7.3 (254 psi)	3096	40

STANDARD PACKAGING FOR DRISCOPIPE® 8600 INDUSTRIAL PIPE

PIPE DESCRIPTION		BUNDLE		TRUCK LOAD BUNDLED		40 FT. FLOAT TRUCKLOAD — LOOSE	
NOMINAL SIZE	O.D.	JOINTS	LINEAR FEET	BUNDLES	LINEAR FEET	JOINTS	LINEAR FEET
2"	2.375	88	3,520	14	49,280		
3"	3.500	46	1,840	14	25,760		
4"	4.500	27	1,080	14	15,120		
5"	5.563	15	600	14	8,400		
6"	6.625	11	440	14	6,160		
7"	7.125	11	440	12	5,280		
8"	8.625	8	320	12	3,840		
10"	10.750					80	3,200
12"	12.750					59	2,360
14"	14.000					48	1,920
16"	16.000					35	1,400
18"	18.000					28	1,120
20"	20.000					20	800
22"	21.500					18	720
24"	24.000					16	640
28"	27.953					10	400
32"	31.496					9	360
36"	36.000					6	240
42"	42.000					4	160
48"	47.244					4	160

NOTE: OBTAIN TRUCK LOAD WEIGHT BY MULTIPLYING LINEAR FEET TIMES PIPE WEIGHT PER FOOT.

NOTE: Approximate ID = Nominal OD − 2 × Minimum Wall
SDR (Standard Dimension Ratio) = OD ÷ Minimum Wall
*These sizes are also Schedule 40 dimensions.
Pressure rating computed on the basis of the following:

$$P = \frac{2S}{SDR-1} \text{ @ } 73.4°F$$

Where:
Dp = Nominal OD of Pipe, Inches
t = Minimum Wall Thickness
S = Hydrostatic Design Stress, 800 psi
P = Pressure Rating, psi @173.4°F

NOTE:
Approximate ID = Dp − 2t
SDR (Standard Dimension Ratio) = $\frac{Dp}{t}$

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Burial Design Guidelines: By combining the Burial Design Considerations with the Total External Soil Pressure, calculated by components, the designer can select the proper pipe SDR and specify the soil density to engineer into the pipeline the desired performance of the "pipe-soil" system. The following guidelines are presented for evaluation when designing a specific Driscopipe system. Because various parameters are available, in different situations, the guidelines may be approached in a mixed order or the equations may require mathematical re-arrangement. These guidelines, along with the following notes and sample problem, should be helpful:

1. Calculate by components the total external soil pressure P_t at the top of the pipe.

2. Examine Short Term Wall Crushing by calculating the compressive stress in the wall of the pipe at the springline:

   $$S_A = \frac{(SDR-1)\,P_t}{2}$$

   (a) If $S_A < 1500$ psi proceed to #3
   (b) If $S_A > 1500$ psi consider a heavier pipe wall

3. Calculate the critical-collapse pressure, P_c, from this formula using the time dependent modulus of elasticity, E, rated at the stress level calculated above in #2. (see Chart 25).

   $$P_c = \frac{2.32\,E}{(SDR)^3}$$

4. Examine Wall-Buckling of the pipe-soil system. By assuming the critical-buckling pressure, P_{cb}, equals the pressure at the top of the pipe, P_t, (see #1), and by using the critical pressure, P_c, calculated in #3, the basic soil modulus, E', required to resist buckling can be calculated by:

   $$E' = \frac{(P_{cb})^2}{.64\,(P_c)}$$

5. To safeguard against wall buckling, multiply E' by a reasonable safety factor (S.F.) equal to or greater than 2.0.

   $$E'_{MIN} = (E')(S.F.)$$

6. Calculate pipe deflection based upon the principle that its deflection will be the same as the backfill surrounding the pipe under the influence of the soil pressure at the top of the pipe:

   $$\%\ \text{Soil Strain} = \xi_S = \frac{P_t}{E'_{MIN}} \times 100$$

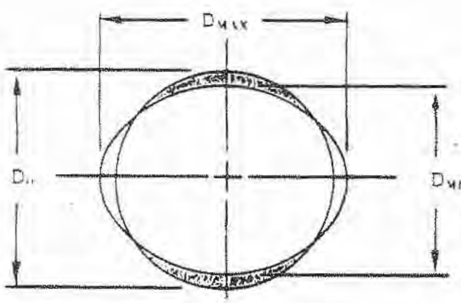
7. Examine allowable Ring Deflection for the specific SDR under consideration to insure the pipe deflection (#6) is less than the allowable deflection for that SDR. (See Chart 27).

