OCCIDENTAL CHEMICAL CORPORATION

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ENGINEERING REPORT

FINAL

VOLUME II

APPENDIX A
CALCULATIONS

102nd STREET LANDFILL SITE

NIAGARA FALLS, NEW YORK

SEPTEMBER 9, 1995

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FLUOR DANIEL, INC.

MARLTON, NEW JERSEY

APPENDIX A

CALCULATIONS

APPENDIX A.1

APL Collection System

SUPPORTING ANALYSIS FOR APL COLLECTION SYSTEM DESIGN

1.0 BACKGROUND/ASSUMPTIONS

The basic elements of the selected remedy for the 102nd Street Landfill Site (Site) are defined by the site-specific Record of Decision (ROD). These elements include, a cap over the landfill, a perimeter slurry wall keyed into the underlying clay/till, an aqueous phase liquid (APL) collection system, and other elements. The purpose of an APL collection system, as stated in the ROD, is to create and maintain an inward gradient across the perimeter slurry wall at the Site, minimizing potential for off-site migration of APL through the perimeter slurry wall.

The general layout of the elements of the Remedial Design were presented in the Remedial Design Work Plan (RDWP, 1992) and subsequently indicated on drawing 594000-30K-01. These documents provide the data used in the calculations reported here. The geohydrologic data needed for these calculations were obtained from the RI and supporting Milestone Reports No. 2, 8, and 14.

The inward gradient across the perimeter slurry wall could be achieved by reducing the static ground water level inside the perimeter slurry wall to at least one (1) foot below the Niagara River elevation and the natural ground water levels outside the perimeter slurry wall. Once this level is attained, infiltration will consist of water from the following sources:

- Precipitation infiltration through the landfill cap,
- Ground water infiltration through the perimeter slurry wall,
- Ground water infiltration beneath the perimeter slurry wall, and
- Ground water migration upward from the bedrock formation through the confining clay/glacial till deposits

An equivalent volume of water to the infiltration listed above must be pumped to maintain these steady state water levels. Each of the water sources was evaluated to estimate steady state pumping requirements. Conservative assumptions were made to estimate maximum infiltration. The following assumptions were used in the calculations:

- The average width of the perimeter slurry wall is 3 feet.
- The average hydraulic conductivity of the perimeter slurry wall is $1x10^{-7}$ cm/sec.
- The average hydraulic conductivity of the underlying clay (Clay) is $1.6x10^{-8}$ cm/sec (Table 4.3, RI, 1990).
- The average hydraulic conductivity of the underlying glacial till (Till) is $6.5x10^{-8}$ cm/sec (Table A.1-1).

- The bottom 2 feet of the Till has gravel or a high permeability base which is not representative of the Till at the Site and therefore will not be considered as part of the confining layer.
- The average river elevation is 563.6 feet above mean sea level (MSL) and significant extended downward variations from this level do not occur (Table 6, Milestone Report No. 8).
- A minimum of 1-foot differential will be maintained across the perimeter slurry wall. This essentially means water level elevations inside the perimeter slurry wall will be maintained at approximately 562.6 ft MSL.
- The area enclosed within the perimeter slurry wall is approximately 1.052 million square feet (Drawing # 594000-30K-01).

The character of the Till does not exhibit significant lateral variation across the Site. However, locally the Till does appear to have a coarse granular base (Milestone Report No. 14, 1987). Also, it should be noted that because of the basal portions of the Till are possibly hydraulically connected to the bedrock, local hydraulic conductivity may appear greater in wells installed at the interface between the Till and bedrock than in the Till unit itself (Milestone Report No. 14, 1987). Therefore, hydraulic conductivities measured at the interface between Till and bedrock were not used in obtaining a geometric average for the Till (Table A.1-1).

TABLE A.1-1: Summary of Hydraulic Conductivity Data in Till

Well/Borehole Number	Tested Interval (Ft. BGS)	Hydraulic Conductivity (cm/sec.)	Analysis Method	Data Source	Relationship to Bedrock Interface	Used in Calculations
OW2-79	23.7-30.5	$2.7x10^{-7}$	A (field)	CRA, 1983	screen bottom 1 foot above bedrock	Yes
OW3-79	25.0-26.5	$6.7x10^{-9}$	C (lab)	CRA, 1980	5 feet above bedrock	Yes
OW6-79	34.9-40.5	$4.5x10^{-5}$	A (field)	CRA, 1983	screen bottom at bedrcok interface	No
OW8-79	35.6-41.5	$1.3x10^{-4}$	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW8-79	29.0-30.5	$1.5x10^{-8}$	C (lab)	CRA, 1980	11 feet above bedrock	Yes
OW9-79	24.1-30.5	$5.8x10^{-7}$ / $2.3x10^{-5}$	A / A (field)	CRA, 1983/87	screen bottom at bedrock interface	No
OW10-79	35.0-36.5	$1.5x10^{-8}$	C (lab)	CRA, 1980	12 feet above bedrock	Yes
OW11-79	34.1-40.0	$4.9x10^{-7}$	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW13-79	31.8-38.0	$6.1x10^{-5}$	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW14-79	39.2-46.2	$1.2x10^{-6}$	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW15-79	30.9-36.5	$1.4x10^{-6}$	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW16-79	32.8-39.5	$4.2x10^{-5}$	A (field)	CRA, 1983	top of screen in alluvium	No
OW17-79	42.3-48.0	$1.9x10^{-4}$	A (field)	CRA, 1983	screen bottom at bedrock interface	No

OW18-79	28.9-35.3	8.1×10^{-6}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW19-79	24.7-32.0	4.3×10^{-7}	A (field)	CRA, 1983	screen bottom 1 foot above bedrock	Yes
OW20-79	38.5-44.2	2.4×10^{-7}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW21-79	35.3-42.0	3.3×10^{-4}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW22-79	36.9-42.7	3.2×10^{-6}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW24-79	38.3-44.5	1.0×10^{-4}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW62-87	27.1-33.3	1.0×10^{-5}	A (field)	CRA, 1987	screen bottom at bedrock interface	No
B-22	21.0-38.5	5.6×10^{-6}	B (field)	Wehran, 1981	screen bottom 1.1 feet above bedrock (bentonite seal)	Yes
B-24	29.5-39.5	3.0×10^{-7}	B (field)	Wehran, 1981	screen bottom 1.1 feet above bedrock (bentonite seal)	Yes
B-29	30.5-37.5	1.0×10^{-7}	B (field)	Wehran, 1981	screen bottom 1.1 feet above bedrock (bentonite seal)	Yes
B-31	34.5-40.0	7.8×10^{-9}	B (field)	Wehran, 1981	screen bottom 1 foot above bedrock (bentonite seal)	Yes
B-34D	28.5-39.0	6.8×10^{-9}	B (field)	Wehran, 1981	screen bottom 1 foot above bedrock (bentonite seal)	Yes
A-252	9.0	5.7×10^{-8}	C (lab)	CRA, 1987	?	Yes
E-286	19.9	7.4×10^{-8}	C (lab)	CRA, 1987	?	Yes

I-136	13.5	6.0×10^{-8}	C (lab)	CRA, 1987	?	Yes
Geometric mean value (at least 1 foot away from the bedrock)		6.5×10^{-8} 1.4×10^{-7} 2.7×10^{-8}	Field + Lab Field Only Lab Only			

NOTES

1. Data from PFAR were not available when PER calculations were done. PFAR data will be included in the FER calculations.

2. Method Equation

A $K = \dfrac{r^2 \ln(L/R)}{2 L T_o}$ $L/R > 8$ for isotropic conditions

B In Situ - Equation Unknown

C Laboratory Tests

Where: L = length of saturated interval (cm)
r = radius of opening (pipe) where water levels were monitored (cm)
R = radius of borehole at interval "L" (cm)
T_o = elapsed time where $\dfrac{H-h_t}{H-h_o} = 0.37$ (sec)
H = water level at equilibrium
H_o = initial water level when slug was introduced/removed
h_t = water level at time t
K = hydraulic conductivity (cm/sec)

2.0 STEADY STATE APL PUMPING REQUIREMENTS

- Infiltration Through the Cap
- Groundwater Infiltration Through the Perimeter Slurry wall
- Groundwater Infiltration Under the Perimeter Slurry Wall
- Groundwater Migration From Bedrock Formation
- Total Flow Into APL Collection Trench at Steady State

2.0 STEADY STATE APL PUMPING REQUIREMENTS

To achieve the purpose of the APL Collection System, it will be necessary to create and maintain inward gradient across the perimeter slurry wall. This could be achieved by reducing the static ground water level inside the perimeter slurry wall to approximately 1 foot below the Niagara River elevation and the natural ground water levels outside the perimeter slurry wall. Once this level is attained, infiltration will consist of water from the following sources:

- Precipitation infiltration through cap,
- Ground water infiltration through the perimeter slurry wall,
- Ground water infiltration beneath the perimeter slurry wall, and
- Ground water migration upward from the bedrock formation through the confining Clay/Till deposits.

An equivalent volume of water must be pumped to maintain steady-state water levels.

Infiltration Through Cap

Infiltration through the cap was estimated using the HELP (Hydrologic Evaluation of Landfill Performance) model (Schroeder et. al., 1988) for the landfill cap designs shown in Figure 1 ("Alternate A") and Drawing 594000-10S-02 ("Alternate B"). Both designs are similar except "Alternate A" incorporates a 12-inch silty clay layer under a 60-mil very low density polyethylene (VLDPE) or equivalent liner while "Alternate B" uses a prefabricated geosynthetic clay liner, with maximum permeability of $1x10^{-9}$ cm/sec, under a 40-mil very low density polyethylene (VLDPE). The "topsoil" was simulated as sandy loam, "select fill" was simulated as silty loam, and the geotextile/geonet layer was simulated as a one-inch layer of coarse sand with a hydraulic conductivity of $1x10^{-2}$ cm/sec. Soil properties were taken from Rawls et al. (1982) and HELP model documentation. A range of 10^{-5} to 10^{-6} cm/sec was used for the hydraulic conductivity of the silty clay layer.

The "Alternate B" membrane was assigned a leakage factor of 0.01 (maximum value of typical range), which corresponds to leaks/openings on an approximately 50-foot square grid. This is a conservative assumption which results in overestimating the seepage through the cap. The 60-mil VLDPE liner ("Alternate A") was assumed to develop fewer leaks/openings and therefore was assigned a leakage factor of 0.003 which corresponds to leaks/openings on an approximately 100-foot square grid. Average monthly temperature data reported in the RI were used as input to the HELP model. Other parameters are shown on the HELP model output (included as Appendix A.2). The area enclosed by the perimeter slurry wall is approximately 1,052,000 square feet.

For "Cap Section - Alternate A" (Figure 1), the HELP model output shows infiltration of 74,600 ft^3/year [1530 gallons per day (gpd)] through the cap using a hydraulic conductivity of 10^{-5} cm/sec for the silty clay layer below the 60-mil VLDPE liner. If 10^{-6} cm/sec hydraulic conductivity of silty clay layer could be achieved, the infiltration through the cap is estimated to be 155 gpd. In comparison, if the leaks/openings in the 60-mil VLDPE liner occur on an approximately 50-foot square grid, then the infiltration is estimated to be as high as 4760 gpd.

For "Cap Section - Alternate B" (Drawing 594000-10S-02), the HELP model estimates infiltration to be 118 gpd (5780 ft^3/year) through the cap.

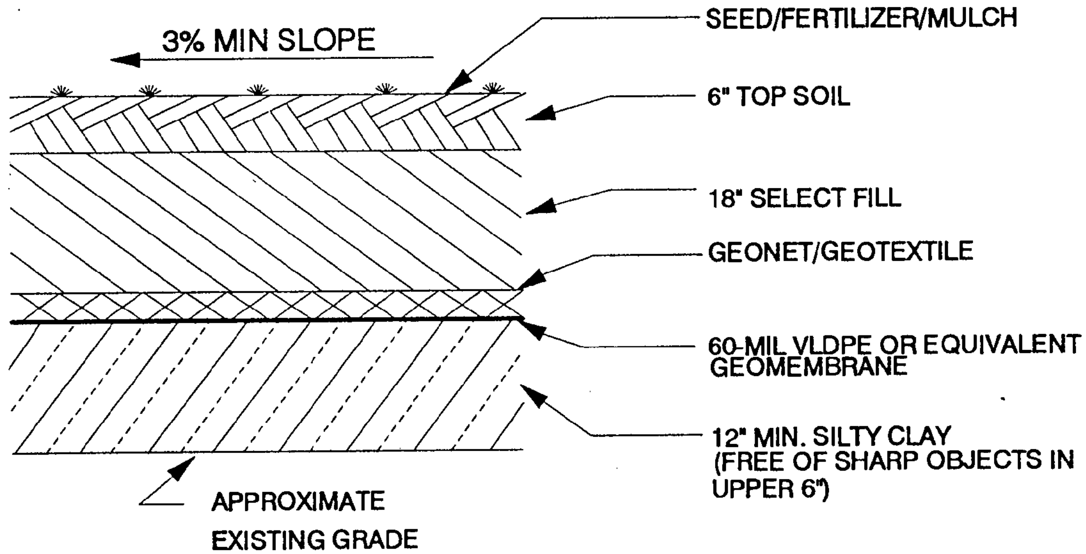
Based on the cap design calculations presented above, the HELP model analysis shows that the "Alternate B" cap design would allow less infiltration than the "Alternate A" cap design. Additionally, it may be difficult to install a 12-inch silty clay layer ("Alternate A") and obtain the desired hydraulic conductivity of 10^{-6} cm/sec, which is required to obtain performance essentially equal to that of "Alternate B" cap design.

<u>Ground Water Infiltration Through the Perimeter Slurry Wall</u>

The saturated area of the perimeter slurry wall through which water may infiltrate varies around the Site with the elevations of the top of the confining layer and the water table. An inward head difference of 1-foot across the slurry wall on the south side (Niagara River) is the controlling factor in selecting water levels in the encapsulated landfill area because the river is on the downgradient side. Water levels inside the perimeter slurry wall are designed to be maintained at 562.6 ft MSL.

Average water level data, from the RI report, on the east, west, and south sides were used to determine the steady state gradients. In the north, the top of the Clay layer is at approximately 566 to 568 ft MSL, i.e., above the water level to be maintained in the area between the curtain wall and the perimeter slurry wall. Once the water level is lowered by pumping APL, the top of the Clay in the north will be in the unsaturated zone. It is assumed that water levels outside of the perimeter slurry wall along the north side will not change appreciably, from the data presented in the RI, once a ground water control system (if required) is in place. Therefore, the average head difference in the north side was set equal to the average saturated thickness of the Fill and Alluvium presented in the RI data.

A hydraulic conductivity of $1x10^{-7}$ cm/sec is assumed for the 3-foot wide slurry wall. Infiltration is estimated using Darcy's Law:



NOT TO SCALE

FIGURE 1	LANDFILL CAP SECTION - ALTERNATE A	OXYCHEM/OLIN REMEDIAL DESIGN 102ND STREET LANDFILL SITE NIAGARA FALLS, NEW YORK
FLUOR DANIEL		

$$Q = K\,i\,A = K\left(\frac{\Delta h}{W}\right) A$$

where:

A	=	saturated area of the perimeter slurry wall,
i	=	hydraulic gradient across the perimeter slurry wall,
K	=	hydraulic conductivity of the perimeter slurry wall (1×10^{-7} cm/sec),
Q	=	infiltration through the perimeter slurry wall,
W	=	perimeter slurry wall thickness (3 feet), and
Δh	=	average head difference across the perimeter slurry wall.

PERIMETER SLURRY WALL SIDE	AREA (ft^2)	AVERAGE HEAD DIFFERENCE (ft)	INFILTRATION (gpd)
South	29,200	1.0	21
West	14,000	5.2	51
East	6,200	4.5	20
North	8,500	5.1	31
TOTAL	**57,900**		**123**

Ground Water Infiltration Under the Perimeter Slurry Wall

The perimeter slurry wall will be keyed into the confining Clay/Till layer. Flow paths may be created beneath the wall and into the encapsulated landfill area, driven by the head differential across the perimeter slurry wall. The infiltration rate for this source is estimated using Darcy's Law:

$$Q = K\,i\,A = K\left(\frac{\Delta h}{L_f}\right)(L_s W_f) = K\,\Delta h\,L_s\left(\frac{W_f}{L_f}\right)$$

where:

A	=	cross-sectional area for flow along the perimeter slurry wall,
i	=	hydraulic gradient across the perimeter slurry wall,
K	=	hydraulic conductivity of Clay or Till,
L_f	=	length of flow line (across which head difference is used),
L_s	=	length of the perimeter slurry wall,
Q	=	infiltration under the perimeter slurry wall,
W_f	=	width of flow zone under the perimeter slurry wall, and
Δh	=	head difference across the perimeter slurry wall.

Average flowlines have to traverse the flow width twice, once going down and once coming up. Therefore, a conservative estimate of 1/2 for the ratio of flow width to flow length is used. Calculations for infiltration under the perimeter slurry wall are summarized in the following table.

SIDE	CONFINING LAYER	PERIMETER SLURRY WALL LENGTH (ft)	AVERAGE HEAD DIFFERENCE (ft)	Infiltration (gpd)
South	clay till	850 870	1 1	0.1 0.6
West	clay till	350 590	6.4 4.5	0.4 1.8
East	clay till	170 395	5.6 4.0	0.2 1.1
Northeast	clay	1660	5.1	1.4
TOTAL		**4885**		**5.5**

Ground Water Migration From Bedrock Formation

At present, the piezometric levels in the bedrock formation are lower than that of the water table. However, once water levels within the encapsulated landfill area are lowered, the bedrock formation will have a higher head than the water table within the encapsulated landfill area. This will result in a potential for upward flow through the confining Clay/Till layer. To calculate water levels along the northen part of the landfill, mounding potential within the encapsulated landfill area is estimated next.

MAXIMUM MOUNDING WITHIN THE ENCAPSULATED LANDFILL AREA

Maximum groundwater mounding within the encapsulated landfill area is estimated using the equation (Bear, 1979):

$$h_{max} = [\frac{L^2 I}{4K} + h_o^2]^{0.5}$$

where:

h_{max} = maximum hydraulic head (ft),
h_o = hydraulic head at southern section of the slurry wall (ft),
I = inflow rate per unit area (ft^3/ft^2-day),
K = hydraulic conductivity of Alluvium/Fill (ft/day), and
L = 2 times the distance between the south APL trench and north perimeter slurry wall (ft).

Using 800 feet for the distance between the south APL trench and north perimeter slurry wall, $2.2x10^{-4}$ cm/sec for the hydraulic conductivity of Alluvium/Fill, 12 feet for the hydraulic head at the south APL trench (measured from the confining layer), and a total inflow of 400 gpd over 1,052,000 square feet area, h_{max} is calculated to be 12.53 feet. Therefore, the maximum groundwater mound within the area contained by the perimeter slurry wall will be 0.53 feet. Actual mounding may be less because of APL Collection Trench on the east side and the west side.

INFLOW FROM BEDROCK

Migration of ground water from bedrock is calculated using Darcy's Law:

$$Q = K\, i\, A$$

where:

A = area enclosed within the perimeter slurry wall,
i = hydraulic gradient across the confining layer,
K = effective hydraulic conductivity of the confining layer, and
Q = infiltration through the confining layer.

The landfill site is partly underlain by Till and partly underlain by the Clay layer followed by the Till layer. Since resistance to flow is in series, effective hydraulic conductivity of the confining layer (K) is calculated by:

$$\frac{L}{K} = \frac{L_c}{K_c} + \frac{L_t}{K_t}$$

where:

L = total thickness of confining layer,
L_c = thickness of Clay layer,
L_t = thickness of glacial till contributing to the confining layer,
K_c = hydraulic conductivity of Clay layer (1.6×10^{-8} cm/sec), and
K_t = hydraulic conductivity of Till layer (6.5×10^{-8} cm/sec).

Calculation of effective hydraulic conductivity of the confining layer and ground water migration rate from bedrock is summarized in the following table. Due to possible granular materials at the base of the Till, effective thickness of the Till was assumed to be 2 feet less than the Till thickness reported in the RI. In the central area, where till thickness is less than 4 feet, it was assumed that till will not act as a confining layer due to possible higher hydraulic conductivity. In that area, only the clay layer will act as a confining layer.

LOCATION	AREA (ft^2)	CONFINING LAYER THICKNESS (ft)		EFFECTIVE K (cm/sec)	AVERAGE HEAD DIFFERENCE (ft)	LEAKAGE (gpd)
		CLAY	TILL			
Northwest corner	165,000	8.5	11	2.8×10^{-8}	2.2	11.1
North central to eastern area	345,000	11	7	2.3×10^{-8}	2.2	20.6
Southwest and south-east corners	400,000	0	13	6.5×10^{-8}	1.9	80.6
Central area	70,000	3	0	1.6×10^{-8}	2.0	15.8
South central area	72,000	4	5	2.6×10^{-8}	1.9	8.4
TOTAL	**1,052,000**					**136.5**

Total Flow Into APL Collection Trench at Steady State

Sources of inflow to the encapsulated landfill area and related rates based on the "Alternate B" cap design are as follows:

SOURCE	INFLOW RATE (gpd)
Infiltration through cap	118[1]
Leakage through the perimeter slurry wall	123
Leakage under the perimeter slurry wall	5.5
Migration from bedrock	136.5
TOTAL	**383**

Note (1): Infiltration shown is for "Alternate B" cap design.

A conservative estimate of total inflow of approximately 400 gallons of water per day into the encapsulated landfill area is anticipated at steady state. These numbers are based on an annual average basis. The actual daily inflow will vary with fluctuations in river elevation, seasonal water table, and bedrock head potential, and changes in rainfall and climatologic conditions.

The total inflow rate is most sensitive to the permeability of the Till and Clay layers. For example, if the effective permeability of the Till is an order of magnitude higher than the value used in this report, then the total flow inflow rate to the APL collection trench will be approximately 1150 gallons per day. Conversely, if the effective permeability of the Till is an order of magnitude lower than was used in this report, then the total inflow rate of the APL collection trench will be approximately 285 gallons per day.

3.0 INITIAL APL VOLUME TO BE PUMPED

3.0 INITIAL APL VOLUME TO BE PUMPED

Volume of APL to be pumped initially to lower the water table to achieve steady state ground water levels is calculated from:

APL Volume = (Area) x (Average Head Difference) x (Total porosity - Field capacity)

It is assumed that after the perimeter slurry wall and cap are constructed and the water table is lowered, Alluvium and Fill should gravity drain to field capacity. This estimate does not include the steady state flow to the APL collection trench calculated in Section 2.0.

Average head difference between initial water level and steady state water level is estimated to be 4.0 to 5.5 feet. Total porosity is taken as 45 percent and field capacity is taken as 20 percent, which gives a specific yield of 25 percent. Total APL volume to be pumped to achieve steady state is then estimated to be 7,900,000 to 10,800,000 gallons.

Figure 2 shows the time required to reach steady state conditions as a function of the initial pumping rate. Figure 2 takes into consideration the infiltration to the encapsulated landfill area during initial pumping stages. The upper curve in Figure 2 represents the case where 10,800,000 gallons of APL needs to be pumped initially, while the lower curve represents 7,900,000 gallons of APL to be pumped initially. Once steady state conditions are achieved, water levels within the encapsulated area will be nearly level with a very small gradient towards the APL collection trenches.

If initial pumping rate is set at 20,000 gallons per day for 7 days a week, it will require 13 to 19 months of pumping to reach the steady state conditions. However, if APL is pumped only 5 days a week at 20,000 gallons per day, it will take from 19 to 26 months to achieve steady state conditions. If steady state conditions are desired within 12 months, then pumping at 30,000 gallons per day for 7 days a week is required. Although, it may take approximately two years to reach the steady state conditions when pumped at 20,000 gallons per day 5 days a week, inward gradients will be established during the early stages of pumping.

4.0 APL COLLECTION TRENCH LOCATIONS

4.0 APL COLLECTION TRENCH LOCATIONS

Anticipated groundwater mounding within the enclosed landfill area is minimal. Furthermore, once the water level is lowered, the confining Clay layer in the north side will be exposed. Therefore, the trench along the full length of the perimeter slurry wall on the east side is not required. Similarly, the trench along the full length of the perimeter slurry wall on the west side is not required. Location of the APL collection trench is shown on the Drawing 594000-30K-01.

5.0 SUPPORTING CALCULATIONS FOR APL PUMP SELECTION

Initial Pumping Rates

All 4 Wet Wells : 20,000 gpd

Maximum for WW-1 : 8,000 gpd

Maximum for WW-1 + WW-2 : 15,000 gpd.

Pressure drop from WW-1 to WW-2 (Max. 750 ft)

$$8{,}000 \ \frac{gal}{day} = \frac{8{,}000}{7.48} \times \frac{1}{3600 \times 24} = 0.0124 \ \frac{ft^3}{sec}$$

$$\text{Velocity} = \frac{0.0124 \ ft^3/sec}{\frac{\pi}{4}\left(\frac{1.61}{12}\right)^2 ft^2} = 0.88 \ \frac{ft}{sec}$$

$$Re = \frac{D v \rho}{\mu} = \frac{(1.61 \times 2.54 \, cm)\left(0.88 \times 30.48 \ \frac{cm}{sec}\right)\left(1 \ \frac{gm}{cm^3}\right)}{(1 \ cp)\left(0.01 \ \frac{gm/cm\text{-}sec}{cp}\right)}$$

$$= 1.09 \times 10^4$$

Table 5-7 of Perry's Handbook provides following surface roughness factors (ϵ)

Commercial steel – 0.0018 inch

Cast iron – 0.01 inch

Using ϵ = 0.01 inch

$$\epsilon/D = \frac{0.01}{1.61} = 0.0062$$

Fanning friction factor (from Perry's Handbook) = f = 0.0098

$$\Delta P = \left(\frac{4fL}{D}\right)\left(\frac{V^2}{2g_c}\right) = \left(\frac{4 \times 0.0098 \times 750}{1.61/12}\right)\left(\frac{0.88 \times 0.88}{2 \times 32.2}\right)$$

$$= 2.62 \text{ ft of water head}$$

<u>Presssure drop from WW-2 to Central "T" for Loadout Facility at the Niagara River (Max. 400 ft)</u>

$$15,000 \frac{gal}{day} = 0.02325 \frac{ft^3}{sec}$$

$$\text{Velocity} = \frac{0.02325}{\frac{\pi}{4}\left(\frac{1.61}{12}\right)^2} = 1.65 \text{ ft/sec}$$

$$Re = \frac{(1.61 \times 2.54)(1.65 \times 30.48)(1)}{1 \times 0.01} = 2.06 \times 10^4$$

Fanning friction factor $= f = 0.0092$

$$\Delta P = \left(\frac{4fL}{D}\right)\left(\frac{V^2}{2g_c}\right) = \left(\frac{4 \times .0092 \times 400}{1.61/12}\right)\left(\frac{1.65 \times 1.65}{2 \times 32.2}\right)$$

$$= 4.64 \text{ ft of water head}$$

<u>Pressure Drop from Central "T" to the Loadout Facility (200 ft. max.)</u>

$$20,000 \text{ gal/day} = 0.031 \frac{ft^3}{sec}$$

$$\text{Velocity} = \frac{0.031}{\pi/4 \left(\frac{1.61}{12}\right)^2} = 2.19 \text{ ft/sec.}$$

$$Re = \frac{(1.61 \times 2.54)(2.19 \times 30.48)(1)}{1 \times 0.01} = 2.73 \times 10^4$$

Fanning friction factor $= f = 0.009$

$$\Delta P = \left(\frac{4 f L}{D}\right)\left(\frac{V^2}{2 g_c}\right) = \left(\frac{4 \times 0.009 \times 200}{1.61/12}\right)\left(\frac{2.19 \times 2.19}{2 \times 32.2}\right)$$

$$= 4.0 \text{ ft}$$

<u>Additional head losses through fittings & Valves</u>

	Velocity Heads
6 ball valves (@ 0.06)	0.36
1 globe valve	9.5
2 check Valve (@ 70)	140.0
10 elbows (@ 0.75)	7.5
	157.36

$$\text{Total head loss} = 157.36 \times \frac{2.19 \times 2.19}{2 \times 32.2}$$

$$= 11.7 \text{ ft}$$

Head loss across flow meter $\approx$ 30 ft

TOTAL HEAD REQUIRED FROM PUMP

Head loss in pipes (2.6 + 4.6 + 4.0)	=	11.2 ft
Head loss through fittings and valves	=	11.7 ft
Head loss through flow meter	=	30.0 ft
Elevation difference	≈	50.0 ft
Total		102.9 ft

In the transition region (N_{Re} from 2000 to 3000), the velocity profile becomes more blunt and the ratio V/u_{max} increases (see Fig. 5-12). Velocity profile curves are given by Patel and Head [*J. Fluid Mech.*, **38**, part 1, 181–201 (1969)] for flow in smooth pipes in the Reynolds number range of about 1500 to 10,000. At higher Reynolds numbers, the flow is generally fully turbulent, and the velocity profile in **smooth-wall pipes** is characterized by a laminar boundary layer ($y^+ < 5$), a turbulent core ($y^+ > 30$), and a buffer layer in between. The local velocity is given by the following relationships:

For the laminar boundary layer,

$$u^+ = y^+ \qquad \text{for } y^+ < 5 \tag{5-51a}$$

For the buffer layer,

$$u^+ = -3.05 + 5.00 \ln y^+ \qquad \text{for } 5 < y^+ < 30 \tag{5-51b}$$

For the turbulent core,

$$u^+ = 5.5 + 2.5 \ln y^+ \qquad \text{for } y^+ > 30 \tag{5-51c}$$

For **rough-wall pipes**, the local velocity in the turbulent core is given by

$$u^+ = 8.5 + 2.5 \ln \frac{y}{e} \qquad \text{for } y^+ > 30 \tag{5-51d}$$

where $u^+ = u/u^*$; u = local velocity, ft./sec., at distance y ft. from the pipe wall; $u^* = \sqrt{\tau_0 g_c/\rho}$ (called friction velocity); τ_0 = wall shear stress $(D\,\Delta p/4L)$, lb. force/sq. ft.; g_c = dimensional constant, 32.17 (lb.)(ft.)/(lb. force)(sec.²); ρ = fluid density, lb./cu. ft.; Δp = pressure drop, lb. force/sq. ft.; D = inside pipe diameter, ft.; L = pipe length, ft.; $y^+ = yu^*\rho/\mu$, dimensionless; μ = fluid viscosity, lb./(ft.)(sec.); e = height of wall roughness, ft. For further details, see Knudsen and Katz, *op. cit.*, pp. 154–169, and Cremer and Davies, *op. cit.*, vol. 4, p. 401.

Equations describing the distribution of residence time for **turbulent flow** in pipes are given by Danckwerts, *loc. cit.*

Velocity Distribution, Other Shapes. For velocity profiles under laminar- and turbulent-flow conditions in annuli, between infinite parallel planes, and in other non-circular cross sections, see Knudsen and Katz, *op. cit.*; Purday, "Mechanics of Viscous Flow," Chap. II, Dover, New York, 1949; Rouse, "Advanced Mechanics of Fluids," p. 219, Wiley, New York, 1959; Goldstein, "Modern Developments in Fluid Dynamics," vol. 2, pp. 359–360, Oxford, London, 1938.

Analytically derived equations are presented by Straub, Silberman, and Nelson [*Trans. Am. Soc. Civil Engrs.*, **123**, 685–714 (1958)] for **laminar flow** through a variety of **open-channel** cross sections, including semicircular, rectangular, triangular, elliptical, trapezoidal, etc.

Experimentally determined velocity profiles are also presented by Straub *et al.* for **turbulent flow** in triangular troughs. Profiles for channels of various cross sections are given in O'Brien and Hickox, "Applied Fluid Mechanics," pp. 268–270, McGraw-Hill, New York, 1937, and Chow, "Open-channel Hydraulics," pp. 24–29, McGraw-Hill, New York, 1959.

Residence-time Distribution, Process Vessels. An extensive treatment of distribution of residence time and of dispersion in a variety of typical process vessels is given by Levenspiel and Bischoff, "Patterns of Flow in Chemical Process Vessels," in Drew, Hoopes, and Vermeulen, "Advances in Chemical Engineering," vol. 4, Academic, New York, 1963. The case of multiple stirred tanks in series is covered in detail by Stokes and Nauman [*Can. J. Chem. Eng.*, **48**, 723–725 (1970)]. Information on residence time and fluid mixing on commercial-scale sieve trays is given by Bell [*Am. Inst. Chem. Engrs. J.*, **18**, 498–505 (1972)].

Incompressible Flow. The flow can be considered to be incompressible if (1) the substance flowing is a liquid or (2) if it is a gas whose density changes within the system no more than 10 per cent. In this event, if the inlet density is employed, the resulting error in computed pressure drop will generally not exceed the uncertainty limits in the friction factor. In the event of larger changes in fluid density, *e.g.*, gases with large pressure drops, the more exact methods described under Compressible Flow (pp. 5–26 to 5–31) should be used.

General Formulas and Methods. The problem of finding one of the three quantities—rate of discharge, size of channel, pressure or head loss—when the other two are given is solved by substituting the data of the problem in an appropriate form of the mechanical energy balance (p. 5–18) after the term F, frictional loss of mechanical energy, has been evaluated. That part of F which arises from friction within the channel proper is considered below. The part due to fittings, bends, and the like, which often constitutes a major part of the friction, is discussed on pp. 5–32 to 5–38.

The **Fanning,** or **Darcy, equation,** Eq. (5-52), for steady flow in uniform **circular pipes** running full of liquid under isothermal conditions

$$F = \left(\frac{4fL}{D}\right)\frac{V^2}{2g_c} = \left(\frac{4fL}{D}\right)h_v = \left(\frac{4fL}{D}\right)\frac{G^2}{2g_c\rho^2}$$
$$= \frac{32fLw^2}{\pi^2\rho^2 g_c D^5} = \frac{32fLq^2}{\pi^2 g_c D^5} \tag{5-52}$$

gives the friction loss F in (ft.)(lb. force)/lb. of fluid flowing (or ft. of fluid flowing), where D = duct diameter, ft.; L = duct length, ft.; ρ = fluid density, lb./cu. ft.; V = fluid velocity, ft./sec.; h_v = velocity head $(V^2/2g_c)$, ft. of fluid flowing; G = mass velocity, lb./(sec.)(sq. ft.); w = weight rate of flow, lb./sec.; q = volumetric rate of flow, cu. ft./sec.; g_c = dimensional constant, 32.17 (lb.)(ft.)/(lb. force)(sec.²); f = **Fanning friction factor** (see below), dimensionless.

The pressure drop due to friction is $\Delta p = F\rho$, lb. force/sq. ft.

The **Fanning friction factor** f is a function of the Reynolds number N_{Re} and the roughness of the channel inside surface ϵ. One widely used correlation [Moody, *Trans. Am. Soc. Mech. Engrs.*, **66**, 671–684 (1944)], as shown in Fig. 5-26, is a plot of Fanning friction factor as a function of Reynolds number and relative roughness ϵ/D or ϵ''/D'', where ϵ = surface roughness, ft.; D = pipe inside diameter, ft.; ϵ'' = surface roughness, in.; and D'' = pipe inside diameter, in. Values of ϵ or ϵ'' for various materials are given in Table 5-7. Substitution of the equation for curve A, Fig. 5-26, into Eq. (5-52) yields Poiseuille's law for laminar flow ($N_{Re} \leq 2000$); see Table 5-13. Care must be exercised when values of f are taken from the literature, because the same name and symbol are sometimes used to denote various multiples of the f given by Fig. 5-26.

A rapid method of solving Eq. (5-52) for turbulent flow ($N_{Re} > 2000$) is to use the **alignment chart** in Fig. 5-27, which is based on curve D of Fig. 5-26 [Genereaux, *Chem. & Met. Eng.*, **44**, 241–248 (1937)]. If, for the value of N_{Re}, obtaining some other value of f, say f', is preferred, the quantity sought, as given by the chart, should be multiplied by the factors given in Table 5-8. A nomograph to find pressure drop, taking into account pipe surface roughness, and a nomograph to determine flow rate or pipe size if either is unknown and if the pressure drop is known, are given by Arnold [*Chem. Eng.*, **66**(11), 103–106 (1959)].

Table 5-7. Values of Surface Roughness for Various Materials*

Material	Surface roughness	
	ϵ, ft.	ϵ'', in.
Drawn tubing (brass, lead, glass, and the like)	0.000005	0.00006
Commercial steel or wrought iron	0.00015	0.0018
Asphalted cast iron	0.0004	0.0048
Galvanized iron	0.0005	0.006
Cast iron	0.00085	0.010
Wood stave	0.0006–0.003	0.0072–0.036
Concrete	0.001–0.01	0.012–0.12
Riveted steel	0.003–0.03	0.036–0.36

*Moody, *Trans. Am. Soc. Mech. Engrs.*, **66**, 671–684 (1944); *Mech. Eng.*, **69**, 1005–1006 (1947). Additional values of ϵ for various types or conditions of concrete, wrought iron, welded steel, riveted steel, and corrugated metal pipes are given in King and Brater, "Handbook of Hydraulics," 5th ed., pp. 6-11 and 6-12, McGraw-Hill, New York, 1963.

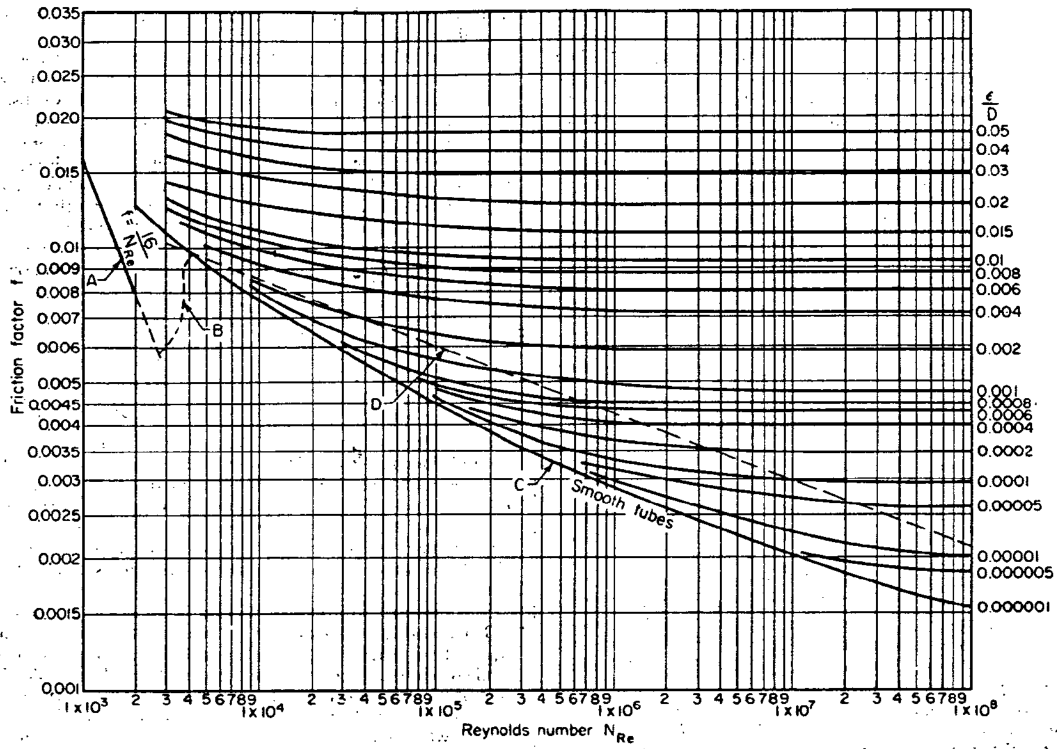


Fig. 5-26. Fanning friction factors. Reynolds number $N_{Re} = DV\rho/\mu$, where D = pipe diameter, ft.; V = velocity, ft./sec.; ρ = fluid density, lb./cu. ft.; μ = fluid viscosity, lb./(ft.)(sec.) = cp./1486. [*Based on Moody, Trans. Am. Soc. Mech. Engrs.*, **66**, *671 (1944)*.]

For rough estimates or checks, the velocity-head concept [Lapple, *Chem. Eng.*, **56**(5), 96–104 (1949)] can be applied to the first two forms of Eq. (5-52). The velocity head is $V^2/2g_c = h_v$ and the number of velocity-head losses in straight pipe is $4fL/D$. Typical values of h_v and L/D for 1 velocity-head loss are given in Table 5-9.

For cross sections other than circular of ducts running full or for open channels when the variation in depth is negligible, where

Table 5-8. Correction Factors for Fig. 5-27.

Quantity sought	$\frac{\Delta p}{L}$ or $\frac{\Delta h}{L}$	D_i	w or G
Factor	f'/f	$(f'/f)^{1/5}$	$\sqrt{f/f'}$

Table 5-9. Approximate Values of Velocity Head and Pipe Length Equivalent to One Velocity-head Loss

Fluid	Fluid velocity, ft./sec.	Velocity head, various units
Any fluid	8	1.0 ft. fluid
Water	4	0.1 lb. force/sq. in.
	12	1.0 lb. force/sq. in.
Air (125°F., 1 atm.)	50	0.5 in water
	70	1.0 in. water
	100	2.0 in. water

Pipe length equivalent to 1 velocity head loss

Fluid	L/D
Water	45 (f = 0.0055)
Air	55 (f = 0.0045)

Table 5-10. Coordinates for Liquids and Aqueous Solutions
For use with Fig. 5-27

	X	Y
Acetaldehyde	−0.3	3.7
Acetic acid, 100%	1.0	4.0
Acetic acid, 77%	2.6	3.8
Acetic anhydride	0.7	4.3
Acetone, 100%	0.9	3.4
Acetone, 35%	2.7	3.7
Ammonia, anhydrous	0.9	3.6
Ammonia, 26%	1.9	3.6
Aniline	2.5	3.4
Benzene	0.6	3.6
Butanol	2.6	2.6
Calcium chloride brine, 25%	2.6	4.2
Carbon disulfide	0.0	5.6
Carbon tetrachloride	0.7	6.0
Chloroform	0.0	6.0
Chlorosulfonic acid	1.5	5.6
Cyclohexanol	5.3	2.2
Diphenyl	0.0	3.5
Ethyl acetate	0.2	3.9
Ethyl alcohol, 95%	1.9	3.0
Ethyl alcohol, 45%	3.6	3.4
Ethyl chloride	0.2	4.3
Ethyl ether	−0.3	3.2
Ethylene glycol	3.5	2.9
Fluorocarbon F-11	0.0	6.2
Fluorocarbon F-12	−1.2	5.9
Fluorocarbon F-21	−0.4	5.9
Fluorocarbon F-22	−1.7	5.5
Fluorocarbon F-113	0.9	6.2
Formic acid	1.5	4.5
Glycerol, 100%	6.9	1.8
Glycerol, 50%	3.0	3.7
Hydrochloric acid, 31.5%	1.1	4.2
Linseed oil, raw	3.4	1.8
Mercury	See chart	
Methanol, 100%	0.8	3.3
Methanol, 40%	2.8	3.6
Methyl acetate	0.0	4.2
Methyl chloride	−0.8	4.3
Nitric acid, 95%	0.8	5.8
Nitric acid, 60%	1.5	4.8
Nitrobenzene	1.7	4.4
Octane	0.4	2.7
Phenol	2.4	3.4
Propionic acid	0.6	3.8
Sodium chloride brine, 25%	2.1	4.4
Sodium hydroxide, 50%	5.3	3.7
Sulfur dioxide	−0.2	6.1
Sulfuric acid, 110%	3.7	4.7
Sulfuric acid, 98%	3.5	4.8
Sulfuric acid, 78%	3.2	4.8
Tetrachloroethylene	0.3	6.2
Toluene	0.4	3.6
Tetrachloroethylene	0.1	5.9
Turpentine	1.1	3.1
Vinyl acetate	0.4	4.2
Water	2.0	4.2

Table 5-19. Additional Frictional Loss for Turbulent Flow through Fittings and Valves[k]

Type of Fitting or Valve	Additional Friction Loss, Equivalent No. of Velocity Heads, K
45-deg. ell, standard[a,b,c,g,i]	0.35
45-deg. ell, long radius[b]	0.2
90-deg. ell, standard[a,b,d,g,l,m]	0.75
Long radius[a,b,c,g]	0.45
Square or miter[m]	1.3
180-deg. bend, close return[a,b,g]	1.5
Tee, standard, along run, branch blanked off[f]	0.4
Used as ell, entering run[d,h]	1.0
Used as ell, entering branch[b,d,h]	1.0
Branching flow[f,h,l]	1[o]
Coupling[b,g]	0.04
Union[g]	0.04
Gate valve,[a,g,j] open	0.17
¾ open[p]	0.9
½ open[p]	4.5
¼ open[p]	24.0
Diaphragm valve,[n] open	2.3
¾ open[p]	2.6
½ open[p]	4.3
¼ open[p]	21.0
Globe valve,[g,j] bevel seat, open	6.0
½ open[p]	9.5
Composition seat, open	6.0
½ open[p]	8.5
Plug disk, open	9.0
¾ open[p]	13.0
½ open[p]	36.0
¼ open[p]	112.0
Angle valve,[a,g] open	2.0
Y or blowoff valve,[a,j] open	3.0
Plug cock[c] (Fig. 5-39) $\theta =$ 5°	0.05
10°	0.29
20°	1.56
40°	17.3
60°	206.0
Butterfly valve[c] (Fig. 5-40) $\theta =$ 5°	0.24
10°	0.52
20°	1.54
40°	10.8
60°	118.0
Check valve,[a,g,j] swing	2.0[q]
Disk	10.0[q]
Ball	70.0[q]
Foot valve[g]	15.0
Water meter,[m] disk	7.0[r]
Piston	15.0[r]
Rotary (star-shaped disk)	10.0[r]
Turbine-wheel	6.0[r]

[a] Flow of Fluids through Valves, Fittings, and Pipe, *Tech. Paper* 410, Crane Co., 1969.
[b] Freeman, "Experiments upon the Flow of Water in Pipes and Pipe Fittings," American Society of Mechanical Engineers, New York, 1941.
[c] Gibson, "Hydraulics and Its Applications," 5th ed., p. 250, Constable, London, 1952.
[d] Giesecke and Badgett, *Heating, Piping Air Conditioning*, 4(6), 443–447 (1932).
[e] Giesecke, *J. Am. Soc. Heat. Vent. Engrs.*, 32, 461 (1926).
[f] Gilman, *Heating, Piping Air Conditioning*, 27(4), 141–147 (1955).
[g] "Pipe Friction Manual," 3d ed., Hydraulic Institute, New York, 1961.
[h] Hoopes, Isakoff, Clarke, and Drew, *Chem. Eng. Progr.*, 44, 691–696 (1948).

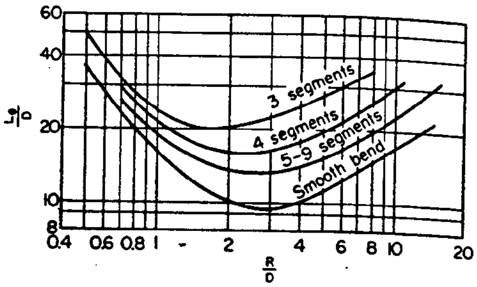


Fig. 5-42. Total friction loss in 90-deg. bends. [*Smooth bend: based on information from Freeman, "Experiments upon the Flow of Water in Pipes and Pipe Fittings," p. 173, A.S.M.E., New York 1941; Ito, J. Basic. Eng., 82, 131 (1960); Locklin, Trans. Am. Soc. Heating Ventilating Engrs., 56, 479 (1950); Snyder, Heating, Piping Air Conditioning, 7(1), 5 (1935). Segmental bends: from Locklin (loc. cit.).*]

curvature R to pipe diameter D, all in consistent units, is given in Fig. 5-42. The curve for smooth bends is based on various published data (see Fig. 5-42 for references) and represents most of the data with an uncertainty of probably ±25 per cent. The curves for segmental bends are based on few data. For a 45-deg. bend, total friction loss is about 65 per cent of the loss for a 90-deg. bend of a proportional number of segments and, similarly, for a 180-deg. bend, the loss is about 140 per cent of that for a 90-deg. bend, based on information presented by Conn, Colborne, and Brown [*Heating, Piping Air Conditioning*, 25(1), 201–205 (1953)]; Ito (*loc. cit.*); Jorgensen ("Fan Engineering," 7th ed., p. 112, Buffalo Forge Co., Buffalo, 1970); Snyder (*loc. cit.*).

Table 5-20. Additional Frictional Loss for Laminar Flow through Fittings and Valves[a]

Type of fitting or valve	Additional frictional loss expressed as K			
	$N_{Re} = 1000$	500	100	50
90-deg. ell, short radius	0.9	1.0	7.5	16
Tee, standard, along run	0.4	0.5	2.5	[illegible]
Branch to line	1.5	1.8	4.9	9.3
Gate valve	1.2	1.7	9.9	24
Globe valve, composition disk	11	12	20	30
Plug	12	14	19	27
Angle valve	8	8.5	11	19
Check valve, swing	4	4.5	17	55

[a] From curves by Kittredge and Rowley, *Trans. Am. Soc. Mech. Engrs.*, 79, 1759–1766 (1957).

[i] Ito, *J. Basic Eng.*, 82, 131–143 (1960).
[j] Lansford, Loss of Head in Flow of Fluids through Various Types of 1½-in. Valves, *Univ. Illinois Eng. Expt. Sta. Bull. Series* 340, 1943.
[k] Lapple, *Chem. Eng.*, 56(5), 96–104 (1949), general survey reference.
[l] McNown, *Proc. Am. Soc. Civil Engrs.*, 79, Separate 258, pp. 1–22 (1953); discussion, *ibid.*, 80, Separate 396, pp. 19–45 (1954).
[m] Schoder and Dawson, "Hydraulics," 2d ed., p. 213, McGraw-Hill, New York, 1934.
[n] Streeter, *Prod. Eng.*, 18(7), 89–91 (1947).
[o] This is pressure drop (including friction loss) between run and branch, based on velocity in the main stream before branching. Actual value depends on the flow split, ranging from 0.5 to 1.3 if main stream enters run and from 0.7 to 1.5 if main stream enters branch.
[p] The fraction open is directly proportional to stem travel or turns of hand wheel. Flow direction through some types of valves has a small effect on pressure drop (see Freeman, *op. cit.*). For practical purposes this effect may be neglected.
[q] Values apply only when check valve is fully open, which is generally the case for velocities more than 3 ft./sec. for water.
[r] Values should be regarded as approximate because there is much variation in equipment of the same type from different manufacturers.