   - If the actual deflection exceeds the permissable value, increase E', the soil strength modulus, and re-calculate #6. The other alternative is to consider another SDR at #1.

<u>Design by Ring Deflection</u>: 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 insure 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 SDR's 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%



$$\%\ \text{RING DEFLECTION} = \left(1 - \frac{D_{MIN}}{D_o}\right) \times 100\%$$

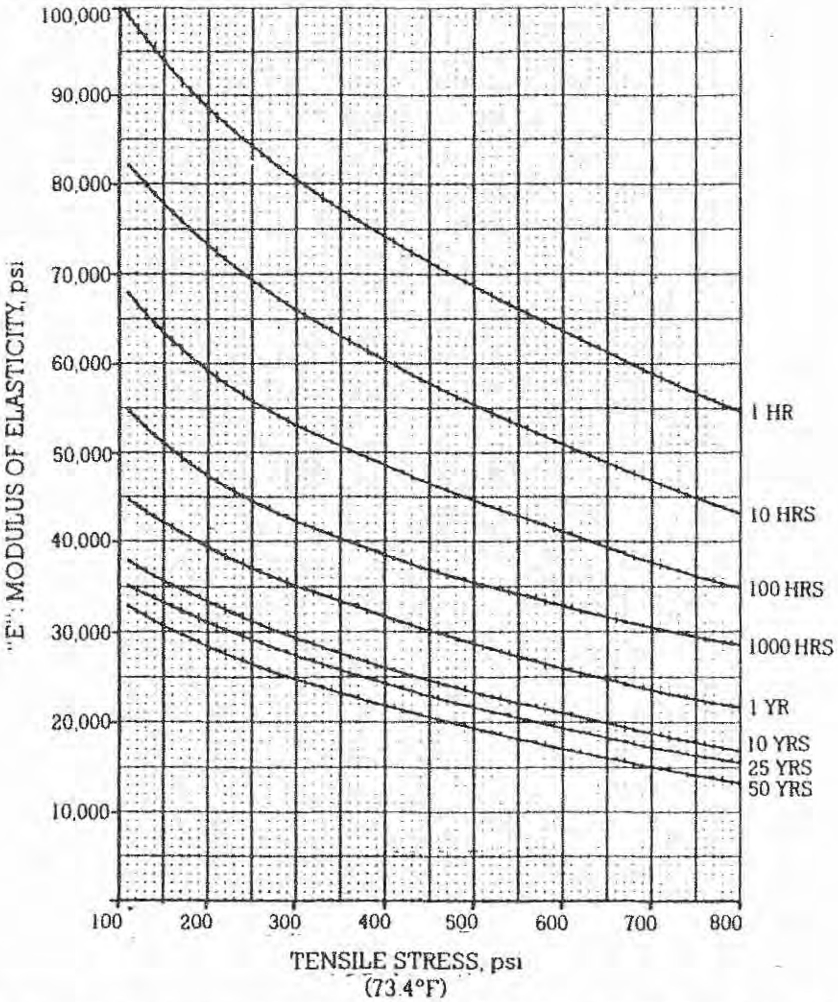
The allowable ring deflection of polyethylene pipe is a function of the allowable tangential strain in the outer surface of the pipe wall. A conservative limit of 1 - 1½% tangential strain in the outer surface of the pipe wall due to vertical deflection of the pipe "ring" by soil compression can be understood by comparing two pipes of the same diameter but different wall thickness.

Assume each of the pipes is equally deflected under loads required to achieve that result. The tangential surface strain developed in the thickwall pipe is much greater than the surface strain in the thinwall pipe. The tangential strain varies directly as the wall thickness (i.e.: distance from the neutral axis) and is proportional to the amount of ring deflection. For a given ring deflection, the thicker the wall, the higher the strain.

Alternately, assume that each of the pipes are subjected to loads such that the tangential surface strain in the pipe's wall surface is equal for both pipes. For equal surface strain, the degree of vertical deflection of the pipe ring is different for the two pipes. Under these circumstances, the degree of deflection would be less for the thickwall pipe and greater for the thinwall pipe.

CHART 25

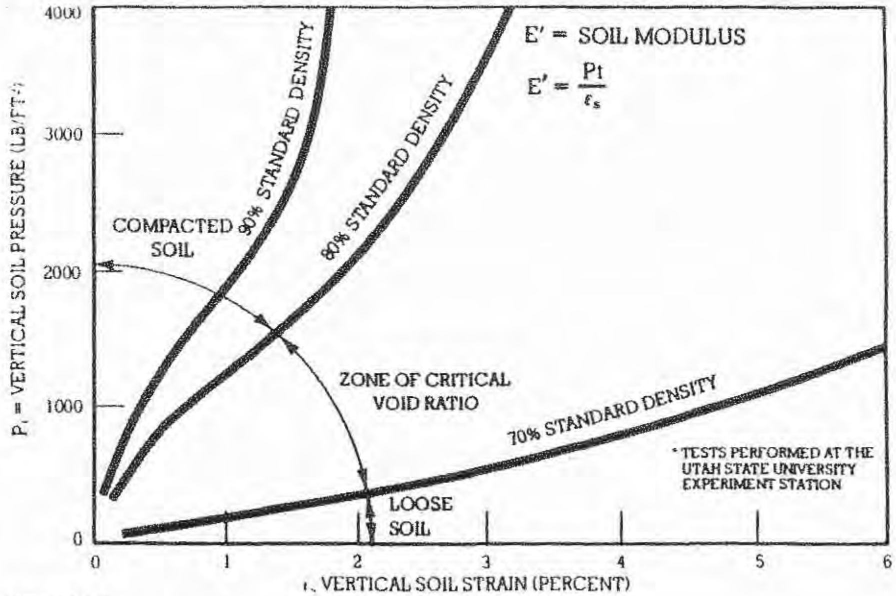
TIME DEPENDENT MODULUS OF ELASTICITY FOR POLYETHYLENE PIPE VS. STRESS INTENSITY (73.4°F)



NOTE: The short term modulus of elasticity of Driscopipe per ASTM D 638 is approximately 100,000 psi. Due to the cold flow (creep) characteristic of the pipe material, this modulus is dependent upon the stress intensity and the time duration of the applied stress.

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: $E' = Pt/\epsilon_s$

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.

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underside of a pad foundation and the top of the soft clay exceeds half the width of the foundation, the resistance of the stiff layer in

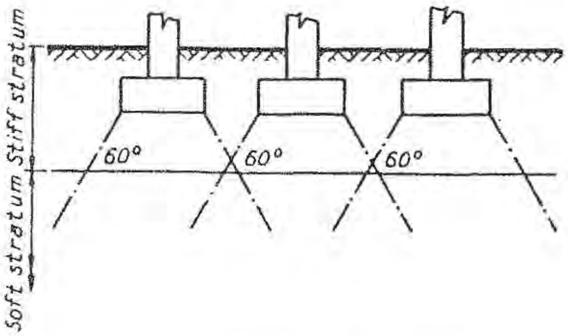


FIG. 4.3. CLOSE-SPACED FOUNDATIONS

forming a natural raft should be allowed for by the following procedure—

Pressure on surface of buried soft stratum $= q$

$$= \frac{W - p_s}{A} \quad . \quad . \quad (4.3)$$

where W = total load at base of foundation

p_s = perimeter shearing resistance

= peripheral area of stiff clay × shear strength of stiff clay

A = base area of foundation.