APL STORAGE TANK - EMMISSION CALCULATION

Location	Niagara Falls, NY
Company	Oxychem/Olin

Tank Data

Type	Horizontal
Shell length (ft)	24.6
Diameter (ft)	9
Working volume (gallons)	10000
Turnovers per year	260
Net throughput (gal/yr)	2600000
Vacuum setting (psig)	-0.03
Pressure setting (psig)	0.03

Meteorological Data

Average maximum ambient temperature (F)	55.8
Average minimum ambient temperature (F)	39.3
Average ambient temperature (F)	47.6
Average annual solar isolation factor (Btu/ft.ft.day)	1034
Average wind speed (mph)	12
Tank paint solar absorptance	0.17
Liquid bulk temperature (F)	47.6
Daily average liquid surface temperature (F)	48.9
Daily maximum liquid surface temperature (F)	53.2
Daily minimum liquid surface temperature (F)	44.7
Daily vapor temperature range (F)	16.80

Tank Contents

Component	Molecular Wieght	Liquid Mass Fraction	Vapor Pressure (psia)			Avg. Vapor Mass Fraction
			@Avg. Temp.	@Min. Temp.	@Max. Temp	
Benzene	78.11	8.93E-06	0.8539	0.7543	0.9642	4.442E-06
Chlorobenzene	112.60	4.31E-06	0.0912	0.0784	0.1057	2.289E-07
Water	18.00	0.99999	1.7172	1.4151	2.1411	0.999995
APL		1.00000	1.7172	1.4151	2.1411	1.00000

Vapor molecular weight	18.00

Standing Storage Loss

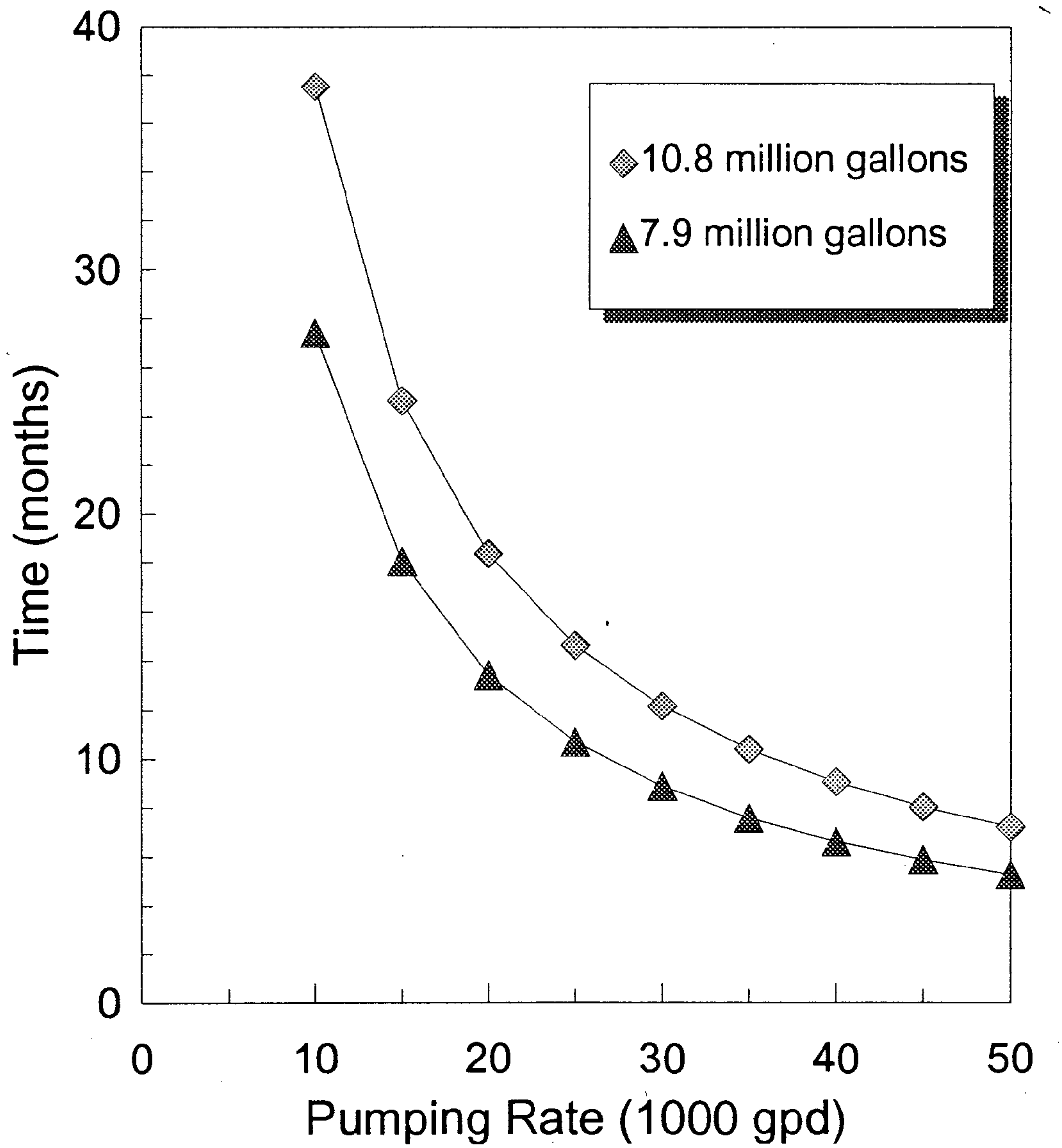
Vapor space volume (ft^3)	996.80
Vapor density (lb/ft^3)	0.0057
Vapor space expansion factor	0.0843
Vented vapor saturation factor	0.7094
Total Standing Storage Loss (lb/yr)	123.16

Withdrawal Losses

Turnover factor (1 for N upto 36; (180+N)/(6N) for N >	0.2821
Working loss product factor	1.0000
Total withdrawal lossess (lb/yr)	539.70
Total losses (lb/yr)	**662.86**

Component	Vapor fraction	Losses (lb/yr)
Benzene	4.442E-06	0.0029
Chlorobenzene	2.289E-07	0.0002
Water	0.99999533	662.8573
APL Vapors	1	**662.8604**

Figure 2: Time Required to Reach Steady State as a Function of Initial Pumping Rate



6.0 SUMMARY

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The "Alternate A" cap design uses a 12-inch silty clay layer beneath a 60-mil VLDPE liner while "Alternate B" uses a geosynthetic clay liner beneath a 40-mil VLDPE. The "Alternate B" cap design has been incorporated into the remedial design and used as the basis for calculations contained herein.

The "Alternate B" cap design should allow less infiltration than the "Alternate A" cap design. A conservative estimate, i.e., more than is anticipated, of total inflow of less than 400 gallons of water per day into the landfill area is estimated at steady state for cap design "Alternate B". These numbers are based on annual average basis. The actual daily inflow will vary with fluctuations in river elevation, seasonal water table, and bedrock head potential, and changes in rainfall and climatologic conditions.

A total of 7,900,000 to 10,800,000 gallons of APL will be pumped out of the landfill before a one-foot head differential across the perimeter slurry wall and a steady state infiltration of less than 400 gallons per day is achieved.

Twenty thousand gallons holding capacity and APL pumping rate of 20,000 gallons per day, 5 days a week, is suggested for the initial APL pumping stages. Once steady state conditions are within the encapsulated landfill, APL pumping once a month should maintain inward gradients across the perimeter slurry wall.

Department of Environmental Conservation

Division of Solid Waste

6 NYCRR Part 360
Solid Waste
Management Facilities

Effective December 31, 1988
Revised May 28, 1991

New York State Department of Environmental Conservation
MARIO M. CUOMO, *Governor* THOMAS C. JORLING, *Commissioner*

L BELL

(o) Filter layer criteria. The filter layer must be designed to prevent the migration of fine soil particles into a coarser grained material, and allow water or gases to freely enter a drainage medium (pipe or drainage blanket) without clogging.

(1) For graded cohesionless soil filters. The granular soil material used as a filter must have no more than five percent by weight passing the No. 200 sieve and no soil particles larger than three inches in any dimension.

(2) Geosynthetic filters. Geotextiles filter material must demonstrate that the hydraulic conductivity, and chemical and physical resistance is not adversely affected by waste placement, any overlying material or leachate generated at the landfill. Geotextile filter openings must be sized in accordance with the following criteria which takes into consideration the soil found in layers located adjacent to the geotextile filter:

$$\frac{O_{95} \text{ of the geotextile}}{d_{85} \text{ of this soil}} < 2 \text{ and}$$

$$\frac{O_{95} \text{ of the geotextile}}{d_{15} \text{ of this soil}} > 2$$

The d_{85} is the soil particle size at which 85 percent of the particles are finer, and the d_{15} is the soil particle size at which 15 percent of the particles are finer. The O_{95} is the apparent opening size of the geotextile at which 95 percent of the soil particles will pass. An apparent opening size test acceptable to the department must be performed to demonstrate compliance with this criteria.

(3) Construction requirements. Both the soil filters and geotextile filters must be installed in accordance with the approved engineering plans, reports, and specifications.

(4) Certification requirements. The project engineer must include in the construction certification report the results of all the required quality assurance and quality control testing performed. The testing procedures and protocols must be acceptable to the department and submitted in accordance with section 360-2.8 of this Part.

(p) Gas venting layer. A gas venting layer must be located directly below the barrier layer of the final cover system and above the compacted waste layer. Such layer must be designed and constructed in accordance with the requirements of this subdivision for a soil venting layer or as a geosynthetic venting layer designed and constructed to effectively perform the equivalent functions of the soil venting layer and found acceptable to the department.

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	90	110	140	170	210
Grab Strength (MD/CD)[1]	lbs	ASTM D-4632	120/95	150/115	230/180	300/235	420/350	540/450	650/570
Grab Elongation (MD/CD)[1]	%	ASTM D-4632	65/75	75/85	75/85	75/85	75/80	80/80	85/85
Trapezoid Tear Strength (MD/CD)[1]	lbs	ASTM D-4533	50/40	55/50	80/75	105/95	140/125	180/165	225/200
Puncture Resistance	lbs	ASTM D-4833	55	65	95	115	155	185	225
Mullen Burst Strength	psi	ASTM D-3786	195	225	320	400	560	700	855
Water Flow Rate	gpm/ft²	ASTM D-4491	195	190	170	150	120	100	80
Permittivity, Ψ	sec^{-1}	ASTM D-4491	2.61	2.54	2.27	2.01	1.60	1.34	1.07
Permeability, $k = \Psi t$	cm/sec	ASTM D-4491	.40	.45	.52	.56	.57	.58	.57
AOS	Sieve Size mm	ASTM D-4751	70-100 .210-.149	70-100 .210-.149	70-100 .210-.149	70-100 .210-.149	100-120 .149-.125	120-140 .125-.106	140-170 .106-.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† 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	3.3	4.0	5.7	7.1	10.0	13.0	16.0
Thickness, t	mils	ASTM D-1777	50	55	75	95	125	150	185
Grab Strength	lbs	ASTM D-4632	80	100	160	210	305	390	500
Grab Elongation	%	ASTM D-4632	60	60	60	60	60	65	70
Trapezoid Tear Strength	lbs	ASTM D-4533	30	40	60	75	100	130	150
Puncture Resistance	lbs	ASTM D-4833	40	50	80	95	130	155	195
Mullen Burst Strength	psi	ASTM D-3786	170	190	275	360	510	640	780
Water Flow Rate	gpm/ft²	ASTM D-4491	155	150	130	110	80	60	40
Permittivity, Ψ	sec^{-1}	ASTM D-4491	2.07	2.01	1.74	1.47	1.07	0.80	0.53
Permeability, $k = \Psi t$	cm/sec	ASTM D-4491	.26	.28	.33	.35	.34	.31	.25
AOS	Sieve Size mm	ASTM D-4751	50 .300	50 .300	70 .210	70 .210	70 .210	100 .149	100 .149

†These minimum values represent minimum test values determined from Q.C. testing on all lots produced in 1989. Certified "Minimum Average Roll Values" representing the industry standard of a 95 percent confidence level (i.e. mean less two standard deviations) may be higher than these values and are determined for each production lot. Please contact your Trevira® Distributor or Hoechst Celanese Corporation for additional information.

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Nonwoven Geotextiles For Your Application.

Technical Data

PROPERTY		TEST PROCEDURE	UNIT	TS 1000	TS 900	TS 800	TS 750	TS 700	TS 650	TS 600	TS 550	TS 500	TS 420
PHYSICAL													
Weight	Minimum(1)	ASTM D 3776	oz/yd²	15.7	13.8	11.8	9.8	7.9	7.1	6.0	5.0	4.2	3.6
	Typical			16.6	14.5	12.4	10.3	8.3	7.5	6.3	5.3	4.5	3.8
Thickness	Minimum(1)	ASTM D5199	MILS	145*	130*	115*	105*	90*	80*	70*	60*	50*	45*
	Typical			170	150	130	120	105	95	80	70	60	55
Asphalt Retention	Minimum(1)	TF25 Method 8	gal/yd²										
	Typical												
MECHANICAL													
Grab Tensile	Minimum(1)	ASTM D4632	lbs	340	310	300	250	210	180	160	130	115	90
	Typical			480/360**	400/330**	350/320**	300	240	210	180	150	125	110
Grab Elongation At Break	Minimum(1)	ASTM D4632	%	80	80	60	50	50	50	50	50	50	50
	Typical			100/95**	100/95**	95/80**	>50	>50	>50	>50	>50	>50	>50
Wide-Width Tensile(3)	Typical	ASTM D4595	lbs/in	140	135	130	110	90	80	70	60	50	45
Puncture Resistance	Minimum(1)	ASTM D4833	lbs	160	140	135	125	100	90	75	70	60	50
	Typical			180	160	150	140	115	110	85	75	65	55
Trapezoidal Tear	Minimum(1)	ASTM D4533	lbs	120	110	110	100	85	75	70	60	50	45
	Typical			200/160**	170/140**	150/130**	120	100	90	80	75	60	50
Mullen Burst	Minimum(1)	ASTM D3786	psi	500	425	400	380	320	290	220	200	160	140
	Typical			550	470	450	425	350	310	260	220	185	155
HYDRAULIC													
Water Flow Rate	Minimum	[illegible]	gpm/ft	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	135	140	170	195
	Typical			[illegible]	80	95	[illegible]	130	140	170	200	225	250
Permittivity	Minimum	ASTM D4491	sec⁻¹	0.6	0.70	1.0	[illegible]	[illegible]	1.6	1.8	1.9	2.3	2.6
	Typical			0.9	[illegible]	1.3	1.4	1.7	1.9	2.3	2.7	3.0	3.4
Permeability, Kv	Minimum	ASTM D4491	cm/sec	0.2	0.2*	0.3*	0.3*	0.3*	0.3*	0.3*	0.3*	0.3*	0.3*
	Typical			0.4	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5
Transmissivity at													
0.3 psi		ASTM 4716	gpm/ft (x 10⁻²)	15.0	13.0	12.7	11.5	11.0	10.0	9.5	8.5	7.0	6.7
14.5 psi				7.0	7.0	7.0	7.0	7.0	5.0	4.0	3.8	3.0	2.7
29.0 psi				4.0	4.0	4.0	4.0	4.0	3.5	3.0	2.5	2.0	1.6
A.O.S.	Minimum(1)	ASTM D4751	Sieve Size	(100)	(100)	(80)	(80)	(70)	(60)	(60)	(50)	(50)	(40)
	Typical			(>140)	(>140)	(140-100)	(140-100)	(120-70)	(100-70)	(100-70)	(80-60)	(80-50)	(70-40)
	Minimum(1)		mm	.150*	.150*	.180*	.180*	.212*	.250*	.250*	.300*	.300*	.425*
	Typical			<.106	<.106	.106-.150	.106-.150	.125-.212	.150-.212	.150-.212	.180-.250	.180-.300	.212-.425
ENDURANCE													
UV Resistance (500 hrs)(2)	Minimum(1)	ASTM D4355	% Str. Ret.	80	80	80	80	70	70	70	70	70	70
	Typical			>90	>90	>90	>90	>85	>85	>85	>85	>85	>85
pH Resistance				2-13	2-13	2-13	2-13	2-13	2-13	2-13	2-13	2-13	2-13

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Geonet Drainage Layer Capacity Requirements

References:

1. Standard Statistical Probability Equation

2. Handbook of Engineering Fundamentals by O. W. Eshbach pg 1-150

3. Designing With Geosynthetics by Robert M. Koerner pgs 90-91, 350-352

4. Hydrologic Evaluation of Landfill Performance (HELP) Model by P. R. Schroeder, R. L. Peyton, B. M. McEnroe and J. W. Sjostrom w/ U.S. Army Engineer Waterways Experiment Station (ATTACHED)

FORM E50 (Rev. 8-91)

Geonet Drainage Layer Capacity Requirements Calculations

Geonet Drainage Layer Capacity Requirements

Worse case scenario for amount of percolated water retained by the geomembrane which must be collected and transported by the geonet is given from the results of the HELP* modeling Analysis is as follows:

Using a 40 mil thick VLDPE Geomembrane underlain by a geosynthetic clay liner (GCL), the HELP Model Analysis predicted a maximum of 11.53 inches / unit area / year of Average Annual Drainage to be collected by the Geonet with a 3.12 inch / unit area / year Standard Deviation.

Average Annual Drainage (Percolation) to Geonet use: Average Annual Drainage + 3 × Std Deviation which will account for 99.7% (95% is considered acceptable) of all probabilities of occurrance. Ref. 1

Using higher value of above alternatives

$$11.53 + (3)(3.12) = 20.89 \text{ inches/unit area/year}$$

* Hydrologic Evaluation of Landfill Performance Modeling Analysis results attached to these calculations as an appendix. Ref 4

Longest flow length through geonet is approximately 460 feet long from the center peak to the slurry wall

Therefore peak capacity of the geonet must be (converting to gallons per minute - GPM)

Ref. 2

$$\frac{\frac{20.89''}{12''/ft}(460\,ft)\frac{7.48\,gallons}{ft^3}}{year\left(\frac{365\,days}{yr}\right)\left(\frac{24\,hrs}{day}\right)\left(\frac{60\,min}{hr}\right)} = 0.0114\ GPM/ft$$

Calculating required transmissivity, θ, is dependent on the hydraulic gradient, which will be considered the same as the slope of the geonet.

The flattest gradient will be used to be conservative.

Ref. 3

$$\text{Transmissivity} = \theta = \frac{q}{i}$$

where:
q = flow rate per unit width of media
i = hydraulic gradient

$$\theta = \frac{0.0114\ GPM/ft}{.03}$$

(NOTE: 3% is the flattest gradient ∴ will be the worse case)

$$= 0.38\ GPM/ft\ \text{(Actual)}$$

Factors of Safety will be applied as follows:

Ref. 3

$$FS = \frac{\text{Design Transmissivity}}{\text{Actual Transmissivity}}$$

Design Transmissivity = FS × Actual Transmissivity

$$FS = FS_{IN} \times FS_{CR} \times FS_{CC} \times FS_{BC}$$

where FS_{IN} = FS for intrusion of geosynthetics into the core space of the geonet, 1.4 is Average

FS_{CR} = FS for creep deformation, 1.3 is Avg.

FS_{CC} = FS for chemical clogging 1.1 is Avg.

FS_{BC} = FS for biological clogging 1.35 is Avg.

$$FS = 1.4 \times 1.3 \times 1.1 \times 1.35 = 2.70$$

Therefore, Design Θ = 0.38 × 2.70 = 1.03 GPM/ft

= 0.21×10^{-3} m²/sec

1.0 Geotextile

GEOTEXTILE ABOVE GEONET
Design for separation / filtration.

REFERENCES:

1. Geotextile Design & Construction Guidelines, Federal Highway Administration Publication No. FHWA-HI-90-001, October, 1989.

2. NYSDEC Solid Waste Mgmt. Facilities Regs [6 NYCRR Part 360-2.13(o)(2)]

REF.

#1 p.38

1. NATURE OF PROJECT: NON-CRITICAL
   SITE CONDITIONS: SEVERE

2. ASSUME VALUES FOR SOIL ABOVE GEOTEXTILE (SELECT COVER FILL)

$D_{10} = 0.03$ mm
$D_{60} = 0.8$ mm
$D_{85} = 4.75$ mm (No. 4 sieve)

(D_{10} = particle size for which 10% are smaller, mm.)

UNIFORMITY COEFFICIENT: $C_u = \frac{D_{60}}{D_{10}} = \frac{0.8}{0.03} = 26.7$

FOR $C_u > 8 \longrightarrow B = 1.0$

3. ASSUME MAX COEFFICIENT OF PERMEABILITY (k)

USE $k = 1 \times 10^{-3}$ (cm/s)

4. DETERMINE GEOTEXTILE REQUIREMENTS:

A. RETENSION CRITERIA:

APPARENT OPENING SIZE: $AOS \leq B D_{85}$

$\leq 1.0(4.75)$

≤ 4.75

$AOS = O_{95}$: opening size in geotextile for which 95% are smaller, mm.

REF.

4. GT CRITERIA

#1

B. PERMEABILITY CRITERIA

$$k_{GT} > 10\,k_{SOIL}$$

$$k_{GT} > 10 \cdot (10^{-3}) = 0.01 \ (cm/s)$$

5. CLOGGING CRITERIA

NOT APPLICABLE — NO PASSAGE OF FINES ALLOWABLE

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6. SURVIVABILITY/ENDURANCE CRITERIA

The attached PHYSICAL REQS. FOR DRAINAGE GEOTEXTILES should be met for Class A Drainage (CONSERVATIVE).

TABLE 2-2

PHYSICAL REQUIREMENTS[1,2] FOR DRAINAGE GEOTEXTILES (AASHTO-AGC-ARTBA TASK FORCE 25, JULY, 1986)

Property	Drainage[3] Class A[4]	Class B[5]	Test Method
Grab Strength (lbs)	180	80	ASTM D4632
Elongation (%)	n/a	n/a	ASTM D4632
Seam Strength[6] (lbs)	160	70	ASTM D4632
Puncture Strength (lbs)	80	25	ASTM D4833
Burst Strength (psi)	290	130	ASTM D3787
Trapezoid Tear (lbs)	50	25	ASTM D4533

1. Acceptance of geotextile material shall be based ASTM D-4759.

2. ~~Contracting agency may require a letter from the supplier certifying that its geotextile meets specification requirements.~~

3. Minimum; Use value in weaker principal direction. All numerical values represent minimum average roll value (i.e., test results from any sampled roll in a lot shall meet or exceed the minimum values in the Table). Stated values are for non-critical, non-serve applications. Lot samples according to ASTM D4354.

4. Class A drainage applications for fabrics are where installation stresses are more severe than Class B applications, i.e., very coarse sharp angular aggregate is used, a heavy degree of compaction (>95% AASHTO T99) is specified or depth of trench is greater than 10 ft.

5. Class B drainage applications are those where fabric is used with smooth graded surfaces having no sharp angular projections, no sharp angular aggregate is used; compaction requirements are light, (<95% AASHTO T99), and trenches are less than 10 ft in depth.

6. Values apply to both field and manufactured seams.

REF.

#2 (see attached)

7. REGULATORY CRITERIA – 6 NYCRR Part 360

$$\frac{O_{95}\ GT}{d_{85}\ SOIL} < 2 \qquad O_{95} < 2\, d_{85}\ SOIL$$
$$< 2(4.75) = 9$$

$$\frac{O_{95}\ GT}{d_{15}\ SOIL} > 2 \qquad O_{95} > 2\, d_{15}\ SOIL$$
$$> 2(0.07) = 0.14$$

$(O_{95} = AOS)$

$\therefore\ 0.14 < AOS < 9$

$0.14 < AOS < 4.75$ ($AOS < 4.75$ per. Ref. #1)

SUMMARY:

FABRIC PROPERTY	MIN. REQUIRED VALUES	MARV TREVIRA 1125	MARV POLYFELT TS-700
FABRIC WT (OZ/YD^2)	—	7.1	7.9
AOS (mm)	$0.14 < AOS < 4.75$	0.210	0.212
k (cm/s)	0.01	0.35	0.3
GRAB STRENGTH (LB) ASTM D4632	180	210	210
PUNCTURE STRENGTH (LB) ASTM D4833	80	95	100
BURST STRENGTH (PSI) ASTM D3786*	290	360	320
TRAPEZOIDAL TEAR (LB) ASTM D4533	50	75	100

USE THESE PRODUCTS OR APPROVED EQUAL.

* ASTM 3787 (LISTED IN TABLE 2-2) COMPARABLE TO ASTM 3786 per M. WAYNE, GRI

FORM 880 (Rev. 8-91)

7.0 REFERENCES

7.0 REFERENCES

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Milestone Report No. 14, 1987. NAPL study, 102nd Street Landfill, Niagara Falls, NY. Revision No. 3, October 1987.

Rawls, W.J., D.L. Brakensiek, and K.E. Saxton, 1982. Estimation of soil water properties. Transactions of American Society of Agricultural Engineers, Vol. 25, pp. 1316-1320, 1328.