The value of q should not exceed the safe bearing capacity of the soft clay. Also, as in all cases of foundations on clay soils, the settlement of the foundation due to consolidation within both the stiff and the soft strata should be considered. The peripheral area of stiff clay is obtained by multiplying the peripheral length of the foundation by the depth of stiff clay *below* foundation level. It is inadvisable to allow for any transfer of load from the foundation sides to the stiff clay because shrinkage of the soil or of the foundation concrete, or a combination of both, will open up a gap between the soil and the concrete. If the zone of soil affected by seasonal moisture content changes extends below foundation level, cracking of the soil in the dry season will destroy the perimeter shearing resistance. Therefore, the latter should be calculated only over the thickness of the clay layer *below* the zone of seasonal moisture changes.

FOUNDATIONS CONSTRUCTED ON A THIN CLAY STRATUM

When foundations are constructed on a thin surface stratum of clay overlying a relatively rigid stratum, there may be a tendency for the thin layer to be squeezed from beneath the foundation, particularly if the soft layer is of varying thickness. Fig. 4.4 shows a foundation of width B on a thin clay layer overlying a stratum of different characteristics and appreciably higher bearing capacity, for example a sand layer. The net ultimate bearing capacity of the thin clay layer is given by the formulae—

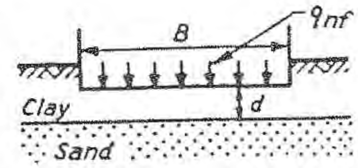


FIG. 4.4. FOUNDATION ON THIN CLAY LAYER

For a strip foundation of width B:

$$q_{nf} = \left(\frac{B}{2d} + \pi + 1\right)c \quad \text{for } \frac{B}{d} \geqslant 2 \quad . \quad . \quad (4.4)$$

For a circular foundation of diameter B:

$$q_{nf} = \left(\frac{B}{3d} + \pi + 1\right)c \quad \text{for } \frac{B}{d} \geqslant 6 \quad . \quad . \quad (4.5)$$

For smaller values of B/d than those given above, q_{nf} can be obtained from equations 2.11 and 2.12, i.e. the formulae for a thick clay layer.