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APPENDIX A.2

OUTPUT FROM HELP MODEL RUNS

SLOPE	=	3.00 PERCENT
DRAINAGE LENGTH	=	500.0 FEET

LAYER 4

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4790 VOL/VOL
FIELD CAPACITY	=	0.3710 VOL/VOL
WILTING POINT	=	0.2510 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4790 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000001000000 CM/SEC
LINER LEAKAGE FRACTION	=	0.00300000

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	75.00
TOTAL AREA OF COVER	=	940000. SQ FT
EVAPORATIVE ZONE DEPTH	=	20.00 INCHES
UPPER LIMIT VEG. STORAGE	=	9.7320 INCHES
INITIAL VEG. STORAGE	=	6.8920 INCHES
INITIAL SNOW WATER CONTENT	=	0.0000 INCHES
INITIAL TOTAL WATER STORAGE IN SOIL AND WASTE LAYERS	=	14.6490 INCHES

SOIL WATER CONTENT INITIALIZED BY USER.

CLIMATOLOGICAL DATA

SYNTHETIC RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND
SOLAR RADIATION FOR BUFFALO NEW YORK

MAXIMUM LEAF AREA INDEX	=	2.00
START OF GROWING SEASON (JULIAN DATE)	=	138
END OF GROWING SEASON (JULIAN DATE)	=	279

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
23.70	24.40	32.10	44.90	55.10	65.70
70.10	68.40	61.60	51.50	39.80	27.90

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102nd STREET LANDFILL SITE, NIAGARA FALLS, NY
ALTERNATE A WITH HYDRAULIC CONDUCTIVITY OF 0.000001 cm/sec AND
HOLES IN LINER AT EVERY 100 FEET                       November 1, 1992

**********************************************************************
**********************************************************************


                                LAYER  1
                                --------

                        VERTICAL PERCOLATION LAYER
  THICKNESS                               =       6.00 INCHES
  POROSITY                                =       0.4530 VOL/VOL
  FIELD CAPACITY                          =       0.1900 VOL/VOL
  WILTING POINT                           =       0.0850 VOL/VOL
  INITIAL SOIL WATER CONTENT              =       0.2200 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY        =       0.000719999953 CM/SEC


                                LAYER  2
                                --------

                        VERTICAL PERCOLATION LAYER
  THICKNESS                               =      18.00 INCHES
  POROSITY                                =       0.5010 VOL/VOL
  FIELD CAPACITY                          =       0.2840 VOL/VOL
  WILTING POINT                           =       0.1350 VOL/VOL
  INITIAL SOIL WATER CONTENT              =       0.3980 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY        =       0.000190000006 CM/SEC


                                LAYER  3
                                --------

                          LATERAL DRAINAGE LAYER
  THICKNESS                               =       1.00 INCHES
  POROSITY                                =       0.4170 VOL/VOL
  FIELD CAPACITY                          =       0.0210 VOL/VOL
  WILTING POINT                           =       0.0200 VOL/VOL
  INITIAL SOIL WATER CONTENT              =       0.4170 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY        =       0.009999999776 CM/SEC
```

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    AVERAGE MONTHLY VALUES IN INCHES FOR YEARS     1 THROUGH   10
------------------------------------------------------------------------------

                          JAN/JUL FEB/AUG MAR/SEP APR/OCT MAY/NOV JUN/DEC
                          ------- ------- ------- ------- ------- -------
 PRECIPITATION
 -------------
   TOTALS                   3.18    2.61    2.78    2.97    2.89    2.21
                            2.95    4.39    3.13    3.11    4.24    2.96

   STD. DEVIATIONS          0.55    1.01    0.91    0.80    0.78    0.65
                            1.06    2.04    1.46    1.37    1.06    0.73

 RUNOFF
 ------
   TOTALS                   1.386   1.595   1.669   0.264   0.030   0.000
                            0.002   0.031   0.000   0.020   0.541   0.889

   STD. DEVIATIONS          1.147   1.013   1.555   0.511   0.078   0.000
                            0.006   0.052   0.000   0.063   1.090   1.156

 EVAPOTRANSPIRATION
 ------------------
   TOTALS                   0.430   0.633   2.220   2.984   3.211   2.964
                            5.516   5.650   2.731   1.990   0.892   0.498

   STD. DEVIATIONS          0.118   0.144   0.187   0.540   1.018   0.646
                            0.551   0.551   1.007   0.360   0.116   0.125

 LATERAL DRAINAGE FROM LAYER  3
 ------------------------------
   TOTALS                   0.0788  0.0759  0.0832  0.0745  0.0723  0.0660
                            0.0642  0.0653  0.0662  0.0690  0.0691  0.0748

   STD. DEVIATIONS          0.0108  0.0082  0.0037  0.0039  0.0027  0.0009
                            0.0004  0.0021  0.0026  0.0029  0.0058  0.0112

 PERCOLATION FROM LAYER  4
 -------------------------
   TOTALS                   0.0085  0.0085  0.0094  0.0086  0.0085  0.0079
                            0.0070  0.0052  0.0046  0.0049  0.0058  0.0077

   STD. DEVIATIONS          0.0016  0.0008  0.0003  0.0003  0.0002  0.0001
                            0.0002  0.0005  0.0007  0.0010  0.0018  0.0020

******************************************************************************


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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS     1 THROUGH   10
------------------------------------------------------------------------------

                                     (INCHES)          (CU. FT.)    PERCENT
                                 ---------------    -----------    -------
 PRECIPITATION                   37.42    ( 2.873)     2931233.     100.00

 RUNOFF                           6.427   ( 3.117)      503470.      17.18
```

EVAPOTRANSPIRATION	29.719	(1.597)	2328003.	79.42
LATERAL DRAINAGE FROM LAYER 3	0.8593	(0.0291)	67311.	2.30
PERCOLATION FROM LAYER 4	0.0866	(0.0053)	6780.	0.23
CHANGE IN WATER STORAGE	0.328	(1.874)	25669.	0.88

****

****

PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

	(INCHES)	(CU. FT.)
PRECIPITATION	2.81	220116.7
RUNOFF	1.667	130553.8
LATERAL DRAINAGE FROM LAYER 3	0.0029	223.5
PERCOLATION FROM LAYER 4	0.0003	24.8
HEAD ON LAYER 4	25.3	
SNOW WATER	3.07	240224.0

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4866
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1199

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FINAL WATER STORAGE AT END OF YEAR 10

LAYER	(INCHES)	(VOL/VOL)
1	2.74	0.4563
2	8.99	0.4996
3	0.42	0.4170
4	5.75	0.4790
SNOW WATER	0.03	

```
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102nd STREET LANDFILL SITE, NIAGARA FALLS, NY
ALTERNATE A WITH HYDRAULIC CONDUCTIVITY OF 0.00001 cm/sec AND
HOLES IN LINER AT EVERY 100 FEET                     November 1, 1992

************************************************************************
************************************************************************


                              LAYER  1
                              --------

                    VERTICAL PERCOLATION LAYER
  THICKNESS                                =      6.00 INCHES
  POROSITY                                 =      0.4530 VOL/VOL
  FIELD CAPACITY                           =      0.1900 VOL/VOL
  WILTING POINT                            =      0.0850 VOL/VOL
  INITIAL SOIL WATER CONTENT               =      0.2200 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY         =      0.000719999953 CM/SEC


                              LAYER  2
                              --------

                    VERTICAL PERCOLATION LAYER
  THICKNESS                                =     18.00 INCHES
  POROSITY                                 =      0.5010 VOL/VOL
  FIELD CAPACITY                           =      0.2840 VOL/VOL
  WILTING POINT                            =      0.1350 VOL/VOL
  INITIAL SOIL WATER CONTENT               =      0.3980 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY         =      0.000190000006 CM/SEC


                              LAYER  3
                              --------

                      LATERAL DRAINAGE LAYER
  THICKNESS                                =      1.00 INCHES
  POROSITY                                 =      0.4170 VOL/VOL
  FIELD CAPACITY                           =      0.0210 VOL/VOL
  WILTING POINT                            =      0.0200 VOL/VOL
  INITIAL SOIL WATER CONTENT               =      0.4170 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY         =      0.009999999776 CM/SEC
```

SLOPE = 3.00 PERCENT
DRAINAGE LENGTH = 500.0 FEET

LAYER 4

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4790 VOL/VOL
FIELD CAPACITY	=	0.3710 VOL/VOL
WILTING POINT	=	0.2510 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4790 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000010000000 CM/SEC
LINER LEAKAGE FRACTION	=	0.00300000

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	75.00
TOTAL AREA OF COVER	=	940000. SQ FT
EVAPORATIVE ZONE DEPTH	=	20.00 INCHES
UPPER LIMIT VEG. STORAGE	=	9.7320 INCHES
INITIAL VEG. STORAGE	=	6.8920 INCHES
INITIAL SNOW WATER CONTENT	=	0.0000 INCHES
INITIAL TOTAL WATER STORAGE IN SOIL AND WASTE LAYERS	=	14.6490 INCHES

SOIL WATER CONTENT INITIALIZED BY USER.

CLIMATOLOGICAL DATA

SYNTHETIC RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND
SOLAR RADIATION FOR BUFFALO NEW YORK

MAXIMUM LEAF AREA INDEX = 2.00
START OF GROWING SEASON (JULIAN DATE) = 138
END OF GROWING SEASON (JULIAN DATE) = 279

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
23.70	24.40	32.10	44.90	55.10	65.70
70.10	68.40	61.60	51.50	39.80	27.90

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 10

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.18	2.61	2.78	2.97	2.89	2.21
	2.95	4.39	3.13	3.11	4.24	2.96
STD. DEVIATIONS	0.55	1.01	0.91	0.80	0.78	0.65
	1.06	2.04	1.46	1.37	1.06	0.73
RUNOFF						
TOTALS	1.259	1.492	1.598	0.234	0.011	0.000
	0.002	0.030	0.000	0.020	0.454	0.820
STD. DEVIATIONS	1.114	1.014	1.573	0.497	0.025	0.000
	0.005	0.050	0.000	0.063	0.944	1.128
EVAPOTRANSPIRATION						
TOTALS	0.428	0.633	2.215	2.999	3.216	2.907
	5.477	5.550	2.670	1.985	0.887	0.496
STD. DEVIATIONS	0.116	0.144	0.182	0.573	0.987	0.597
	0.583	0.707	1.050	0.364	0.117	0.123
LATERAL DRAINAGE FROM LAYER 3						
TOTALS	0.0781	0.0756	0.0826	0.0737	0.0711	0.0647
	0.0639	0.0659	0.0668	0.0703	0.0702	0.0750
STD. DEVIATIONS	0.0105	0.0086	0.0049	0.0042	0.0026	0.0006
	0.0003	0.0021	0.0030	0.0039	0.0056	0.0112
PERCOLATION FROM LAYER 4						
TOTALS	0.0840	0.0843	0.0932	0.0855	0.0842	0.0774
	0.0688	0.0503	0.0448	0.0471	0.0560	0.0754
STD. DEVIATIONS	0.0168	0.0085	0.0040	0.0033	0.0023	0.0008
	0.0017	0.0044	0.0055	0.0088	0.0183	0.0207

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	37.42	(2.873)	2931233.	100.00
RUNOFF	5.920	(2.984)	463736.	15.82

EVAPOTRANSPIRATION	29.464	(1.647)	2308021.	78.74
LATERAL DRAINAGE FROM LAYER 3	0.8578	(0.0302)	67196.	2.29
PERCOLATION FROM LAYER 4	0.8510	(0.0538)	66658.	2.27
CHANGE IN WATER STORAGE	0.327	(1.901)	25622.	0.87

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

	(INCHES)	(CU. FT.)
PRECIPITATION	2.81	220116.7
RUNOFF	1.662	130193.1
LATERAL DRAINAGE FROM LAYER 3	0.0029	223.5
PERCOLATION FROM LAYER 4	0.0032	247.7
HEAD ON LAYER 4	25.2	
SNOW WATER	3.07	240155.4

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.4866

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1199

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FINAL WATER STORAGE AT END OF YEAR 10

LAYER	(INCHES)	(VOL/VOL)
1	2.73	0.4553
2	8.99	0.4996
3	0.42	0.4170
4	5.75	0.4790
SNOW WATER	0.03	

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102nd STREET LANDFILL SITE, NIAGARA FALLS, NY
ALTERNATE A WITH HYDRAULIC CONDUCTIVITY OF 0.00001 cm/sec AND
HOLES IN LINER AT EVERY 50 FEET                        November 1, 1992

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*************************************************************************


                              LAYER  1
                              --------

                   VERTICAL PERCOLATION LAYER
  THICKNESS                               =       6.00 INCHES
  POROSITY                                =       0.4530 VOL/VOL
  FIELD CAPACITY                          =       0.1900 VOL/VOL
  WILTING POINT                           =       0.0850 VOL/VOL
  INITIAL SOIL WATER CONTENT              =       0.2200 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY        =       0.000719999953 CM/SEC


                              LAYER  2
                              --------

                   VERTICAL PERCOLATION LAYER
  THICKNESS                               =      18.00 INCHES
  POROSITY                                =       0.5010 VOL/VOL
  FIELD CAPACITY                          =       0.2840 VOL/VOL
  WILTING POINT                           =       0.1350 VOL/VOL
  INITIAL SOIL WATER CONTENT              =       0.3980 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY        =       0.000190000006 CM/SEC


                              LAYER  3
                              --------

                     LATERAL DRAINAGE LAYER
  THICKNESS                               =       1.00 INCHES
  POROSITY                                =       0.4170 VOL/VOL
  FIELD CAPACITY                          =       0.0210 VOL/VOL
  WILTING POINT                           =       0.0200 VOL/VOL
  INITIAL SOIL WATER CONTENT              =       0.4170 VOL/VOL
  SATURATED HYDRAULIC CONDUCTIVITY        =       0.009999999776 CM/SEC
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SLOPE	=	3.00 PERCENT
DRAINAGE LENGTH	=	500.0 FEET

LAYER 4

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4790 VOL/VOL
FIELD CAPACITY	=	0.3710 VOL/VOL
WILTING POINT	=	0.2510 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4790 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000010000000 CM/SEC
LINER LEAKAGE FRACTION	=	0.01000000

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	75.00
TOTAL AREA OF COVER	=	940000. SQ FT
EVAPORATIVE ZONE DEPTH	=	20.00 INCHES
UPPER LIMIT VEG. STORAGE	=	9.7320 INCHES
INITIAL VEG. STORAGE	=	6.8920 INCHES
INITIAL SNOW WATER CONTENT	=	0.0000 INCHES
INITIAL TOTAL WATER STORAGE IN SOIL AND WASTE LAYERS	=	14.6490 INCHES

SOIL WATER CONTENT INITIALIZED BY USER.

CLIMATOLOGICAL DATA

SYNTHETIC RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND
SOLAR RADIATION FOR BUFFALO NEW YORK

MAXIMUM LEAF AREA INDEX	=	2.00
START OF GROWING SEASON (JULIAN DATE)	=	138
END OF GROWING SEASON (JULIAN DATE)	=	279

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
23.70	24.40	32.10	44.90	55.10	65.70
70.10	68.40	61.60	51.50	39.80	27.90

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 10

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.18	2.61	2.78	2.97	2.89	2.21
	2.95	4.39	3.13	3.11	4.24	2.96
STD. DEVIATIONS	0.55	1.01	0.91	0.80	0.78	0.65
	1.06	2.04	1.46	1.37	1.06	0.73
RUNOFF						
TOTALS	0.985	1.258	1.337	0.184	0.001	0.000
	0.001	0.025	0.000	0.020	0.267	0.617
STD. DEVIATIONS	0.976	0.960	1.588	0.455	0.003	0.000
	0.003	0.043	0.000	0.062	0.570	1.039
EVAPOTRANSPIRATION						
TOTALS	0.429	0.633	2.215	3.030	3.182	2.954
	5.438	5.058	2.557	1.990	0.888	0.496
STD. DEVIATIONS	0.116	0.144	0.180	0.559	1.011	0.555
	0.632	1.004	1.081	0.371	0.113	0.123
LATERAL DRAINAGE FROM LAYER 3						
TOTALS	0.0800	0.0736	0.0803	0.0704	0.0676	0.0629
	0.0635	0.0690	0.0713	0.0785	0.0829	0.0800
STD. DEVIATIONS	0.0087	0.0101	0.0071	0.0049	0.0021	0.0007
	0.0004	0.0018	0.0027	0.0066	0.0150	0.0147
PERCOLATION FROM LAYER 4						
TOTALS	0.2615	0.2675	0.3024	0.2754	0.2662	0.2414
	0.2087	0.1515	0.1362	0.1391	0.1666	0.2318
STD. DEVIATIONS	0.0760	0.0467	0.0260	0.0135	0.0109	0.0083
	0.0087	0.0081	0.0062	0.0179	0.0591	0.0827

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	37.42	(2.873)	2931233.	100.00
RUNOFF	4.694	(2.672)	367734.	12.55

EVAPOTRANSPIRATION	28.871	(1.887)	2261591.	77.15
LATERAL DRAINAGE FROM LAYER 3	0.8801	(0.0246)	68940.	2.35
PERCOLATION FROM LAYER 4	2.6484	(0.2500)	207462.	7.08
CHANGE IN WATER STORAGE	0.326	(2.256)	25506.	0.87

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

	(INCHES)	(CU. FT.)
PRECIPITATION	2.81	220116.7
RUNOFF	1.657	129787.0
LATERAL DRAINAGE FROM LAYER 3	0.0035	271.5
PERCOLATION FROM LAYER 4	0.0106	827.9
HEAD ON LAYER 4	25.4	
SNOW WATER	3.06	240019.5

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.4866

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1199

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FINAL WATER STORAGE AT END OF YEAR 10

LAYER	(INCHES)	(VOL/VOL)
1	2.72	0.4528
2	8.99	0.4996
3	0.42	0.4170
4	5.75	0.4790
SNOW WATER	0.03	

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102nd STREET LANDFILL SITE, NIAGARA FALLS, NY
ALTERNATE B WITH TYPICAL MAXIMUM LEAKAGE FACTOR
(HOLES IN LINER AT EVERY 50 FEET) OCTOBER 26, 1992

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LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	6.00 INCHES
POROSITY	=	0.4530 VOL/VOL
FIELD CAPACITY	=	0.1900 VOL/VOL
WILTING POINT	=	0.0850 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2200 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000719999953 CM/SEC

LAYER 2

VERTICAL PERCOLATION LAYER

THICKNESS	=	18.00 INCHES
POROSITY	=	0.5010 VOL/VOL
FIELD CAPACITY	=	0.2840 VOL/VOL
WILTING POINT	=	0.1350 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3980 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000189999992 CM/SEC

LAYER 3

LATERAL DRAINAGE LAYER

THICKNESS	=	1.00 INCHES
POROSITY	=	0.4170 VOL/VOL
FIELD CAPACITY	=	0.0210 VOL/VOL
WILTING POINT	=	0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4170 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.009999999776 CM/SEC

SLOPE	=	3.00 PERCENT
DRAINAGE LENGTH	=	500.0 FEET

LAYER 4

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	0.20 INCHES
POROSITY	=	0.4790 VOL/VOL
FIELD CAPACITY	=	0.3710 VOL/VOL
WILTING POINT	=	0.2510 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4790 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000000006000 CM/SEC
LINER LEAKAGE FRACTION	=	0.01000000

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	75.00
TOTAL AREA OF COVER	=	940000. SQ FT
EVAPORATIVE ZONE DEPTH	=	20.00 INCHES
UPPER LIMIT VEG. STORAGE	=	9.7320 INCHES
INITIAL VEG. STORAGE	=	6.8920 INCHES
INITIAL SNOW WATER CONTENT	=	0.0000 INCHES
INITIAL TOTAL WATER STORAGE IN SOIL AND WASTE LAYERS	=	8.9968 INCHES

SOIL WATER CONTENT INITIALIZED BY USER.

CLIMATOLOGICAL DATA

SYNTHETIC RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND
SOLAR RADIATION FOR BUFFALO NEW YORK

MAXIMUM LEAF AREA INDEX	=	2.00
START OF GROWING SEASON (JULIAN DATE)	=	138
END OF GROWING SEASON (JULIAN DATE)	=	279

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
23.70	24.40	32.10	44.90	55.10	65.70
70.10	68.40	61.60	51.50	39.80	27.90

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 10

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.18	2.61	2.78	2.97	2.89	2.21
	2.95	4.39	3.13	3.11	4.24	2.96
STD. DEVIATIONS	0.55	1.01	0.91	0.80	0.78	0.65
	1.06	2.04	1.46	1.37	1.06	0.73
RUNOFF						
TOTALS	1.392	1.599	1.672	0.264	0.031	0.000
	0.002	0.031	0.000	0.020	0.544	0.892
STD. DEVIATIONS	1.147	1.011	1.554	0.512	0.080	0.000
	0.006	0.052	0.000	0.063	1.095	1.156
EVAPOTRANSPIRATION						
TOTALS	0.430	0.633	2.220	2.984	3.211	2.964
	5.516	5.655	2.733	1.990	0.892	0.498
STD. DEVIATIONS	0.118	0.144	0.187	0.540	1.018	0.646
	0.551	0.546	1.005	0.360	0.116	0.125
LATERAL DRAINAGE FROM LAYER 3						
TOTALS	0.0788	0.0760	0.0832	0.0745	0.0723	0.0661
	0.0642	0.0653	0.0661	0.0689	0.0690	0.0748
STD. DEVIATIONS	0.0108	0.0082	0.0037	0.0039	0.0027	0.0009
	0.0004	0.0021	0.0026	0.0029	0.0059	0.0112
PERCOLATION FROM LAYER 4						
TOTALS	0.0065	0.0068	0.0075	0.0067	0.0065	0.0059
	0.0047	0.0025	0.0019	0.0021	0.0033	0.0055
STD. DEVIATIONS	0.0019	0.0009	0.0003	0.0004	0.0003	0.0001
	0.0003	0.0006	0.0008	0.0012	0.0022	0.0023

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	37.42	(2.873)	2931233.	100.00
RUNOFF	6.447	(3.119)	505044.	17.23

EVAPOTRANSPIRATION	29.726	(1.594)	2328510.	79.44
LATERAL DRAINAGE FROM LAYER 3	0.8593	(0.0290)	67310.	2.30
PERCOLATION FROM LAYER 4	0.0600	(0.0063)	4698.	0.16
CHANGE IN WATER STORAGE	0.328	(1.870)	25670.	0.88

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

	(INCHES)	(CU. FT.)
PRECIPITATION	2.81	220116.7
RUNOFF	1.666	130517.9
LATERAL DRAINAGE FROM LAYER 3	0.0029	223.5
PERCOLATION FROM LAYER 4	0.0003	20.3
HEAD ON LAYER 4	25.3	
SNOW WATER	3.07	240225.5

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4866
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1199

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FINAL WATER STORAGE AT END OF YEAR 10

LAYER	(INCHES)	(VOL/VOL)
1	2.74	0.4563
2	8.99	0.4996
3	0.42	0.4170
4	0.10	0.4790
SNOW WATER	0.03	

APPENDIX A.3

CAP

Reference
4

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102nd STREET LANDFILL SITE, NIAGARA FALLS, NY
ALTERNATE B WITH MAXIMUM LATERAL DRAINAGE (1 CM/SEC GEONET EFF. PERMEA
(HOLES IN LINER AT EVERY 50 FEET)   OCTOBER 26, 1992

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LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	6.00 INCHES
POROSITY	=	0.4530 VOL/VOL
FIELD CAPACITY	=	0.1900 VOL/VOL
WILTING POINT	=	0.0850 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2200 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000719999953 CM/SEC

LAYER 2

VERTICAL PERCOLATION LAYER

THICKNESS	=	18.00 INCHES
POROSITY	=	0.5010 VOL/VOL
FIELD CAPACITY	=	0.2840 VOL/VOL
WILTING POINT	=	0.1350 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3980 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000189999992 CM/SEC

LAYER 3

LATERAL DRAINAGE LAYER

THICKNESS	=	1.00 INCHES
POROSITY	=	0.4170 VOL/VOL
FIELD CAPACITY	=	0.0210 VOL/VOL
WILTING POINT	=	0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4170 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	1.000000000000 CM/SEC

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SLOPE                                  =      5.00 PERCENT
DRAINAGE LENGTH                        =    500.0 FEET


                              LAYER  4
                              --------

        BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER
THICKNESS                              =      0.20 INCHES
POROSITY                               =      0.4790 VOL/VOL
FIELD CAPACITY                         =      0.3710 VOL/VOL
WILTING POINT                          =      0.2510 VOL/VOL
INITIAL SOIL WATER CONTENT             =      0.4790 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY       =      0.000000006000 CM/SEC
LINER LEAKAGE FRACTION                 =      0.01000000


                       GENERAL SIMULATION DATA
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SCS RUNOFF CURVE NUMBER                =      75.00
TOTAL AREA OF COVER                    =  940000. SQ FT
EVAPORATIVE ZONE DEPTH                 =      20.00 INCHES
UPPER LIMIT VEG. STORAGE               =       9.7320 INCHES
INITIAL VEG. STORAGE                   =       6.8920 INCHES
INITIAL SNOW WATER CONTENT             =       0.0000 INCHES
INITIAL TOTAL WATER STORAGE IN
  SOIL AND WASTE LAYERS                =       8.9968 INCHES

           SOIL WATER CONTENT INITIALIZED BY USER.