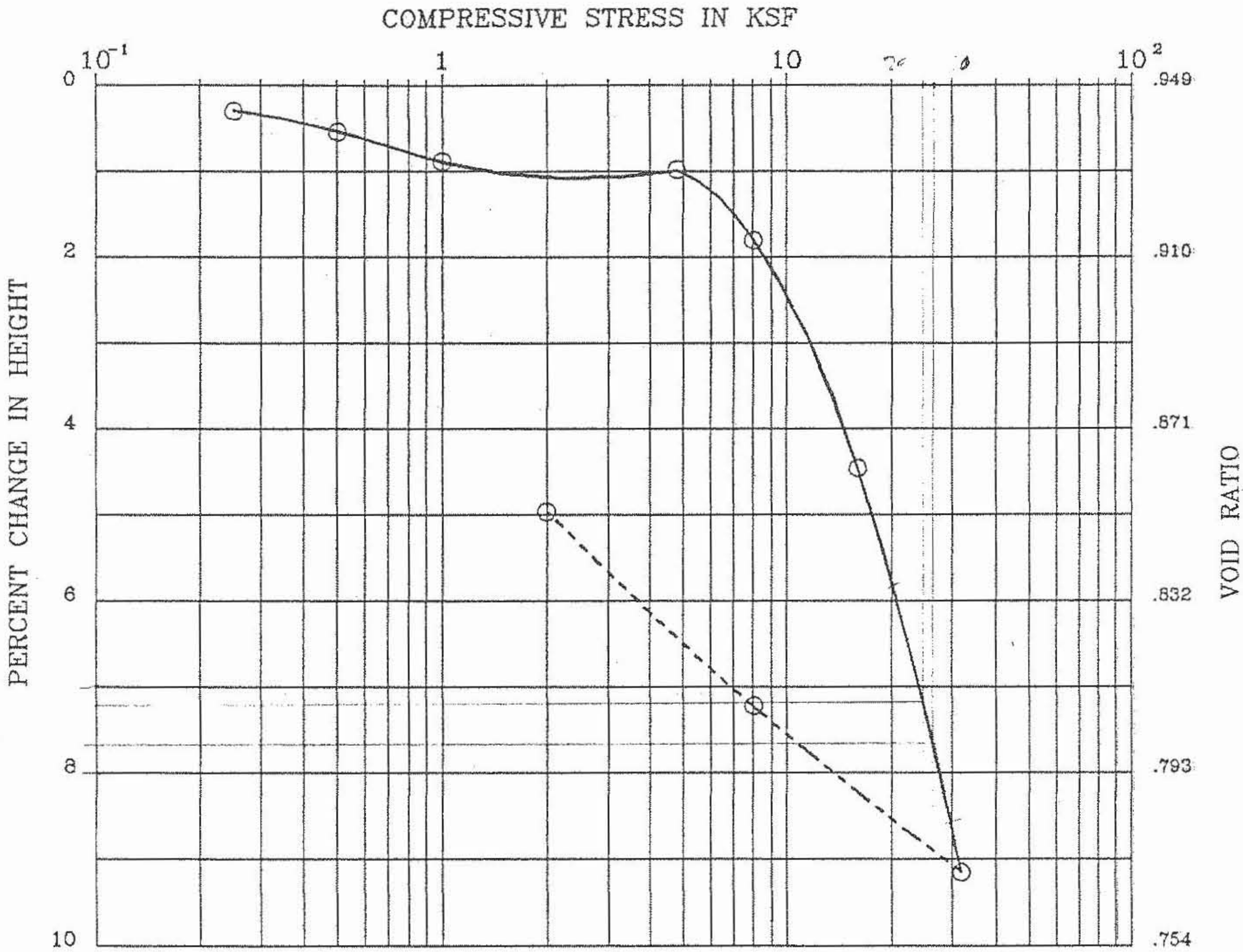
It should also be noted that, with a thin clay layer, plastic deformation resulting from overstressing begins at a lower foundation pressure than with a thick clay layer. For both strip and circular foundations, the maximum shear stress induced in the clay stratum is approximately $\frac{1}{2}q_n$.

Spread Foundations Carrying Eccentric Loading

Examples of foundations subject to eccentric loading are column foundations to tall buildings where wind pressures cause appreciable bending moments at the base of the columns, foundations of stanchions carrying brackets supporting travelling crane girders, and the foundations of retaining walls.

The pressure distribution below eccentrically loaded foundations is assumed to be linear as shown in Fig. 4.5 (*a*), and the maximum pressure must not exceed the maximum pressure permissible for a centrally loaded foundation. For the pad foundation shown in Fig. 4.5 (*a*), where the resultant falls within the middle third of the base,

$$\text{Maximum pressure} = q_{max} = \frac{W}{BL} + \frac{My}{I} \quad . \quad . \quad . \quad (4.6)$$