                         CLIMATOLOGICAL DATA
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SYNTHETIC RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND
SOLAR RADIATION FOR      BUFFALO              NEW YORK


MAXIMUM LEAF AREA INDEX                   = 2.00
START OF GROWING SEASON (JULIAN DATE)     =  138
END OF GROWING SEASON (JULIAN DATE)       =  279


      NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL     FEB/AUG     MAR/SEP     APR/OCT     MAY/NOV     JUN/DEC
-------     -------     -------     -------     -------     -------

 23.70       24.40       32.10       44.90       55.10       65.70
 70.10       68.40       61.60       51.50       39.80       27.90
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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 10

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.18	2.61	2.78	2.97	2.89	2.21
	2.95	4.39	3.13	3.11	4.24	2.96
STD. DEVIATIONS	0.55	1.01	0.91	0.80	0.78	0.65
	1.06	2.04	1.46	1.37	1.06	0.73
RUNOFF						
TOTALS	0.010	0.002	0.024	0.000	0.000	0.000
	0.001	0.025	0.000	0.018	0.001	0.000
STD. DEVIATIONS	0.033	0.004	0.056	0.000	0.000	0.000
	0.002	0.043	0.000	0.056	0.001	0.000
EVAPOTRANSPIRATION						
TOTALS	0.430	0.633	2.223	3.001	3.193	2.998
	3.286	4.141	2.587	2.009	0.901	0.500
STD. DEVIATIONS	0.118	0.145	0.179	0.559	1.072	0.640
	1.122	1.220	1.109	0.409	0.109	0.126
LATERAL DRAINAGE FROM LAYER 3						
TOTALS	2.4226	2.0945	2.2979	0.6744	0.2810	0.1220
	0.0653	0.0350	0.0401	0.1860	1.3170	1.9920
STD. DEVIATIONS	1.1301	0.7109	1.4949	0.4754	0.1817	0.0385
	0.0114	0.0034	0.0540	0.3542	1.4769	1.2221
PERCOLATION FROM LAYER 4						
TOTALS	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
STD. DEVIATIONS	0.0000	0.0000	0.0001	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	37.42	(2.873)	2931233.	100.00
RUNOFF	0.081	(0.092)	6315.	0.22

EVAPOTRANSPIRATION	25.903	(1.987)	2029071.	69.22
LATERAL DRAINAGE FROM LAYER 3	11.5280	(3.1201)	903028.	30.81
PERCOLATION FROM LAYER 4	0.0010	(0.0001)	77.	0.00
CHANGE IN WATER STORAGE	-0.093	(0.840)	-7258.	-0.25

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

	(INCHES)	(CU. FT.)
PRECIPITATION	2.81	220116.7
RUNOFF	0.177	13893.4
LATERAL DRAINAGE FROM LAYER 3	0.5610	43945.2
PERCOLATION FROM LAYER 4	0.0001	6.1
HEAD ON LAYER 4	7.8	
SNOW WATER	3.06	239407.7

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4051
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1199

****

****

FINAL WATER STORAGE AT END OF YEAR 10

LAYER	(INCHES)	(VOL/VOL)
1	1.54	0.2573
2	6.35	0.3530
3	0.05	0.0463
4	0.10	0.4790
SNOW WATER	0.03	

3.0 Collection System (Subsurface)

2.0 Drainage Layer (Geonet)

SUBDRAIN

COLLECTION SYSTEM ([illegible])

DESIGN CALCULATIONS

REFERENCES:

1. Handbook of Engineering Fundamentals
   by O.W. Eshbach, 2nd Edition, pg 1-150

2. Advanced Drainage Systems, Inc.
   Specifier Manual

3. Open Channel Flow by F. M. HENDERSON
   pages 3, 91, and 96

4. Groundwater by R. A. Freeze and J. A. Cherry
   pg 15-29

5. DRAINAGE DIVIDE PLAN DWG 594000-109-03
   REV. A,

From sheet 1 of 3 "Drainage Layer Capacity Requirements Calculations";

1. Average Annual Drainage (Percolation) to be transported by the Geonet plus three times the standard deviation was calculated to be 20.89 inches/ft²/year (see sheet 1 of 3 Geonet Capacity Calculations). To determine the peak flow rate of each section of the collection system, the watershed area of each section must be determined, (See sketch sheet 2).

△1 Ref. 5

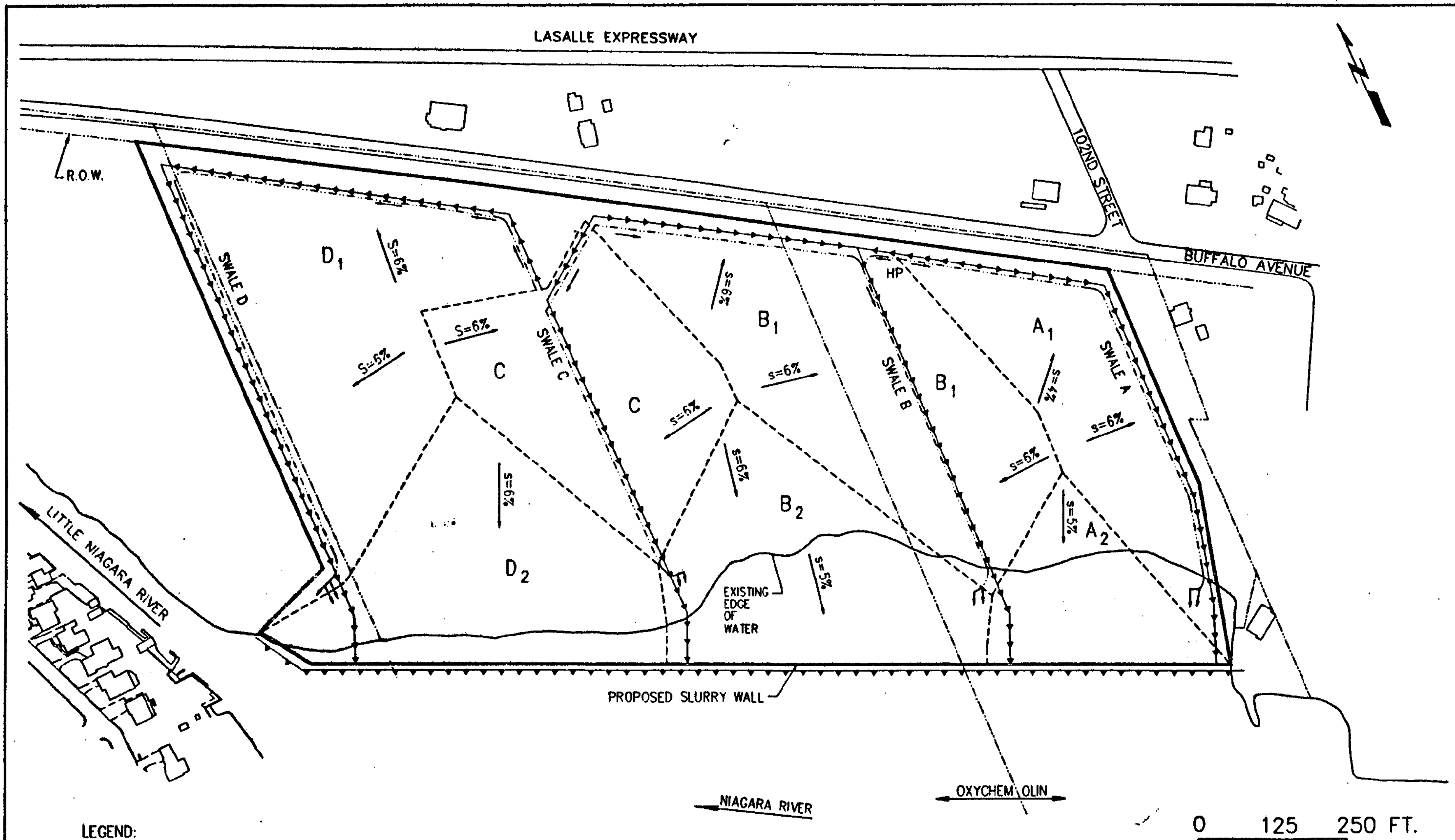
△1 Watershed Areas	△1 Peak Flow Rate*	
A_1 = 3.01 Acres	0.007 CFS	or 3.14 GPM
A_2 = 1.44 Acres	0.004	1.80
B_1 = 5.73 Acres	0.014	6.73
B_2 = 2.63 Acres	0.006	2.69
C = 4.27 Acres	0.010	4.49
D_1 = 5.43 Acres	0.013	5.83
D_2 = 3.17 Acres	0.008	3.59

$$* \frac{20.89''/yr}{12''/ft} \times \left(\text{Watershed Area} \times 43{,}560\ ft^2/\text{Acre}\right) \times \frac{1\ yr}{365\ days} \times \frac{day}{24\ hr} \times \frac{hr}{3600\ sec}$$

= Peak Flow Rate, ft³/sec (CFS)

Ref. 1

or $\times \frac{7.48}{ft^3} \times \frac{60\ sec}{min}$ for gallons/min (GPM)



LEGEND:

- PERMANENT DIVERSION SWALE
- PROPERTY LINES
- SUBSURFACE COLLECTION PIPE SYSTEM OF STORM WATER
- WATERSHED DIVIDE (WITH PERCENT DOWN SLOPE ARROWS)
- A_2 TYPICAL WATERSHED AREA LABEL

REFERENCES:

1. TOPOGRAPHICAL SURVEY OF 102nd STREET LANDFILL, McINTOSH & McINTOSH, P.C. – MAY 11, 1992

FLUOR DANIEL	STORM WATER CONTROL SITE PLAN	DATE: 01-11-93 REV: 8 PROJ No: 23594000	OXYCHEM/OLIN 102ND STREET LANDFILL SITE REMEDIAL DESIGN NIAGARA FALLS, NEW YORK

P 3/6

7-13-93

Ref. 2

check capacity of 4"Ø ADS Corrugated Polyethelene Pipe (HDPE)

Manning's $n = 0.018$

Ref. 3

Manning's Equation

$$\text{Velocity} = V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

$S = 0.005$ min.

$$R = \frac{A}{P}$$

Cross Sec A for 4"Ø = 0.087
P = 1.047

$$R = \frac{0.087}{1.047} = 0.083$$

$$V = \frac{1.49}{0.018} (0.083)^{2/3} (.005)^{1/2} = 1.11 \text{ ft/sec (fps)}$$

$$Q = AV = (0.087)(1.11) = 0.097 \text{ ft}^3/\text{sec (CFS)}$$

Flowing half full Q = 0.048 CFS

Conclusion:

4"Ø ADS adequate to handle peak flows from geonet from any one watershed or combinations shown below:

△ $D_1 + D_2 + \frac{1}{2}C = 0.026$ CFS   △ River Frontage = 695'
$\frac{1}{2}C + B_2 + \frac{1}{2}B_1 = 0.018$ CFS   " " = 590'
$A_1 + A_2 + \frac{1}{2}B_1 = 0.018$ CFS   " " = 390'

FORM E50 (Rev. 8-91)

check capacity requirements of gravel drainage media between geonet and rip rap at river.

Desired peak flow would be the higher of the combination values shown on the bottom of sheet 3 which is 0.026 CFS = 11.67 GPM = 16,804.8 gal./day ⚠

Ref. 1

Discharge through a porous media is shown as the formula

Ref. 4.

$Q = KiA$ where Q = flow rate through the media
K = hydraulic conductivity
i = hydraulic gradient
A = Cross Sectional Area

minimum gradient = 0.005
K for gravel ranges between 1×10^4 to 1×10^6 gal/day/ft²

Solve for cross sectional area required
use Avg. $K = 1\times10^5$

$$A = \frac{Q}{Ki} = \frac{16,804.8}{1\times10^5 \times .005} = 33.61\ ft^2$$ ⚠

Using a 12" thick drain media layer,
1.0' thick × River Frontage = Cross Sectional Area provided.

⚠ 1.0' × 695' = 1695.0 ft² > 34.91 ft² OK

Checking other shorter River Frontage sections:

⚠ 1.0' × 590' = 590 ft² > 34.91 ft² OK
1.0 × 390 = 890 ft² > 34.91 ft² OK

SUBSURFACE COLLECTION PIPE

REFERENCE:

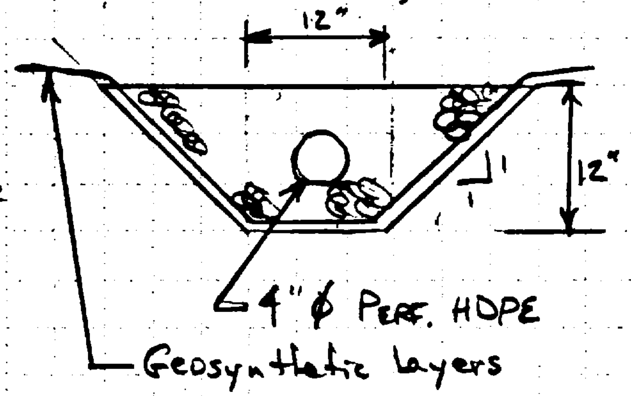
1. DWG NO. 594000-10U-02 thru 10U-05, Rev. A △1

Section Area:

$$\tfrac{1}{2}(2)(1.0\times1.0)+(1.0\times1.0)-\left(\frac{\pi\left(\frac{4}{12}\right)^2}{4}\right)$$

$$= 2.00 - .09 = 1.91\ ft^2$$

use 2.0



Quantity of Drainage Gravel
Volume = Area x Length:
(Trench sections only)

		OXYCHEM	OLIN
△1	$(2.0)(2695) = 5390\ ft^3 =$	200 CY	—
△1	$(2.0)(1950) = 3900\ ft^3 =$	—	145 CY

NOTE: ABOVE SKETCH IS TYPICAL INSTALLATION DETAIL UNDER CAP DIVERSION SWALES. 45° SIDE SLOPES SHOWN FOR PLACEMENT OF GEOSYNTHETIC LAYERS, NOT FOR REQUIRED AREA SECTION. GEONET CAN NOT BE BENT 90°.

4.0 Geomembrane Liner

5.0 Geosynthetic Clay Liner

REQUIRED THICKNESS OF CAP

References

1) Designing with Geosynthetics, Koerner, 1990
2) Slope Stability Failure Investigation, Kettleman Hills, CA, Chemical Waste Management, 1988
3) 6 NYCRR Part 360 Solid Waste Management Facilities, New York State Dept. of Environmental Conservation, May 1991.

Procedure

Use the method outlined by Koerner to determine the minimum geomembrane thickness required for the stresses found in the cap. Compare to table 5-11 and use minimum value.

Calculate Required Thickness

1) P.411

$$t = \frac{p}{\cos\beta}\,\frac{x}{\sigma_{allow}}\left(\tan\delta_u + \tan\delta_L\right)$$

Where:
ΔH = settlement mobilizing the stresses,
F = force mobilized in the liner ($= \sigma_{allow} t$),
σ_{allow} = liner allowable stress,
t = liner thickness,
T_U = shear force on top of liner (note that if water is above liner, the shear stress is zero; this is essentially the same even if a thin soil cover is above it),
T_L = shear force below liner,
$T = p \tan\delta$,
p = applied pressure from reservoir contents,
δ = angle of shearing resistance between liner and the adjacent material (i.e., soil or geotextile), and
x = distance of mobilized liner deformation.

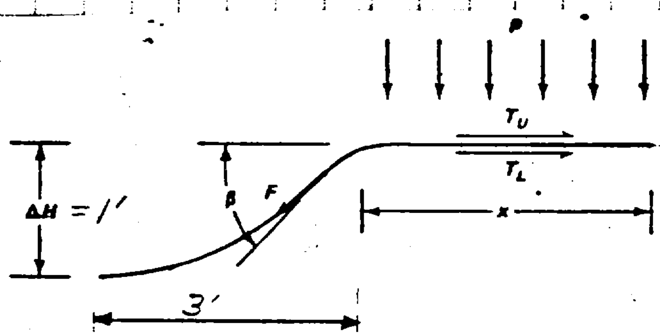


Figure 5.20 Design model used to calculate geomembrane thickness.

The general ranges of the variables in the equation above are as follows:

p = 300 to 3000 lb./ft.^{2} (≅5 to 50 ft. of water)
β = 0 to 45 deg.
x = 0.5 to 3.0 in. (determined from the laboratory test described in Section 5.1.3.9, recall Figure 5.9) mobilized dist. for liner deformation
σ_{allow} = 50,000 to 400,000 lb./ft.^{2} (determined from laboratory tests described in Sections 5.1.3.2 and 5.1.3.3)
δ_u = 0 deg. for liquid containment; 10 to 40 deg. for landfill containment (determined from laboratory tests described in Section 5.1.3.8)
δ_L = 10 to 40 deg. (determined from laboratory tests described in Section 5.1.3.8)

Assume localized settlement of one foot in 3 feet

$\tan \beta = 1/3 \Rightarrow \underline{\beta = 18.4°}$

Loading on liner = p

Two foot soil layer = 2' x 135 pcf = 270 psf
Construction Loading = Assumed surcharge = 300 psf
570 psf

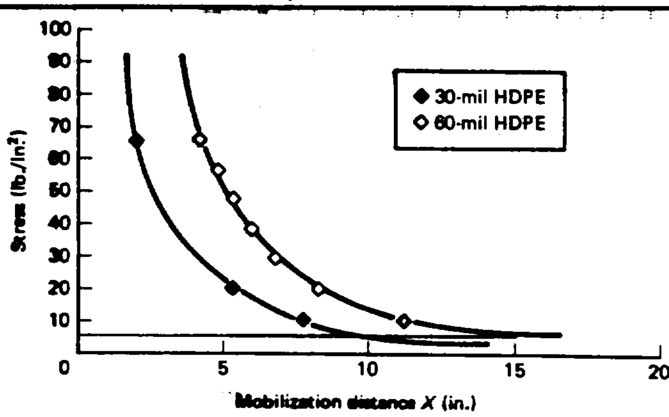
say $\underline{p = 600 \text{ psf}}$

$\underline{p = 4.2 \text{ psi}}$

1) p.385 Mobilization distance = x
Using stress loading of 4.2 psi

Using Fig 5.9

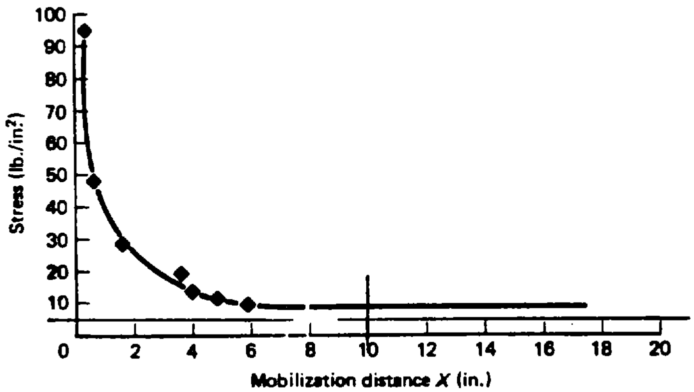
FORM E50 (Rev. 5-91)



(a) HDPE – 30 and 60 mil (0.75 and 1.50 mm)

For $p = 4.2$ psi

$X = 10$ in for 30 mil HDPE *



(c) PVC – 30 mil (0.75 mm)

$X = 10''$ for 30 mil PVC

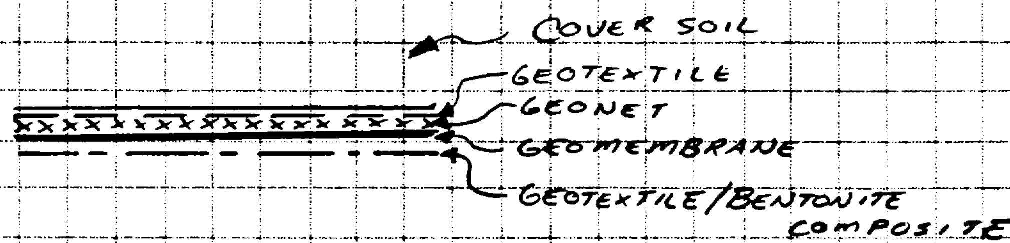
Figure 5.9 Embedment depth curves versus applied normal stress for various geomembranes.

General range = $X = 0.5$ to $3.0''$
Using a value of $10''$ adds a F.S. of 3.

* Assumption: Mobilization distance for V.L.DPE is equivalent to value for HDPE. HDPE value is very conservative.

FORM E50 (Rev. 8-91)

Friction angles δ_u & δ_L



2) δ_u = interface friction between membrane & geonet — Use 8°

1) δ_L = interface friction between membrane & GCL — Use 21° VLDPE *
p.382 Use 21° PVC

* Value for PVC
VLDPE has similar qualities to PVC.
Value for HDPE is 8°, higher value is conservative

Yield Stress — σ_{allow}
Using Figure 5.3

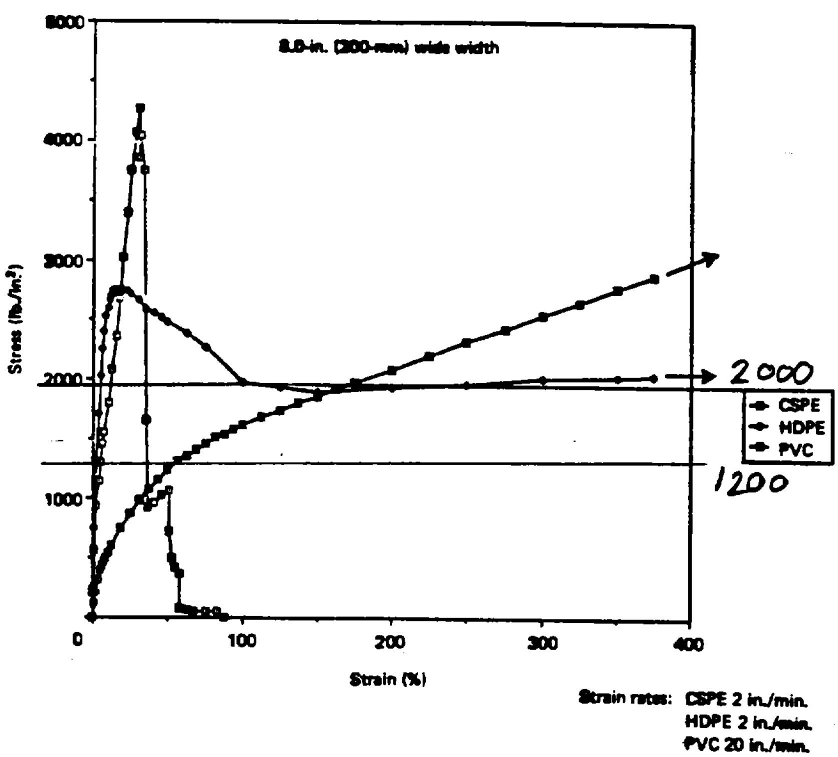


Figure 5.3 Tensile test results on 1.0-in. (25-mm) narrow-width (upper figure) and 8.0-in. (200-mm) wide-width specimens (lower figure).

HDPE
σ_{ALLOW} = 2000 * psi

PVC
σ_{ALLOW} = 1200 psi

Use 1200 psi for VLDPE to be conservative

FORM E50 (Rev. 5-91)

Summarize values:

	VLDPE	PVC
$p = 4.2$ psi		
$\beta = 18.4°$		
$x =$	10"	10"
$\sigma_{ALLOW} =$	1200 psi	1200 psi
$\delta_u =$	8°	8°
δ_L	21°	21°

$$t = \frac{p}{\cos\beta}\,\frac{x}{\sigma_{ALLOW}}\left(\tan\delta_u + \tan\delta_L\right)$$

$$t = \frac{4.2\text{ psi}}{\cos 18.4°}\,\frac{10''}{1200\text{ psi}}\left(\tan 8° + \tan 21°\right)$$

$$t = 0.019$$

$$= 19 \text{ mils}$$

3) p2-48 4 These values must now be checked against regulatory (>40 mils) and installation survivability (Table 5.11)

1) p.404

TABLE 5.11 RECOMMENDED MINIMUM PROPERTIES FOR GENERAL GEOMEMBRANE INSTALLATION SURVIVABILITY

Property and test method	Required degree of survivability			
	Low[a]	Medium[b]	High[c]	Very high[d]
Thickness (D1593) mils (mm)	20 (0.50)	25 (0.63)	$\boxed{30\ (0.75)}$	40 (1.00)

(a) *Low* refers to careful hand placement on very uniform well-graded subgrade with light loads of a static nature—typical of vapor barriers beneath building floor slabs.

(b) *Medium* refers to hand or machine placement on machine-graded subgrade with medium loads—typical of canal liners.

(c) *High* refers to hand or machine placement on machine-graded subgrade of poor texture with high loads—typical of landfill liners and covers.

(d) *Very high* refers to hand or machine placement on machine-graded subgrade of very poor texture with high loads—typical of reservoir covers and liners for heap leach pads.

The required 19 mils is less than both the regulatory & survivability thicknesses, therefore 40 mils is the design value.

∴ Use 40 mil cap geomembrane

FORM E50 (Rev. 5-91)

GUNDLINE® VL (VLDPE) SPECIFICATIONS

Gundline VL is a special formulation of very low density polyethylene containing approximately 97.5% polymer and 2.5% carbon black, anti-oxidants and heat stabilizers.

TYPICAL PROPERTIES*	TEST METHOD	GAUGE (NOMINAL) 20 mil (0.5 mm)	30 mil (0.75 mm)	40 mil (1.0 mm)	60 mil (1.5 mm)	80 mil (2.0 mm)	100 mil (2.5 mm)
Tensile Properties. (Typical)							
1. Tensile Strength at Break (Pounds/inch width)	ASTM D638 Type IV Dumb-bell at 2 ipm. 2 inch gauge length, 2.5 inch grip separation	70	105	140	210	280	350
2. Elongation at Break (Percent)		900	900	900	900	900	900
Puncture Resistance. Pounds. (Typical)	FTMS 101 Method 2065	38	51	64	72	80	88
Tear Resistance Initiation. Pounds. (Typical)	ASTM D1004 Die C	8	12	16	24	32	40
Dimensional Stability. % Change. Each Direction. (Max.)	ASTM D1204 212°F 1 hr.	±2	±2	±2	±2	±2	±2
Low Temperature Brittleness. °F (Typical)	ASTM D746M Procedure B	−112	−112	−112	−112	−112	−112
Resistance to Soil Burial. Percent change in original value. (Typical) Tensile Strength at Break.	ASTM D3083 Type IV Dumb-bell at 2 ipm	±10	±10	±10	±10	±10	±10
Environmental Stress crack. Hours. (Min.)	ASTM D1693 10% Igepal, 50°C	1500	1500	1500	1500	1500	1500

*Note: All values, except when specified as minimum or maximum, are typical test results.

SUPPLY SPECIFICATIONS

The following describes typical roll dimension for Gundline VL

THICKNESS mil	mm	WIDTH ft.	m	LENGTH ft.	m	AREA ft.²	m²	ROLL WEIGHT lb.	kg.
20	0.5	22.5	6.86	1250	381	28,125	2613	2800	1272
30	0.75	22.5	6.86	840	256	18,900	1756	2800	1272
40	1.0	22.5	6.86	650	198	14,625	1359	2800	1272
60	1.5	22.5	6.86	420	128	9,450	878	2800	1272
80	2.0	22.5	6.86	320	98	7,200	670	2800	1272
100	2.5	22.5	6.86	250	76	5,625	522	2800	1272

GUNDLINE VL is rolled on 6" I.D. hollow cores. Each roll is provided with 2 slings to aid handling on site. Dimensions and weights are approximate. Custom lengths available upon request.

Gundle Lining Systems Inc.

Gundle®

19103 Gundle Road
Houston, Texas 77073 U.S.A.
Phone: (713) 443-8564
Toll Free: (800) 435-2008
Telex: 166657 GundleHou
Fax: (713) 875-6010

These specifications are offered as a guide for consideration to assist engineers with their specifications; however, Gundle assumes no liability in connection with the use of this information. The specifications on this data sheet are subject to change without notice.

PVC OXYFLEX® PVC GEOMEMBRANE LINER 40 MIL Data Sheet

OxyChem

Property	Test Method	Required NSF 54*	Typical** Values
Gauge (nominal)	-----------	40	
Thickness, Mills (minimum)	ASTM D1593	38	39.5-40.5
Specific Gravity (minimum)	ASTM D792	1.20	1.280
Minimum Tensile Properties (each direction)	ASTM D882		
1. Breaking Factor (pounds/inch width)	Method A (1 inch wide)	92	MD 120 TD 115
2. Elongation at break (%)	Method A (2" jaw separation)	350	MD 500 TD 550
3. Modulus (force) at 100% elongation (pounds/inch width)	Method A	36	MD 60 TD 55
Tear Resistance (pounds, minimum)	ASTM D1004 Die C	10	MD 15.0 TD 16.0
Low Temperature, °F	ASTM D1790	-20	Pass
Dimensional Stability (each direction, % change maximum)	ASTM D1204 212°F, 15 min.	Less than 5%	Pass
Water Extraction (% loss maximum)	ASTM D3083 (as modified in Appendix A)	0.35	0.11
Volatile Loss (% loss maximum)	ASTM D1203 Method A	0.5	0.40
Resistance to Soil Burial (% change maximum in original value)	ASTM D3083 (as modified in Appendix A)		
1. Breaking Factor		±5%	Pass
2. Elongation at break		±20%	Pass
3. Modulus at 100% elongation		±20	Pass
Hydrostatic Resistance (pounds/sq. in. min.)	ASTM D751 Method A	82 (110)***	157

Occidental Chemical Corporation
Vinyls Division
5005 LBJ Freeway
Dallas, Texas 75244
214/404-3800

*May 1991 revision
***(Proposed Value)

COMPACTED CLAY VERSUS A GEOSYNTHETIC CLAY LINER

REFERENCES

1. MANUFACTURER'S SPECIFICATIONS / TEST RESULTS (CLAYMAX, BENTOMAT, BENTOFIX, GUNDSEAL)

2. [illegible]

#1. Given: GCL $k_{MAX} = 5 \times 10^{-9}$ cm/s

$t = 0.5''$ (conservative hydrated thickness)

ASSUME: COMPACTED CLAY LAYER: $K = 10^{-6}$ cm/s

EQUIV. CLAY THICKNESS: $t_{CLAY} = t_{GCL} \left(\dfrac{k_{CLAY}}{k_{GCL}} \right)$

$$= 0.5'' \left(\frac{10^{-6} \left(\frac{cm}{s}\right)}{5 \times 10^{-9} \left(\frac{cm}{s}\right)} \right) \left[\frac{1'}{12''} \right]$$

$$= \underline{8.3'}$$

CLAYMAX GEOSYNTHETIC CLAY LINER MATERIAL SPECIFICATIONS

	PROPERTY	TEST METHOD *	UNITS	CLAYMAX STYLE 200R	CLAYMAX STYLE 500 SP
BENTONITE PROPERTIES (1)	SODIUM MONTMORILLONITE CONTENT	X-RAY DIFFRACTION	%	90 (TYP)	90 (TYP)
	FREE SWELL	USP-NF-XVII	ml	27 (MARV)	27 (MARV)
	FLUID LOSS	API 13 B	ml	12 (MAX. A.R.V.)	12 (MAX. A.R.V.)
	MOISTURE CONTENT (3)	ASTM D4643	%	20 (TYP)	20 (TYP)
ADHESIVE	ADHESION	VISUAL		CONTINUOUS ADHESION TO BACKING MATERIAL	CONTINUOUS ADHESION TO BACKING MATERIAL
PHYSICAL PROPERTIES	THICKNESS (excluding fabrics)	ASTM D1777	inches	0.17 (MARV)	0.17 (MARV)
	COMPOSITE THICKNESS	ASTM D1777	inches	0.20 (MARV)	0.20 (MARV)
	WIDE WIDTH TENSILE	ASTM D4595	PPI	60 (TYP) (2)	100 (TYP)
	GRAB TENSILE	ASTM D4632	LB	90 (MARV) (2)	90 (MARV) (2)
	BENTONITE CONTENT (3) • 20% moisture	WEIGH 12" X ROLL WIDTH	LB/SF	0.95 (MARV)	0.95 (MARV)
	SHEAR RESISTANCE	ASTM D35.01.81.07 (DRAFT)			
	HYDRATED		DEG	>10	>40
	DRY		DEG	>35	>40
HYDRAULIC PROPERTIES	PERMEABILITY				
	A) 2 psi EFFECTIVE STRESS	ASTM D5084	CM/S	5×10^{-9} (MAX.A.R.V.)	5×10^{-9} (MAX.A.R.V.)
	B) 30 psi EFFECTIVE STRESS	ASTM D5084	CM/S	$< 5 \times 10^{-10}$ (TYP)	$< 5 \times 10^{-10}$ (TYP)
	PERMEABILITY (2 psi effective stress)				
	C) 2" OVERLAPPED CLAYMAX (without the use of granular bentonite between the seams)	ASTM D5084	CM/S	$< 5 \times 10^{-9}$ (TYP)	$< 5 \times 10^{-9}$ (TYP)
	D) DAMAGED CLAYMAX (3 each, 1" holes)	ASTM D5084	CM/S	$< 5 \times 10^{-9}$ (TYP)	N/A (4)
	CLAYMAX underneath damaged HDPE geomembrane (1" hole)	ASTM D5084	CM/S	$< 5 \times 10^{-9}$ (TYP)	N/A (4)
	F) AFTER 3 WET/DRY CYCLES	ASTM D5084	CM/S	$< 5 \times 10^{-9}$ (TYP)	N/A (4)
	G) AFTER 5 FRZ/THAW CYCLES	ASTM D5084	CM/S	$< 5 \times 10^{-9}$ (TYP)	N/A (4)

** See opposite page for footnotes.

SUMMARY OF BENTOMAT TRIAXIAL PERMEABILITY TEST DATA

Test Lab[1]	Date	Product	Max. Effective Conf. Stress (psi)	Head(ft)	Gradient	Total Time(hrs)	Permeability(cm/sec)
J & L	07-05-90	SS	8.2	1	30	26	2.1×10^{-9}
			10.6	12	380	6	7.5×10^{-10}
			13.2	24	760	13	5.8×10^{-10}
			15.6	35	1100	4	6.6×10^{-10}
		CS	8.2	1	35	62	5.6×10^{-9}
			10.6	12	450	19	1.1×10^{-9}
			13.2	24	900	46	9.8×10^{-10}
			15.6	35	1315	2	2.6×10^{-9}
J & L	09-21-90	SS	8.2	01	30	36	7.3×10^{-10} [2]
			10.6	12	400	28	7.3×10^{-10}
			13.2	24	800	7	1.4×10^{-9}
J & L	12-17-90	CS	NA	30	2250	25	1.4×10^{-9}
GeoSyntec	12-20-90	CS	NA	15	NA	72	2.0×10^{-9}
J & L	01-08-91	CS	18.7	35	1800	25	1.6×10^{-9}
ACC	05-02-91	CS	30	7	200	216	3.6×10^{-9} [3]
J & L	07-05-91	SS	37	9	530	4	2.1×10^{-10}

(Continued next page)

CS – VOLCLAY BENTONITE
SS TREATED VOLCLAY

P4/64

GCLs are part of an important trend toward the use of geosynthetics in containment applications. In a typical double-composite liner system, GCLs have combined with polyethylene geomembranes to decrease permeability and increase efficiency.

Assured quality control

Since Bentofix GCLs are a manufactured product, detailed quality control can be performed in the manufacturing plant. As a result, expensive and time-consuming on-site quality control testing is not necessary.

Multi-lift clay liners, on the other hand, require time-consuming compaction and detailed on-site testing to meet stringent moisture content and density criteria.

Needlepunching gives Bentofix GCLs distinct advantages.

By needlepunching fibers from the non-woven geotextile layer through the sodium bentonite and into the opposite geotextile layer, a completely uniform GCL — with clear advantages — is produced.

- **High internal shear resistance (27° to 34°)** of needlepunched GCLs eliminates potential planes of weakness within the composite in the hydrated state. When hydrated, other GCLs which are not needlepunched have a slip plane which is susceptible to failure under low shear stress, since the inner bentonite layer has little shear strength.

  With Bentofix GCLs, on the other hand, the needlepunched fibers provide the shear strength needed to transfer the critical friction plane out of the bentonite layer.

  This high shear resistance allows Bentofix to be installed successfully on steep slopes. In all applications, design-specific parameters will determine actual values, and site specific testing should be carried out to determine the shear angle in each application.

- **A consistent bentonite content** is preserved throughout the plane of the composite, because needlepunching prevents lateral migration of the bentonite material within the GCL — in either a dry or hydrated state.

- **During installation, the needlepunched fibers hold the sodium bentonite in place,** even after hydration of the GCL.

- **The inherent confining stress** caused by needlepunching decreases permeability and prevents delamination of the geotextiles when the bentonite hydrates.

Bentofix
GEOSYNTHETIC CLAY LINERS

Specifications

Property	Standard	Specified Values
BENTONITE Bentonite Content	Minimum	1.0 lb/ft^2
Water Permeability	Maximum	1.0 x 10^{-9} cm/sec
Volumetric Increase (Free Swell)	Minimum	300%
GEOTEXTILE UPPER LAYER Weight	Typical	3.25 oz/yd^2
GEOTEXTILE BOTTOM LAYER (NONWOVEN) Weight	Typical	6.5 oz/yd^2
ROLL SIZE Width	Nominal	15 ft
Length	Nominal	100 ft

LANDFILL CAP SECTION STABILITY

OBJECTIVE: ANALYZE SLOPE STABILITY OF THE FINAL COVER AT 3-6% SLOPES

GIVEN: TYPICAL CROSS SECTION AND DETAILED COMPONENTS OF COVER SHOWN ON FOLLOWING SHEETS

ASSUMPTIONS: UNIT WEIGHT OF COVER SOIL = 125 PCF (γ)
FRICTION ANGLE OF COVER SOIL = 25° (ϕ)
SOIL SLOPE = 6 % (i.e 100 H: 6 V OR 16.6 : 1) (ω=3.43)
L = 600 FT, H = 2.0 FT, C_a = 0, c = 0300 lb/FT^2
CASE 1 $\delta = 5^\circ - 8^\circ$ (BETWEEN GEONET & GEOMEMBRANE) (REF. 2)
CASE 2 $\delta = 18.2^\circ$ (BETWEEN PVC & TENSAR DN-3 DRAINAGE NET)

METHOD: SLIDING WEDGE METHOD

REFERENCES:

(1) DESIGN AND CONSTRUCTION OF RCRA/CERCLA FINAL COVERS
MAY 1991, EPA/625/4-91/025
BY: EASTERN RESEARCH GROUP, INC.
6 WHITTEMORE STREET
ARLINGTON, MA 02174

(2) "SLOPE STABILITY FAILURE INVESTIGATION: LANDFILL UNIT B-19, PHASE I-A"
CHEMICAL WASTE MANAGEMENT, INC., FACILITY
KETTLEMAN HILLS, CA.
RAYMOND SEED, JAMES MITCHELL, H. BOLTON SEED.
JUNE 29, 1988

(3) "AN INNOVATION IN VALUE ENGINEERING FOR THE ENVIRONMENTAL CONTAINMENT INDUSTRY"

POLY-FLEX DIVISION, POLY-CO INC.

GEORGE YAZDANI & JIM NOBERT

INCLUDING INDEPENDENT STUDIES BY DR. ROBERT KOERNER, P.E.

REFERENCE	CALCULATIONS
	CASE 1
REF-2	$\delta = 5^\circ - 8^\circ$ (BETWEEN GEONET & GEOMEMBRANE)
	$a = 0.5\ \gamma LH \sin^2 2\omega$
	$= 0.5(125)(600)(2)\sin^2(2)(3.43^\circ)$
	$= 0.5(125)(600)(2)(0.014266806) = \underline{1070}$ lb/FT.
	$b = -[\gamma LH\cos^2\omega \tan\delta \sin(2\omega) + C_a L\cos(\omega)\sin(2\omega) + \gamma LH\sin^2(\omega)\tan(\phi)\sin(2\omega) + 2CH\cos(\omega) + \gamma H^2\tan(\phi)]$
	$= -[(125)(600)(2)\cos^2(3.43^\circ)\tan(8^\circ)\sin(2\times3.43^\circ) + 0 + (125)(600)(2)\sin^2(3.43^\circ)\tan(25^\circ)\sin(2\times3.43^\circ) + 0 + (125)(2)^2\tan(25^\circ)]$
	$= -[(125)(600)(2)(0.996420486)(0.140540835)(0.119443734) + 0 + (125)(600)(2)(0.003579514)(0.466307658)(0.119443734) + 0 + (125)(4)(0.466307658)]$
	$= -[2508.99 + 0 + 29.90 + 0 + 233.15] = \underline{-2772}$ lb/FT
	$c = (\gamma LH\cos(\omega)\tan(\delta) + C_a L)(\tan(\phi)\sin(\omega)\sin(2\omega))$
	$= [(125)(600)(2)\cos(3.43^\circ)\tan(8^\circ) + 0][\tan(25^\circ)\sin(3.43^\circ)\sin(2\times3.43^\circ)]$

FORM E50 (Rev. 8-91)

$$C = \left[(125)(600)(2)(0.998208638)(0.140540835)\right]\left[(0.461307658)(0.059829043)(0.119443734)\right]$$

$$= \left[(21043.36132)\right]\left[0.00333233\right] = \underline{70} \text{ lb/FT}$$

REF-1

$$F.S = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$= \frac{-(-2772) \pm \sqrt{(-2772)^2 - 4(1070)(70)}}{2(1070)} = \underline{2.57} \quad \text{O.K}$$

CASE 2

REF-3

$\delta = 18.2^\circ$ (PVC & TENSAR DN-3 DRAINAGE NET)

$a = 1070$ lb/FT

$b = -6133$ lb/FT

$c = 164$ lb/FT

REF-1

$$F.S = \frac{-(-6133) \pm \sqrt{(-6133)^2 - 4(1070)(164)}}{2(1070)} = \underline{5.70} \quad \text{O.K}$$

FORM E50 (Rev. 8-91)

REF. 1

Model #1: Stability of Cover Soil Above a Geomembrane

Consider a cover soil (usually a permeable soil like gravel, sand or silt) placed directly on a geomembrane at a slope angle of "ω". Two discrete zones can be visualized as seen in Figure 3. Here one sees a small passive wedge resisting a long, thin active wedge extending the length of the slope. It is assumed that the cover soil is a uniform thickness and constant unit weight. At the top of the slope, or at an intermediate berm, a tension crack in the cover soil is considered to occur thereby breaking communication with additional cover soil at higher elevations.

Resisting the tendency for the cover soil to slide is the adhesion and/or interface friction of the cover soil to the specific type of underlying geomembrane. The values of "c_a" and "δ" must be obtained from a simulated laboratory direct shear test as described earlier. Note that the passive wedge is assumed to move on the underlying cover soil so that the shear parameters "c" and "ϕ", which come from soil-to-soil friction tests, will also be required.

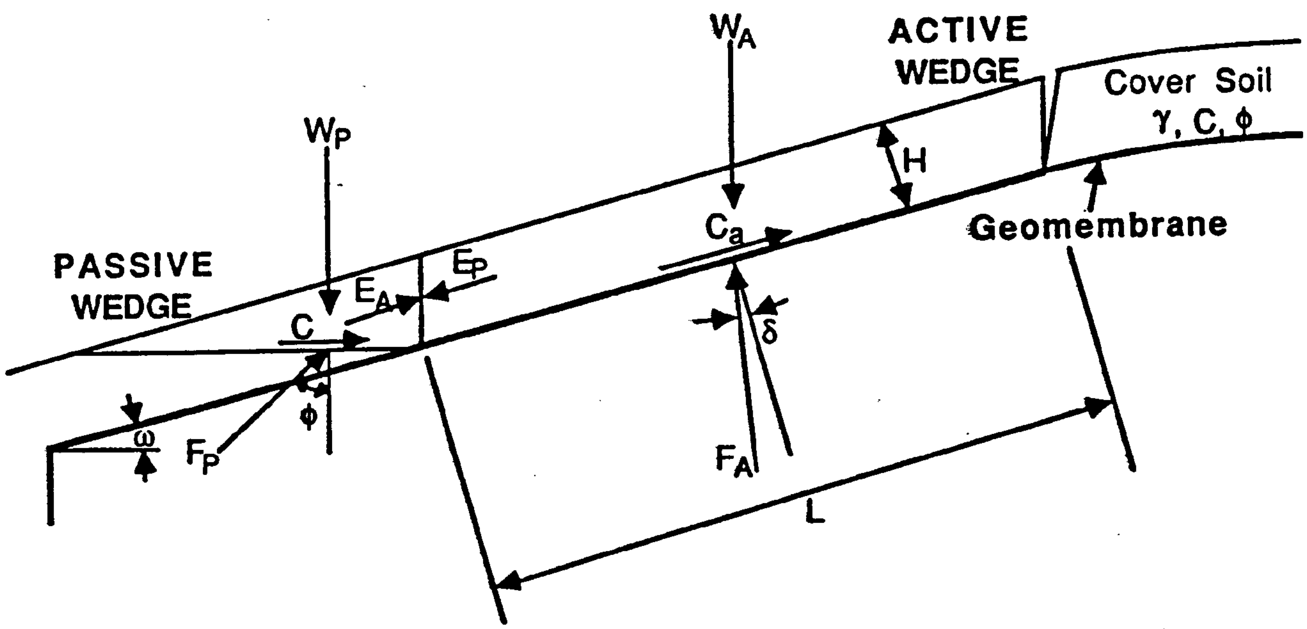


Figure 3 - Cross Section of Cover Soil on a Geomembrane Illustrating the Various Forces Involved on the Active and Passive Wedges

By taking free bodies of the passive and active wedges with the appropriate forces being applied, the following formulation for the stability factor-of-safety results, see Equation 3. Note that the equation is not an explicit solution for the factor-of-safety (FS), and must be solved indirectly. The complete development of the equation is given in Appendix "A".

$$(FS)^2\,[0.5\,\gamma\,LH\sin^2(2\,\omega)] - (FS)\,[\gamma\,LH\cos^2\omega\tan\delta\sin(2\,\omega) + c_a L\cos\omega\sin(2\,\omega)$$
$$+\gamma\,LH\sin^2\omega\tan\phi\sin(2\,\omega) + 2\,c\,H\cos\omega + \gamma H^2\tan\phi]$$
$$+\,[(\gamma LH\cos\omega\ \tan\delta + c_a L)\,(\tan\phi\sin\omega\sin(2\,\omega)] = 0 \qquad (3)$$

Using $ax^2 + bx + c = 0$, where

$$a = 0.5\,\gamma\,LH\sin^2 2\omega$$
$$b = -[\,\gamma\,LH\cos^2\omega\tan\delta\sin(2\omega) + c_a L\cos\omega\sin(2\,\omega)$$
$$+\,\gamma\,LH\sin^2\omega\tan\phi\sin(2\omega) + 2cH\cos\omega + \gamma\,H^2\tan\phi]$$
$$c = (\gamma\,LH\cos\omega\tan\delta + c_a L)\,(\tan\phi\ \sin\omega\sin(2\omega))$$

the resulting factor-of-safety is as follows:

$$FS = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} \qquad (4)$$

When the calculated factor-of-safety value falls below 1.0, a stability failure of the cover soil sliding on the geomembrane is to be anticipated. However, it should be recognized that seepage forces, seismic forces and construction placement forces have not been considered in this analysis and all of these phenomena tend to lower the factor-of-safety. Thus a value of greater than 1.0 should be targeted as being the minimum acceptable factor-of-safety. An example problem illustrating the use of the above equations follows:

Example Problem: Given a soil cover soil slope of $\omega = 18.4°$ (i.e., 3 to 1), $L = 300$ ft., $H = 3.0$ ft, $\gamma = 120$ lb/ft^3, $c = 300$ lb/ft^2, $c_a = 0$, $\phi = 32°$, $\delta = 14°$, determine the resulting factor-of-safety

Solution:

$$a = 0.5\,(120)\,(300)\,(3)\sin^2(36.8°)$$
$$= 19{,}400 \text{ lb/ft}$$
$$b = -[(120)\,(300)\,(3)\cos^2(18.4°)\tan(14°)\sin(36.8°)$$
$$+\,0 + (120)\,(300)\,(3)\sin^2(18.4°)\tan(32°)\sin(36.8°)$$
$$+\,2\,(300)\,(3)\cos(18.4°) + 120\,(9)\tan(32°)]$$

6.0 Cap Stability

LANDFILL CAP STABILITY @ BUFFALO AVE.

Ref. #

Objective: Analyze slope stability of the final cover for 3.3:1 slopes at Buffalo Avenue. Determine req'd friction angle.

Given: Typical Detail of Landfill Toe at Buffalo Avenue – Nov. 2, 1992 version

References:

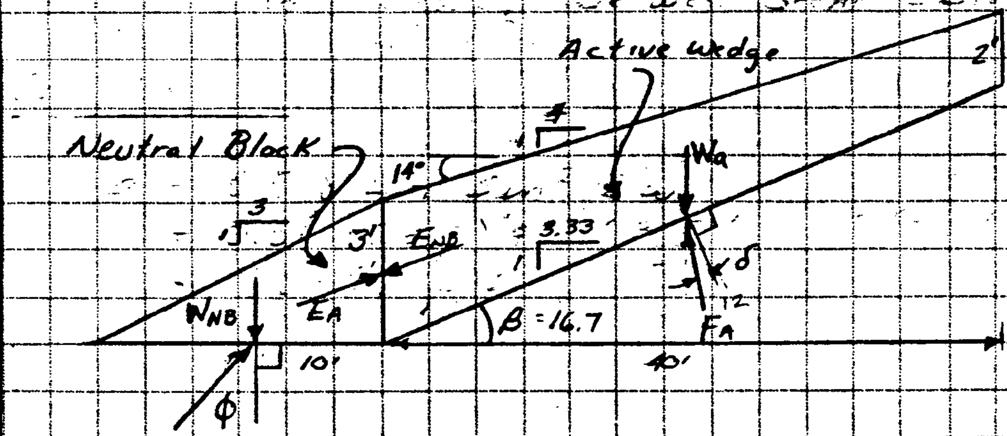
1) Design and Construction of RCRA/CERCLA Final Covers, U.S. Environmental Protection Agency, EPA/625/4-91/025, May 1991
2) Designing with Geosynthetics, 2nd Ed. Koerner, R.M., 1990
3) Direct Shear Test Results, by GeoServices, Inc.

Method: Sliding Wedge Method

Because the proposed section is tapered (thicker at bottom), the solution of an infinite slope discussed in Ref. 1 cannot be used. A wedge method utilizing force polygons can be used.