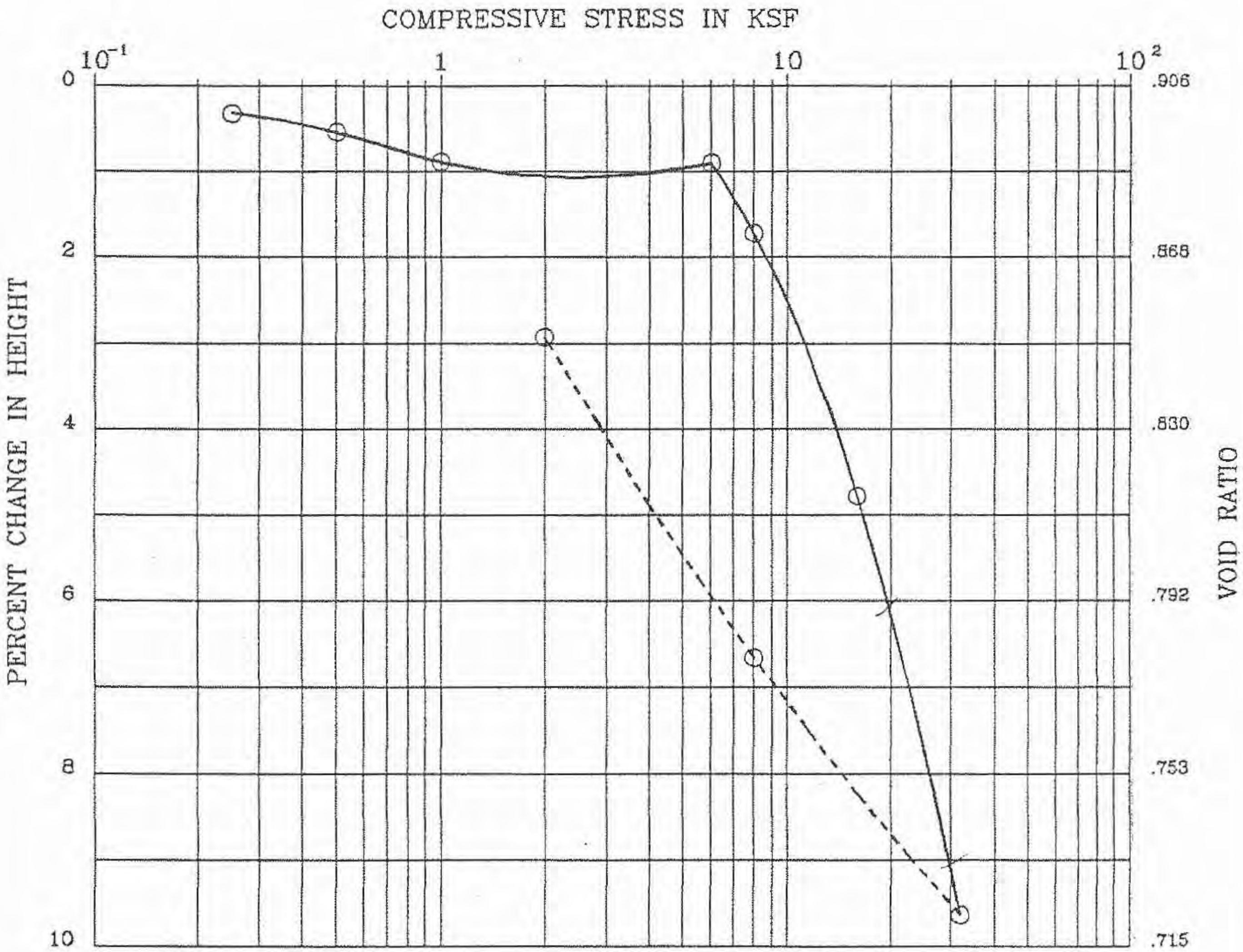


BORING	:	DT-C, B-1	DESCRIPTION	:	silty CLAYSTONE, yellow brn (CH)
DEPTH (ft)	:	8	LIQUID LIMIT	:	76
SPEC. GRAVITY	:	2.79	PLASTIC LIMIT	:	45

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	28.1	89.4	83	.949
FINAL	30.5	94.1	100	.852

Remark : July 1990

Project ESK-101A	Kettleman
Wahler Associates	CONSOLIDATION TEST Figure No.



BORING : DT-A, B-2
DEPTH (ft) : 5
SPEC. GRAVITY : 2.84

DESCRIPTION : silty CLAYSTONE, yellow brn (CH)
LIQUID LIMIT : 82
PLASTIC LIMIT : 54

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	27.3	93.1	86	.906
FINAL	29.9	95.9	100	.851

Remark : July 1990

Project ESK-101A	Kettleman
Wahler Associates	CONSOLIDATION TEST Figure No.