② p. 421

② p. 420



FORM E50 (Rev. 8-91)

Assumptions:

- Unit Weight of Cover Soil = 125 PCF = γ
- Neutral Block will slide at its base, along an internal geo layer surface, instead of through the soil as shown in Koerner, P. 420.
  Therefore $\phi = \delta \Rightarrow$ To be determined
- Ea is parallel to cover slope (14°)

Calcs

$$\beta = \tan^{-1}\frac{1}{3.33} = 16.7°$$

$$W_a = 40' \times \frac{2+3'}{2} \times 125\,pcf = 12500\ p/f$$

$$W_{NB} = 1/2 \times 10' \times 3' \times 125\,pcf = 1875\ p/f$$

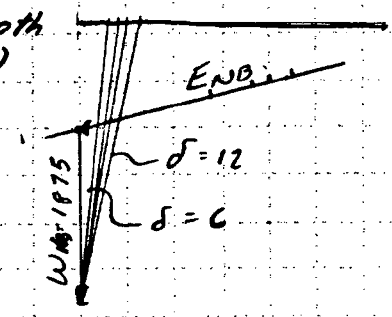
$\delta = 12°$ starting δ

	δ
FS = 1	12
FS = 1.25	9.6
FS = 1.5	8
FS = 2	6

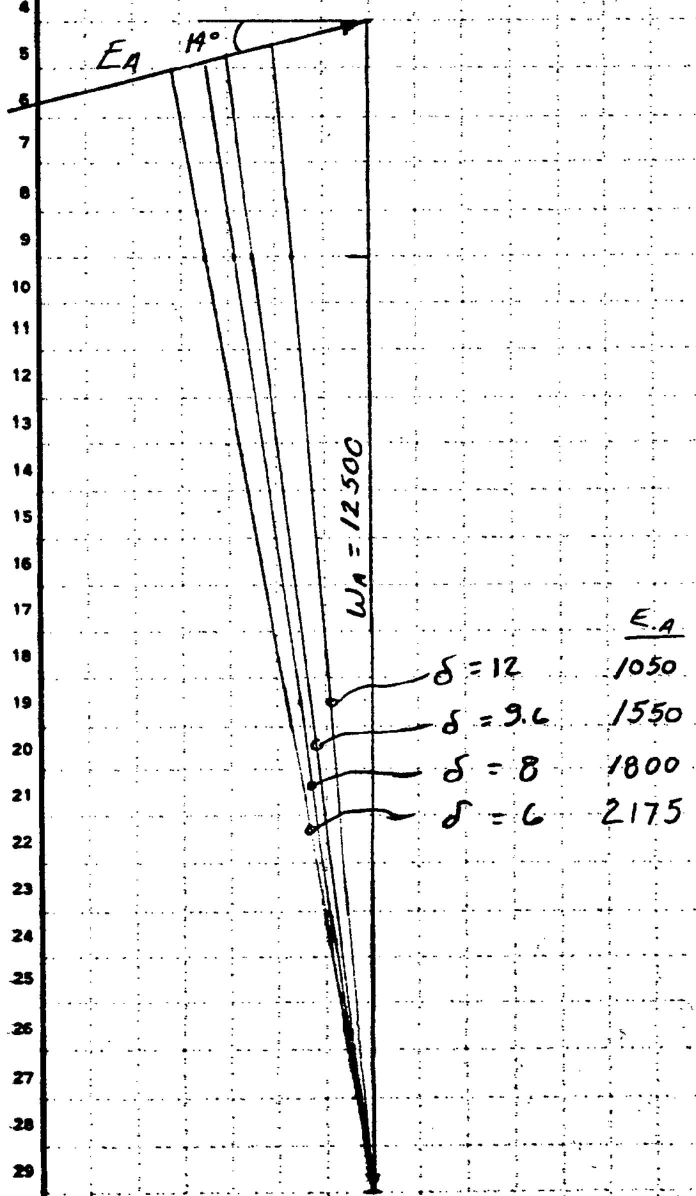
$\delta = 21.4$

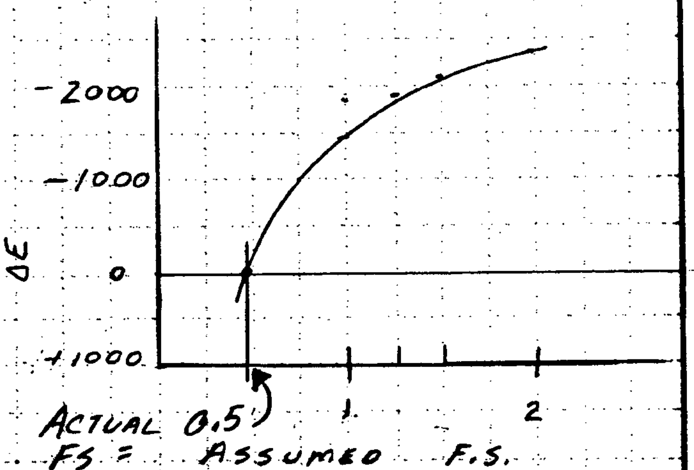
	δ
FS = 1	21.4
FS = 1.25	17.1
FS = 1.5	14.3
FS = 2	10.7

Force polygons using $\delta = 12°$
(corresponds to wet Claymax & smooth HDPE)



Neutral Block

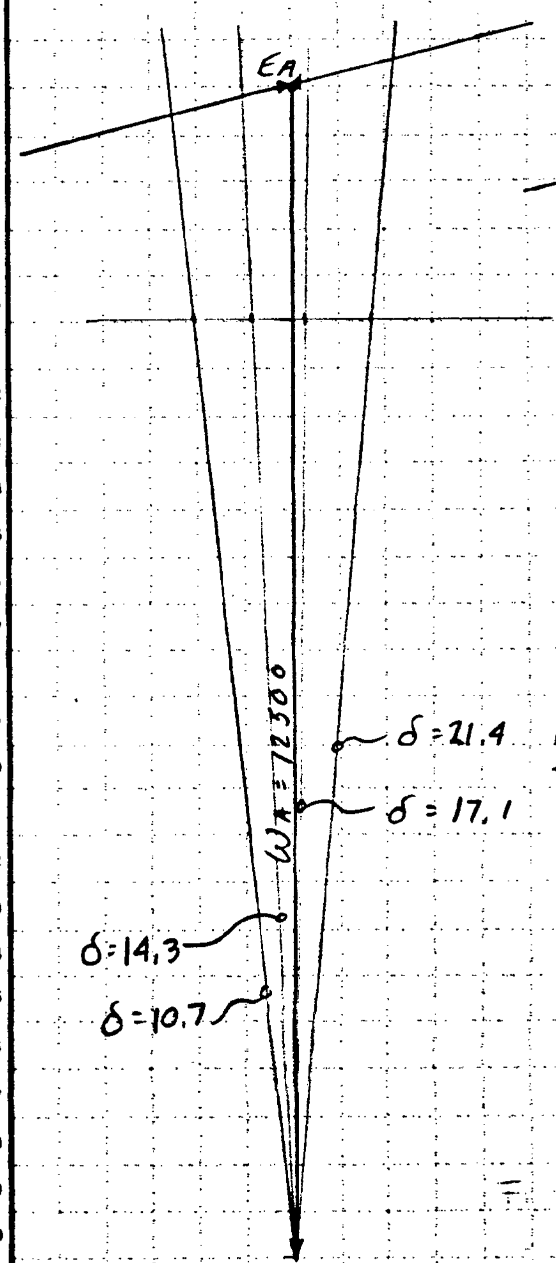


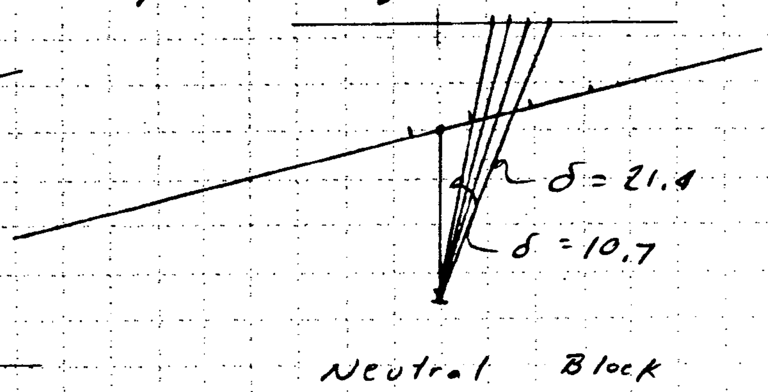


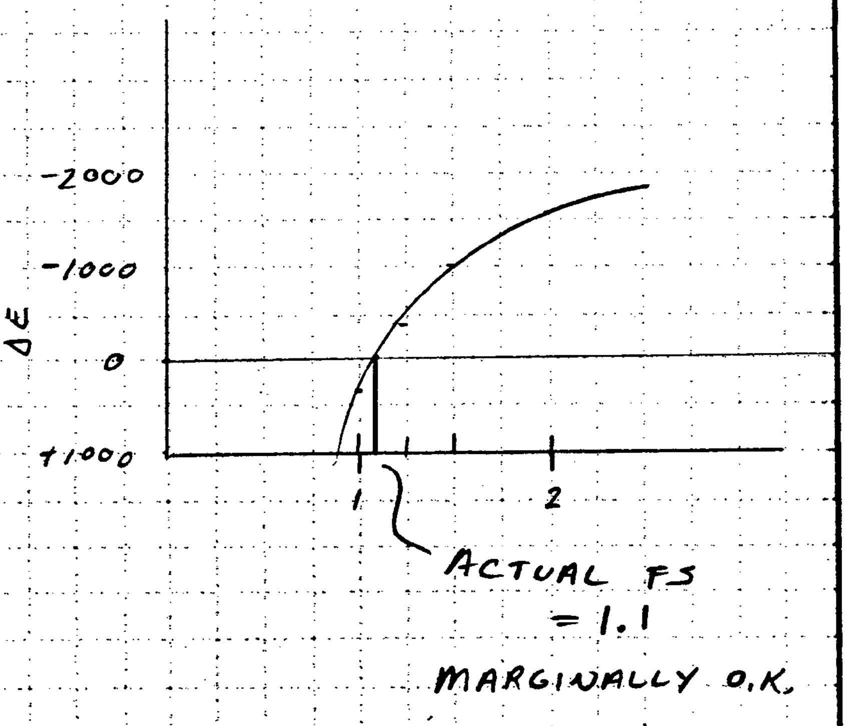
Calculated F.S < 1

Active Wedge
Force Polygon
1" = 2000'

Force polygons using $\delta = 21.4°$
(corresponds to wet Claymax & Roughed HDPE)







To achieve an acceptable FS, try a required minimum interface friction angle of 27°.

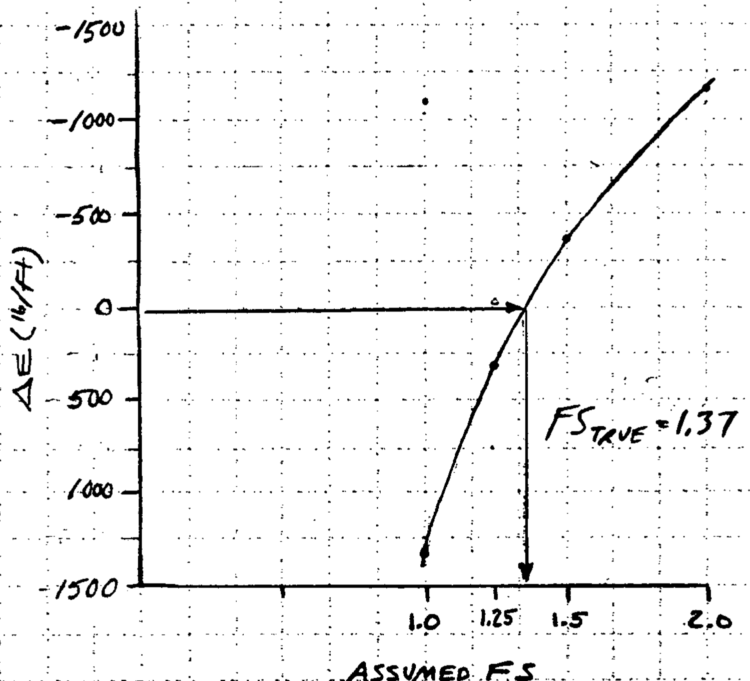
REFS.

Try 27° interface friction angle (δ).

Use force polygons to determine FS for $\delta = 27°$
(drawn on following page)

#2 p423

Plot ΔE (lb/ft) vs. Assumed FS

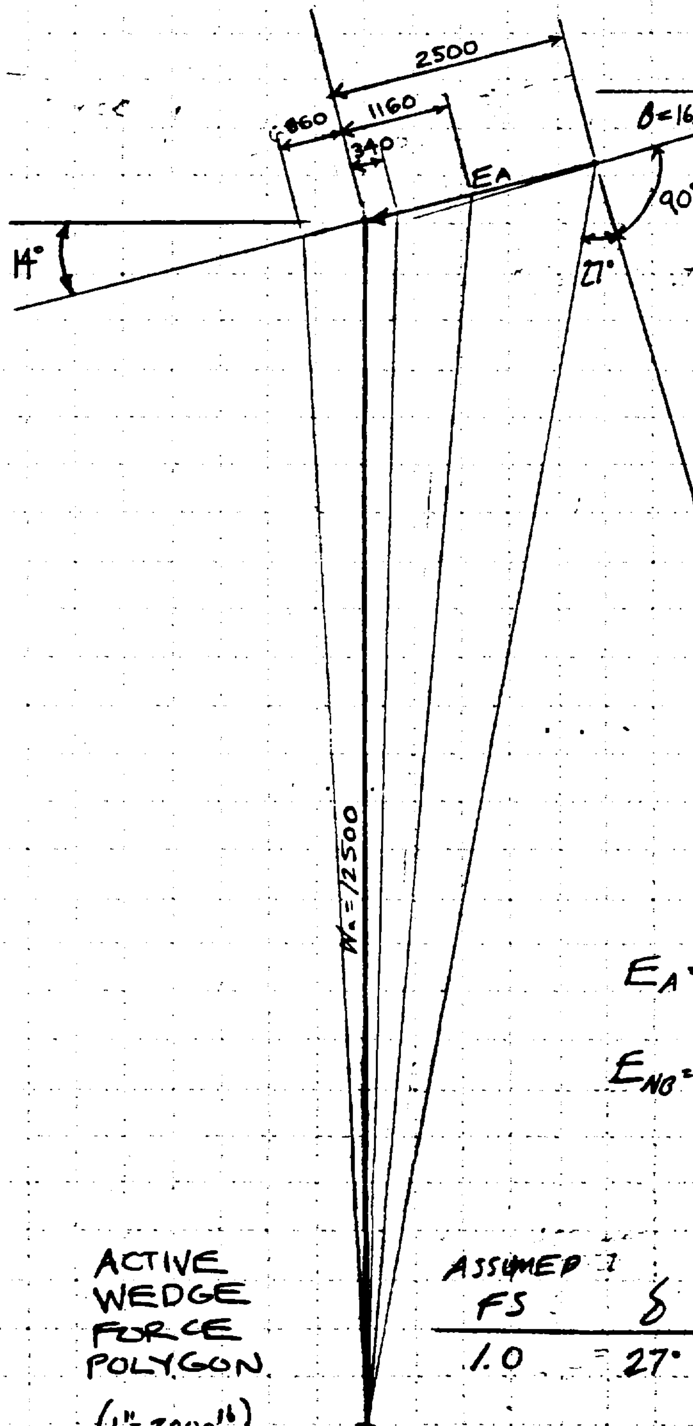


FOR AN INTERFACE FRICTION ANGLE (δ) OF 27° → FS = 1.37

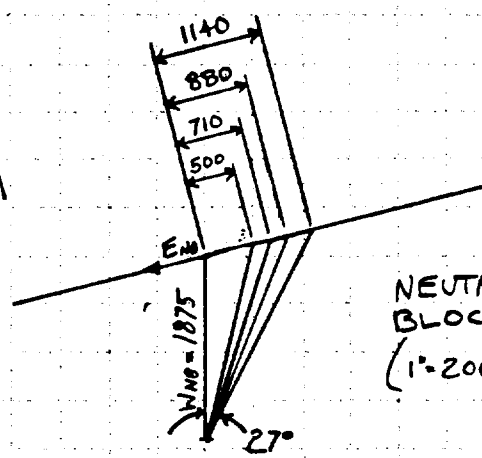
REFS.

#2

p.423



ACTIVE WEDGE FORCE POLYGON (1"=2000lb)



NEUTRAL BLOCK (1"=2000lb)

E_A = FORCE FROM NEUTRAL BLOCK ACTING ON ACTIVE WEDGE

E_{NB} = FORCE FROM ACTIVE WEDGE ON NEUTRAL BLOCK

ASSUMED FS	δ	E_A	E_{NB}	ΔE
1.0	27°	2500	-1140	-1360
1.25	21.6°	1160	880	-280
1.5	18°	340	710	-370
2.0	13.5°	-660	500	-1160

Conclusions

The critical elements of the design are the angle of the slope, and the interfacial friction angles between the elements of the cap sandwich. To provide a factor of safety against sliding, the minimum interfacial friction angle between any elements of the cap should be 27°

Note for drawing regarding cap section at Buffalo Avenue.

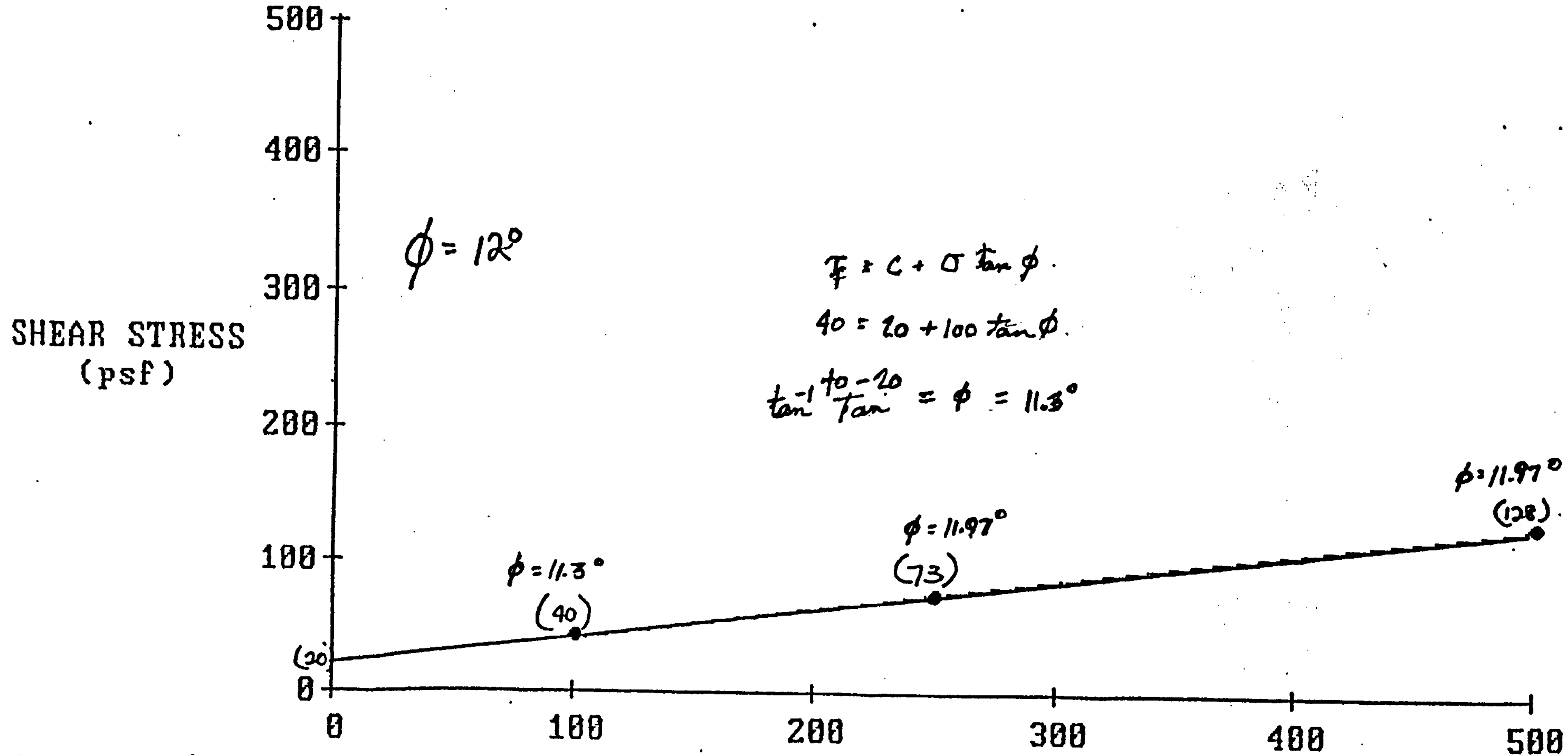
The proposed materials for the 3.33H to 1V cap slope along Buffalo Avenue should be submitted by the Contractor for direct shear testing. A minimum friction angle of 27 degrees must be obtained using the proposed geosynthetic materials and site specific soils. Testing shall meet the following requirements:

* Submit a composite of the proposed components of the cap system including select cover fill, geotextile/geonet/geotextile drainage composite, 40 mil textured VLDPE geomembrane, hydrated geosynthetic clay liner, and subbase material for direct shear testing.
* Follow the procedures for Direct Shear testing for geosynthetic systems as described in ASTM Standard Test Method D5321 Determining the Coefficient of Soil and Geosynthetic Friction by the Direct Shear Method. Use a minimum 12" x 12" shear box.
* Three points on the Mohr-Coulomb envelop are required. Use normal stresses of 150,300 & 600 p.s.f.
* Hydrate the geosynthetic clay liner under the three previosly specified normal stresses, transfer the GCL to the direct shear box, reestablish the normal stress, and then perform the shear.

* **Manufacturer's test data is not sufficient.**

FORM E50 (Rev. 8-91)

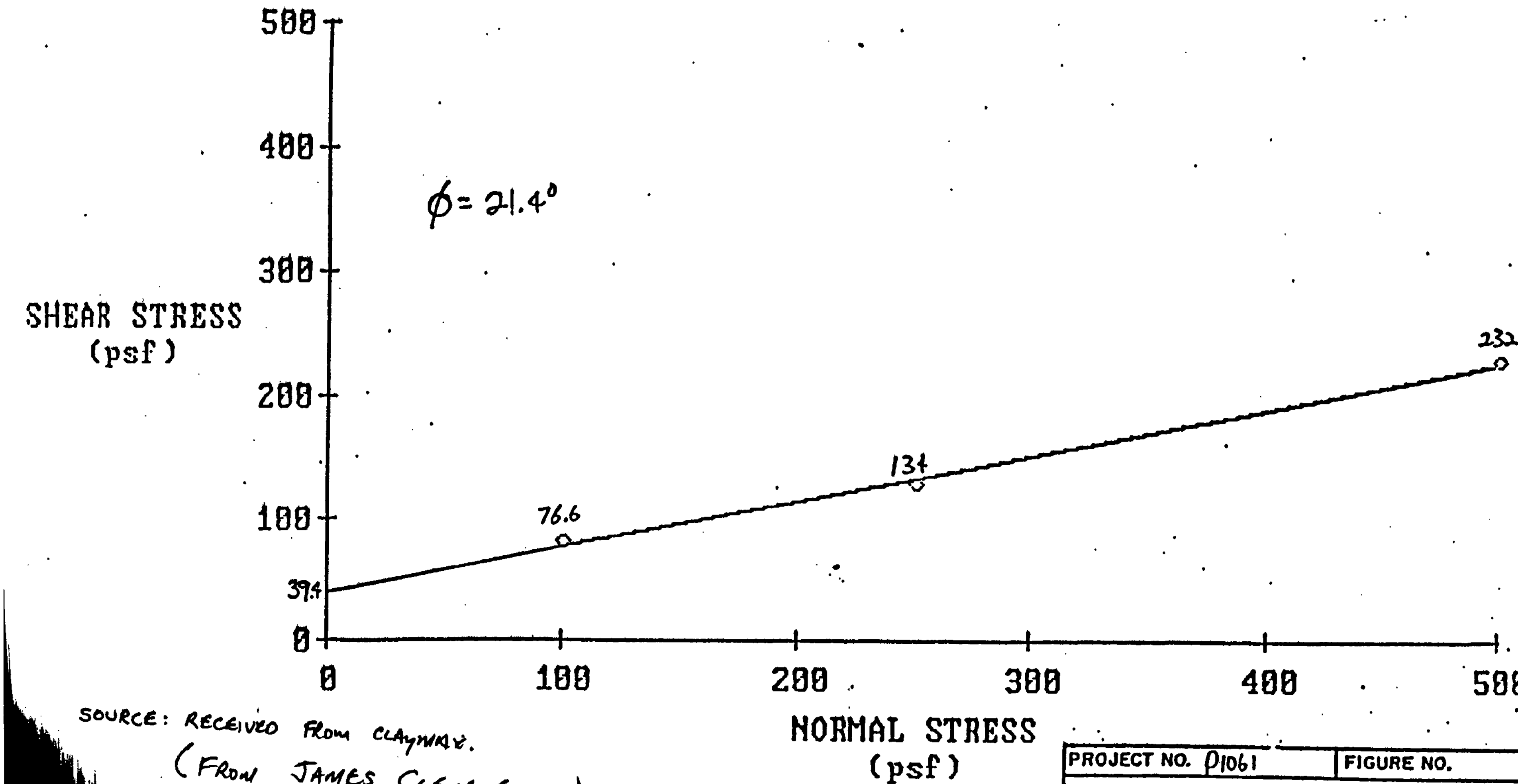
CLAYMAX DIRECT SHEAR TEST
CONFIG: SOAKED CLAYMAX (woven side)/SMOOTH HDPE



SOURCE: RECEIVED FROM CLAYMAX (JAMES CLEM CORP.)