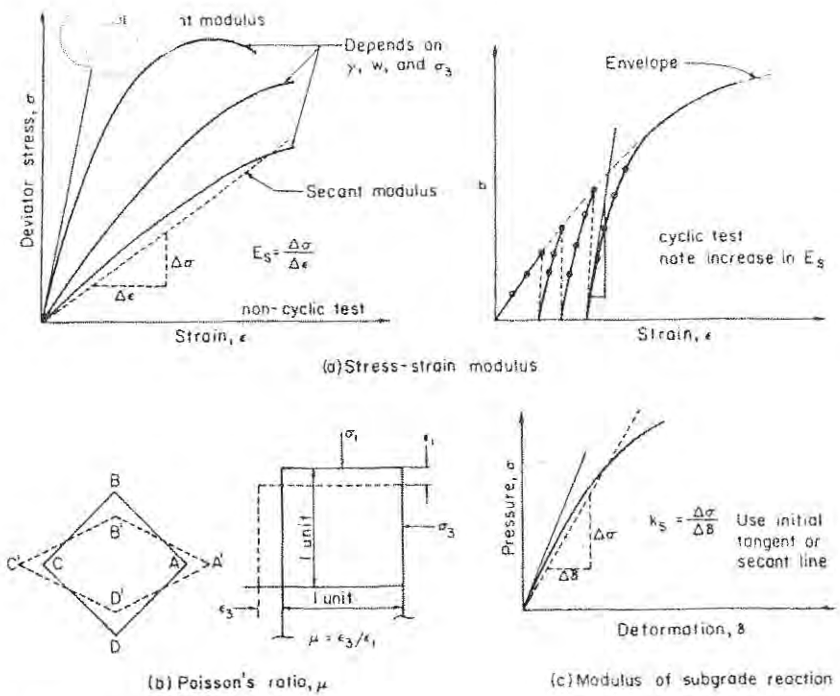


Figure 2-8. Elastic properties of soil.

The stress-strain modulus is computed from mechanics of materials (refer to Fig. 2-8*a*; typical values in Table 2-3) as

$$E_s = \frac{\text{stress}}{\text{strain}} = \frac{\sigma}{\varepsilon} \tag{d}$$

Poisson's ratio is defined as the ratio of lateral strain ε_3 to longitudinal strain ε_1 when the applied stress is uniaxial (refer to Fig. 2-8*b*; typical values in Table 2-4).

$$\mu = \frac{\varepsilon_3}{\varepsilon_1} \tag{e}$$

The modulus of subgrade reaction is defined as the ratio of stress to deformation (refer to Fig. 2-8*c*)

$$k_s = \frac{\sigma}{\delta} \tag{2-17}$$

The shearing modulus G, defined as the ratio of shear stress to shear strain, is related to E_s and μ as follows:

$$G = \frac{\text{shear stress}}{\text{shear strain}} = \frac{s}{\varepsilon_s} = \frac{E_s}{2(1+\mu)} \tag{f}$$

ndation Analysis to Design by J.E. Bowles

Table 2-3. Typical range of values for the static stress-strain modulus E_s for selected soils. Field values depend on stress history, water content, density, etc.

Soil	E_s ksi	E_s kg/cm²
Clay		
Very soft	0.05–0.4	3–30
Soft	0.2–0.6	20–40
Medium	0.6–1.2	45–90
Hard	1–3	70–200
Sandy	4–6	300–425
Glacial fill	1.5–22	100–1,600
Loess	2–8	150–600
Sand		
Silty	1–3	50–200
Loose	1.5–3.5	100–250
Dense	7–12	500–1,000
Sand and gravel		
Dense	14–28	800–2,000
Loose	7–20	500–1,400
Shales	20–2,000	1,400–14,000
Silt	0.3–3	20–200

2880 — 288000 ksf

Table 2-4. Typical range of values for Poisson's ratio μ

Type of soil	μ
Clay, saturated	0.4–0.5
Clay, unsaturated	0.1–0.3
Sandy clay	0.2–0.3
Silt	0.3–0.35
Sand (dense)	0.2–0.4
Coarse (void ratio = 0.4–0.7)	0.15
Fine-grained (void ratio = 0.4–0.7)	0.25
Rock	0.1–0.4 (depends somewhat on type of rock)
Loess	0.1–0.3
Ice	0.36
Concrete	0.15

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