PROJECT NO. P1061	FIGURE NO.
GEOSERVICES INC. CONSULTING ENGI[illegible]RS	

CLAYMAX DIRECT SHEAR TEST
CONFIG: SOAKED CLAYMAX (woven side)/GUNDLE ROUGH HDPE



SOURCE: RECEIVED FROM CLAYMAX.
(FROM JAMES CLEM CORP.)

PROJECT NO. P1061	FIGURE NO.
GEOSERVICES INC. CONSULTING ENGINEERS	

TABLE

INTERFACE DIRECT SHEAR TEST RESULTS
LOW NORMAL STRESS SHEAR STRENGTH PARAMETERS
(for Normal Stresses of 50 to 400 psf)
JAMES CLEM CORPORATION
SELECT CLAYMAX SHEAR-PRO BENTONITE COMPOSITE INTERFACES

Test Series Number	Interface Tested[1,2]	Peak Strength		Residual Strength	
		Friction Angle	Adhesion[3] (psf)	Friction Angle	Adhesion[3] (psf)
1	Dry Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Textured HDPE Geomembrane	35°	10	29°	10
2	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Textured HDPE Geomembrane	35°	10	26°	20
3	Hydrated Claymax Shear-Pro Prototype No. 4 Bentonite Composite/60-mil NSC Textured HDPE Geomembrane	23°	0	22°	0
4	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Textured VLDPE Geomembrane	37°	20	28°	10
5	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Smooth HDPE Geomembrane	11°	10	11°	10
6	Soaked AASHTO No. 57 Stone/Claymax Shear-Pro Prototype No. 4 Bentonite Composite	39°	10	38°	0
7	Soaked Sandy Clay Soil/Claymax Shear-Pro 500SP Bentonite Composite	25°	50	25°	50

Notes: (1) For Test Series 1, the bentonite composite-geomembrane interface was tested under dry conditions. For Test Series 2, 4, 5, 6, and 7, the bentonite composite-geomembrane or soil-bentonite composite interface was soaked in water for 24 hours under a normal stress of 100 psf prior to shearing. For Test Series 3, the bentonite composite was hydrated in water for 24 hours under a normal stress of 100 psf and then placed in the shear box.

(2) Test specimens were sheared immediately after application of the normal stress.

(3) The reported value of adhesion may not be the "true adhesion" of the interface and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

SOURCE: RECEIVED FROM JAMES CLEM CORP.

GL3160/GFL92211

APPENDIX A.4

Subdrain Collection

SUBDRAIN COLLECTION SYSTEM

Objective: Check subdrain collection system design calculations. (3.0 Collection System Subsurface)

Background:

The final grades have been modified, such that only two piles are to be covered. These calculations are prepared to address that the subdrain line located beneath the swales is adequate for the design flows.

References:

1. Engineering Report Preliminary and Intermediate Appendix A Calculations.
2. Cedergren H.R. (1989) Seepage, Drainage and Flow Nets Third Edition
3. Bowles J.E (1979) Physical and Geotechnical Properties of Soils

Assumptions:

1. The average annual drainage (percolation) from the HELP model to be collected by the GEONET is 11.53 inches/unit area/year. With the addition of 3X Standard Deviation of (3.12 inch/unit area/yr) the design flowrate is 20.89 inches/unit area/yr.

   HELP Model analysis was based on the flattest slopes and was included in the original computations

2. Watersheds are attached.

CLIENT		DEPARTMENT	
PROJECT	OSM25100	ORIGINATED BY	T. Szymoniak DATE 9/1/95
ENG. COMM. NO.	PROJECT NO.	CHECKED BY	DATE 9/
AREA		REVISION NUMBER	DATE
SUBJECT	SUBDRAIN COLLECTION SYSTEM	REVISED BY	

Form E53-2 (2-90)

1. Determine Peak flow Rates

Use the average annual percolation rate to compute the peak flow rate.

$$P = \frac{(20.89^{in}/\text{unit area}/yr)}{12\ in/ft} * \underset{43560 ft^2/acre}{\text{AREA}}(sq\,ft) \times \frac{yr}{365\,day} \times \frac{day}{24\,hr} \times \frac{hr}{3600\,sec}$$

$$P = 0.00240\ \text{AREA cfs}\ \left(\text{or} \times \frac{7.48\,gal}{ft^3} \times \frac{60\,sec}{min}\ \text{for GPM}\right) \quad 1.07917\ gpm$$

AREA = Watershed area in acres

TABLE 1.0
PEAK FLOW RATES

Watershed #	Area acres	Peak Flowrate. Cfs	GPM
A1	3.0	0.007	3.24
B1	1.9	0.0046	2.05
B2	5.8	0.0139	6.26
C1	7.4	0.0178	8.0
D1	5.2	0.01248	5.6
E1	0.7	0.00168	0.8

2. Check capacity of 4" ϕ ADS ~~Corrugated~~ Polyethelene Pipe (HDPE) using manning equation: ASTM F714, SDR 17
4" OD

$$Q = VA = \frac{1.49}{n}(A)(r_H)^{2/3}(\sqrt{S})$$

ID ⇒ 3.97
− .265
− .265
3.44

$n = 0.009$ (Plexco EHMWPE, Chevron)

$A = \left(\frac{4}{12}\right)^2 \frac{\pi}{4} = 0.087\ ft^2$ .065 $\quad S = 0.5\%$

$r_H = D/4 = 0.333/4 = 0.0833$ (Assume flowing full or half full)

4.9..
.327
.1..

CLIENT	DEPARTMENT
PROJECT 0	ORIGINATED BY T. Szymoniak DATE 9/1/95
ENG. COMM. NO. PROJECT NO. OSM	CHECKED BY DATE
AREA	REVISION NUMBER DATE
SUBJECT SUBDRAIN COLLECTION SYSTEM	REVISED BY

Form E53-2 (2-90)

.065 .0716

$$Q = \frac{1.49}{0.009}(0.087)(0.0833)^{2/3}(0.005)^{1/2}$$

.13

$Q = 0.19$ cfs $>$ PEAK FLOW RATES OK

Check velocity OK

.13

$$V = Q/A = \frac{0.19\ ft^3/sec}{0.087\ ft^2}$$

.065

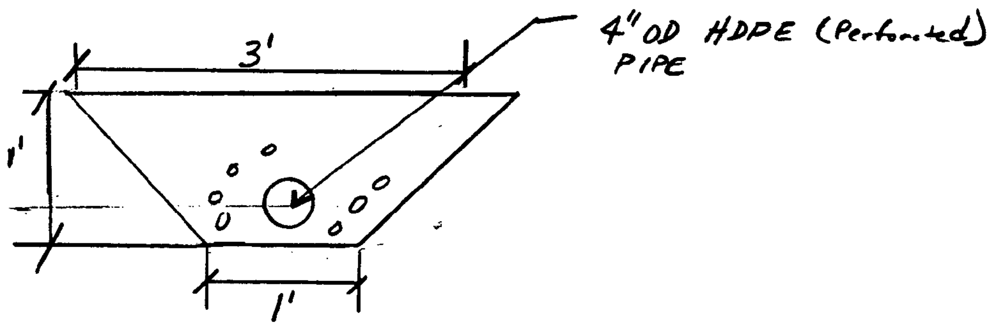
2.0

$V = 2.23$ ft/sec $\geq$ 2.0 ft/sec OK

3. Determine the aggregate size and trench configuration for the drain pipe. It is desirable to have a drain rock surrounding the pipe that has adequate flow comparable to the pipe. This would be a backup for the pipe. Therefore, use the highest design peak flow rate shown on table 1.0

$Q = 0.0178$ cfs.

Using Darcy's law compute required permability given the cross-section area shown below



$A = 2$ sq ft.

<u>APPENDIX A.5</u>

Stormwater Management

Determine required aggregate size.

$Q = kiA$ — where $i = 0.5\%$

$A = 2\ ft^2$

$Q = 0.0178\ ft^3/sec.$

Solve for k

$$k = \frac{Q}{iA} = \frac{0.0178\ ft^3/sec}{(0.005)(2)\ ft^2}$$

$$k_{Min} = 1.78\ ft/sec \approx \frac{30.48\ cm}{ft}$$

$$k_{min} \approx 1000\ cm/sec$$

Use the Hazen-Williams equation to approximate the D_{10} size whereby

$$k = C\,D_{10}^2,\ (\text{Bowles, } 1979) \qquad \text{let } C = 100$$

therefore

$$D_{10} = \left(k/C\right)^{1/2} = \left(\frac{1000\ cm/sec}{100}\right)^{1/2}$$

$$D_{10} = 3.2\ cm \sim 32\ mm$$

For optimal performance, the aggregate $D_{85}/W_s > 2$ (Cedegren) where W_s = typical slot width of perforated pipe. Solve for D_{85}

$D_{85} = 2 \times W_s$ ~ use a slot width of 0.5mm

$D_{85} = 1.0\ mm$

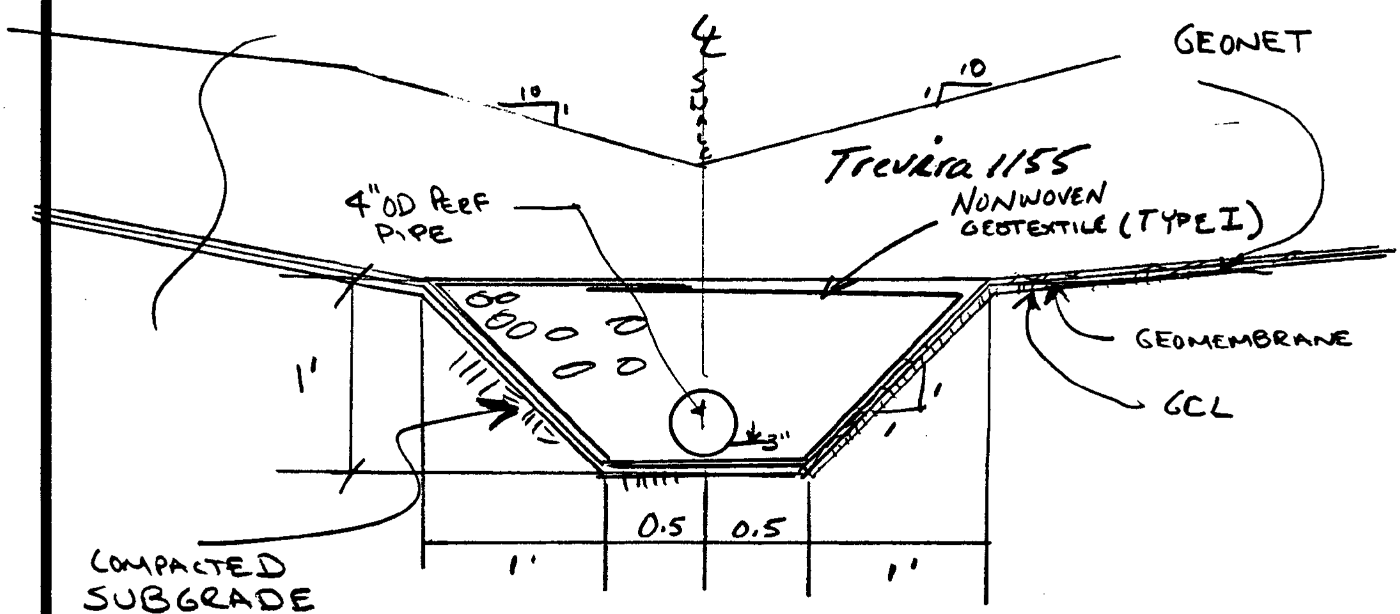
CLIENT		DEPARTMENT	
PROJECT	102 ND Landfill Closure	ORIGINATED BY	T. Szymoniak — DATE 9/1/95
ENG. COMM. NO.	PROJECT NO. 05M251	CHECKED BY	JTB — DATE
AREA		REVISION NUMBER	DATE
SUBJECT	SUBDRAIN COLLECTION SYSTEM	REVISED BY	

Form E53-2 (2-90)

4. Select gradation of aggregate based upon these design consideration. Utilize a # 357 aggregate to surround pipe. (2" to 4 in)

CONCLUSION:

4" OD Perforated pipe (0.5mm slots) is adequate to accomodate the anticipated flow rates. Pipe should be surrounded with a # 357 (ASTM D448-91) size aggregate and then wrapped with a geotextile.



1. END GEONET AT BOTTOM OF TRENCH, protect liner and pipe with the TYPE I Nonwoven geotextile.
2. Provide a minimum of 3" of bedding material for the pipe, for protection of the liner.
3. Refer to site grading plans 10210 U01-05 for swale locations.

CLIENT		DEPARTMENT	
PROJECT		ORIGINATED BY T. Szymoniak	DATE 9/1/95
ENG. COMM. NO.	PROJECT NO.	CHECKED BY	DATE
AREA		REVISION NUMBER	DATE
SUBJECT SUB DRAIN		REVISED BY	

Form E53-2 (2-90)

Objective: Compute the Q_{10}, V_{10} and d_{10} (depth) for the stormwater pipeline located adjacent to Buffalo Rd and the Q_{10}, V_{10} and d_{10} in the swales located along the east, central and west portion of the closure.

References

1. Preliminary Intermediate Engineering Report, Appendix A - Calculations.

2. Lindeburg, M. R. (1988) Civil Engineering Reference Manual Fourth Edition

3. Frederick, R. H et. al (1977) Five to 60-minute Precipitation Frequency for Eastern and Central U.S. NOAA Technical Memorandum NWS Hydro-35

4. Walesh, S. (1989) Urban Surface Water Management

SCOPE:

PIPELINE

1. Determine drainage areas and composite runoff coefficient for each of the areas.

2. Areas are taken from the grading sheets 10U01-10U05 and the proposed location of the stormwater pipeline (30K-01). Utilize rational formula ($Q = CIA$) to compute the discharge at a point. Develop a site specific rainfall intensity curve using the frequency atlas.

3. Utilize the manning equation for the pipeline to compute the depth, velocity and flow. Assume a roughness coefficient of 0.018 (Concrete, roughened). Values are summarized on attached spreadsheet, pg 8.

Stormwater Management 9/1/95 2/19

Compute composite 'C' for the paved and grassed areas.

Appendix A: Rational Method Runoff Coefficients

C

categorized by surface	
forested	0.059–0.2
asphalt	0.7–0.95 ~ use 0.85
brick	0.7–0.85
concrete	0.8–0.95
shingle roof	0.75–0.95
lawns, well drained (sandy soil)	
up to 2% slope	0.05–0.1
2% to 7% slope	0.10–0.15
over 7% slope	0.15–0.2
lawns, poor drainage (clay soil)	
up to 2% slope	0.13–0.17 — use 0.35 Conservatism
2% to 7% slope	0.18–0.22
over 7% slope	0.25–0.35
driveways, walkways	0.75–0.85
categorized by use	
farmland	0.05–0.3
pasture	0.05–0.3
unimproved	0.1–0.3
parks	0.1–0.25
cemeteries	0.1–0.25
railroad yard	0.2–0.40
playgrounds (except asphalt or concrete)	0.2–0.35
business districts	
neighborhood	0.5–0.7
city (downtown)	0.7–0.95
residential	
single family	0.3–0.5
multi-plexes, detached	0.4–0.6
multi-plexes, attached	0.6–0.75
suburban	0.25–0.4
apartments, condominiums	0.5–0.7
industrial	
light	0.5–0.8
heavy	0.6–0.9

After Civil Engineering Review, Lindeburg

Construct Rainfall Intensity Curve for 10-yr and

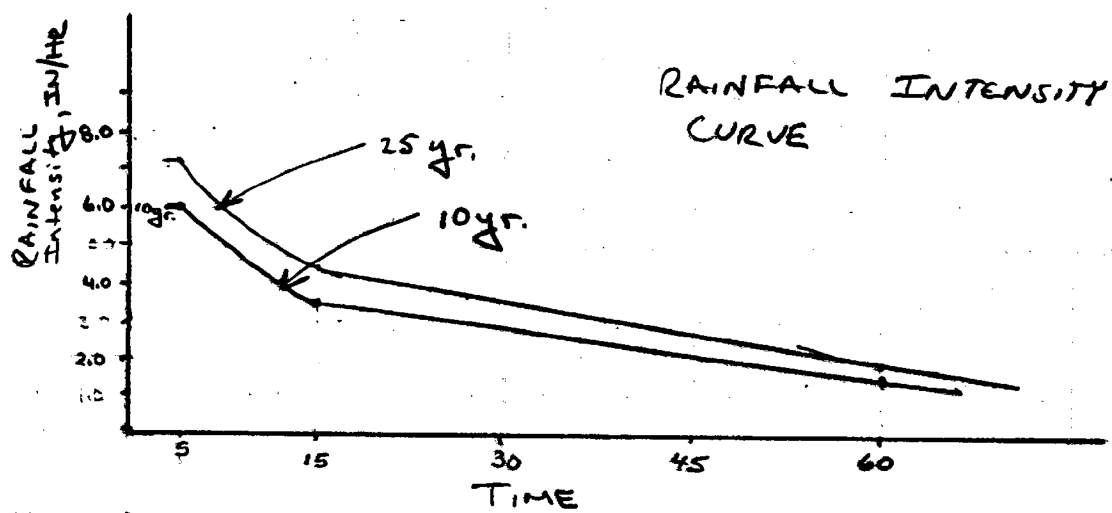
Return	Time, min	Precipitation P, in	I in/hr
100-yr.	5 15 60 120	0.70 1.3 2.25	8.4 5.2 2.25
25-yr.	5 15 60 120	0.60 1.1 1.8	7.2 4.4 1.8
10 yr.	5 min 15 60	0.50 0.91 1.50	6.0 3.6 1.5
2 yr.	5 15 60	0.35 0.66 1.0	4.2 2.6 1.0

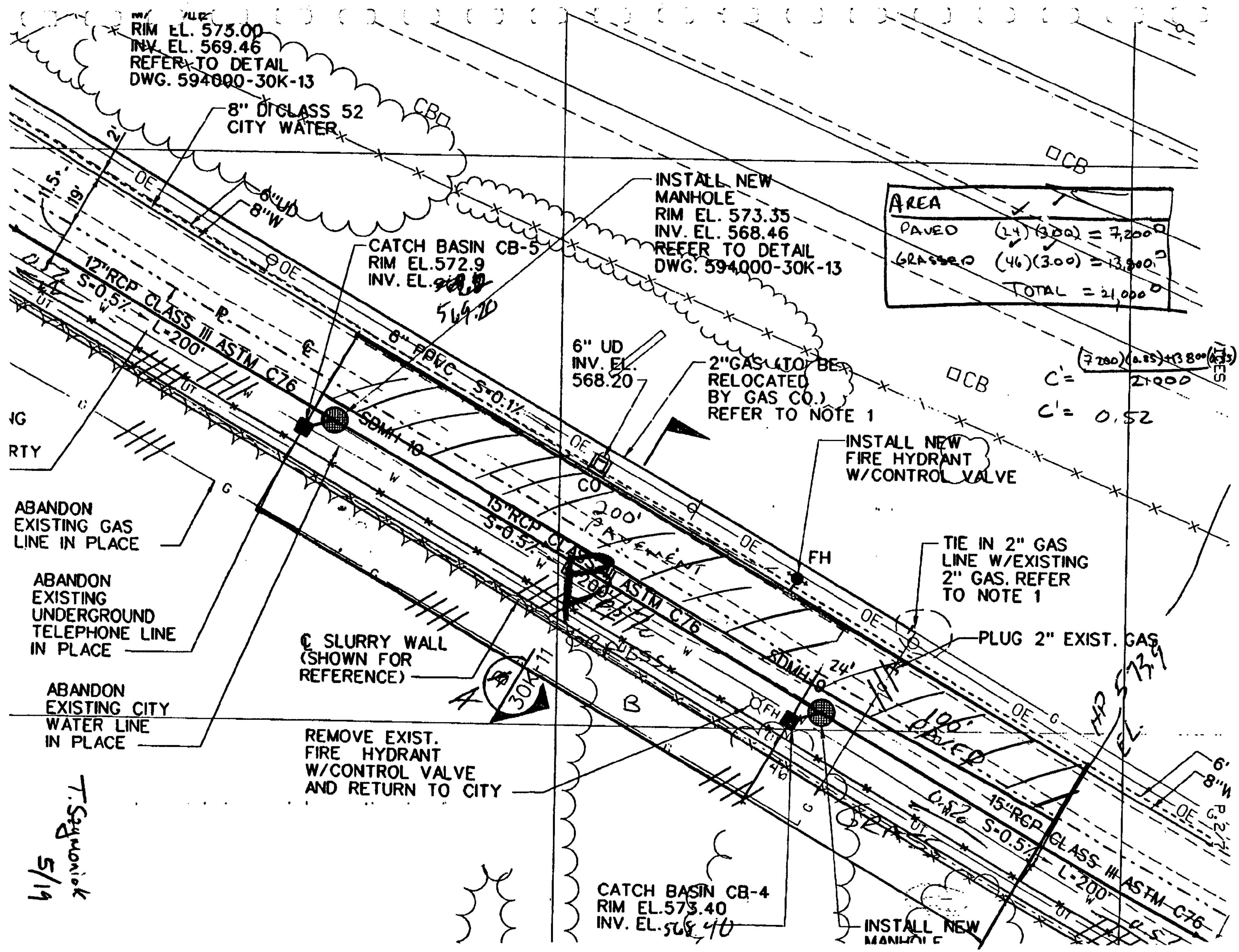
(SOURCE: FIVE TO 60 MINUTE PRECIPITATION FREQUENCY FOR EASTERN AND CENTRAL UNITED STATES NOAA Technical Memorandum NWS HYDRO-35)

10 yr = 0.449 (100-yr) + 0.496 (2-yr)

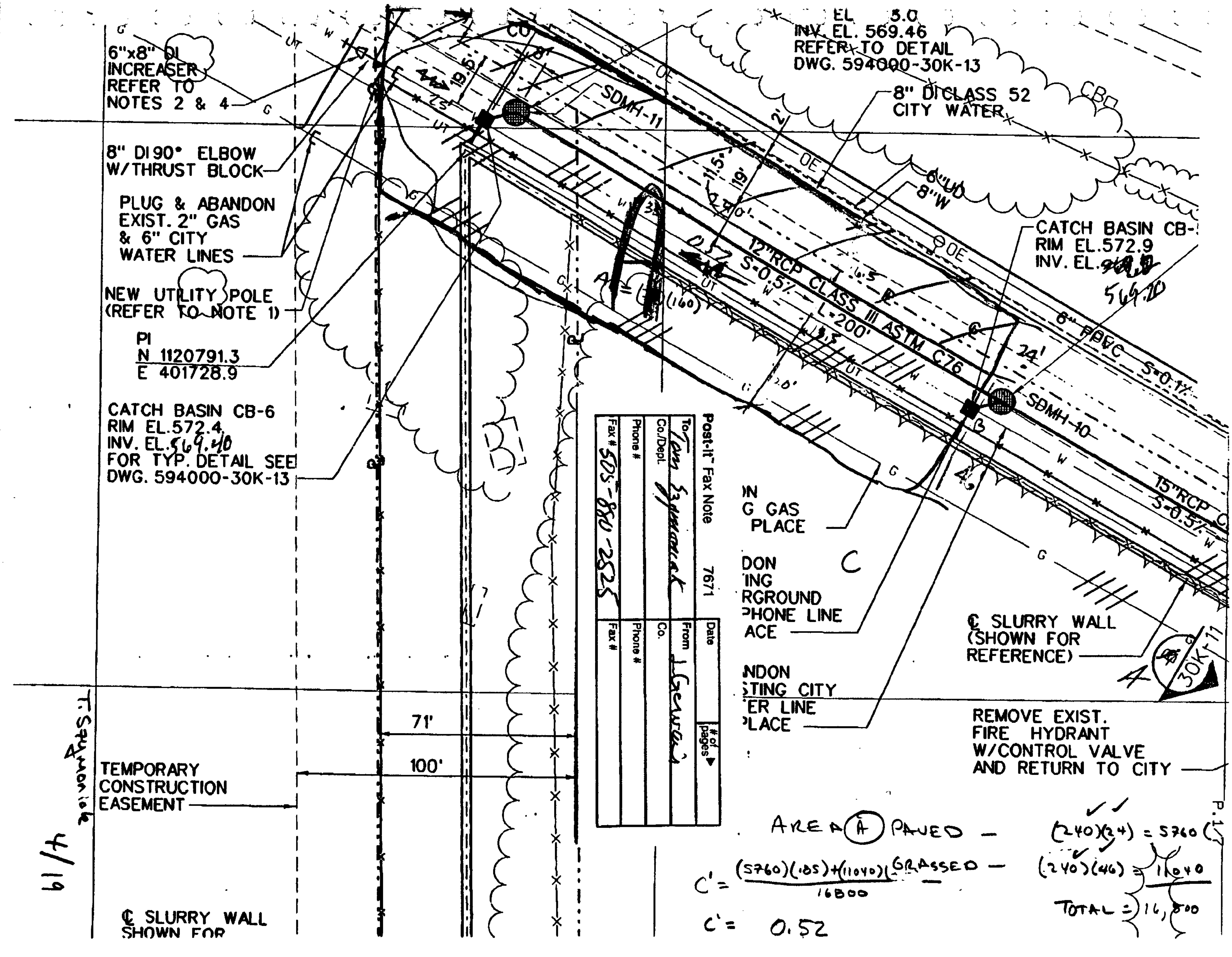
25 yr = 0.669 (100 yr) + 0.293 (2 yr)

PLOT INTENSITY DURATION CURVES

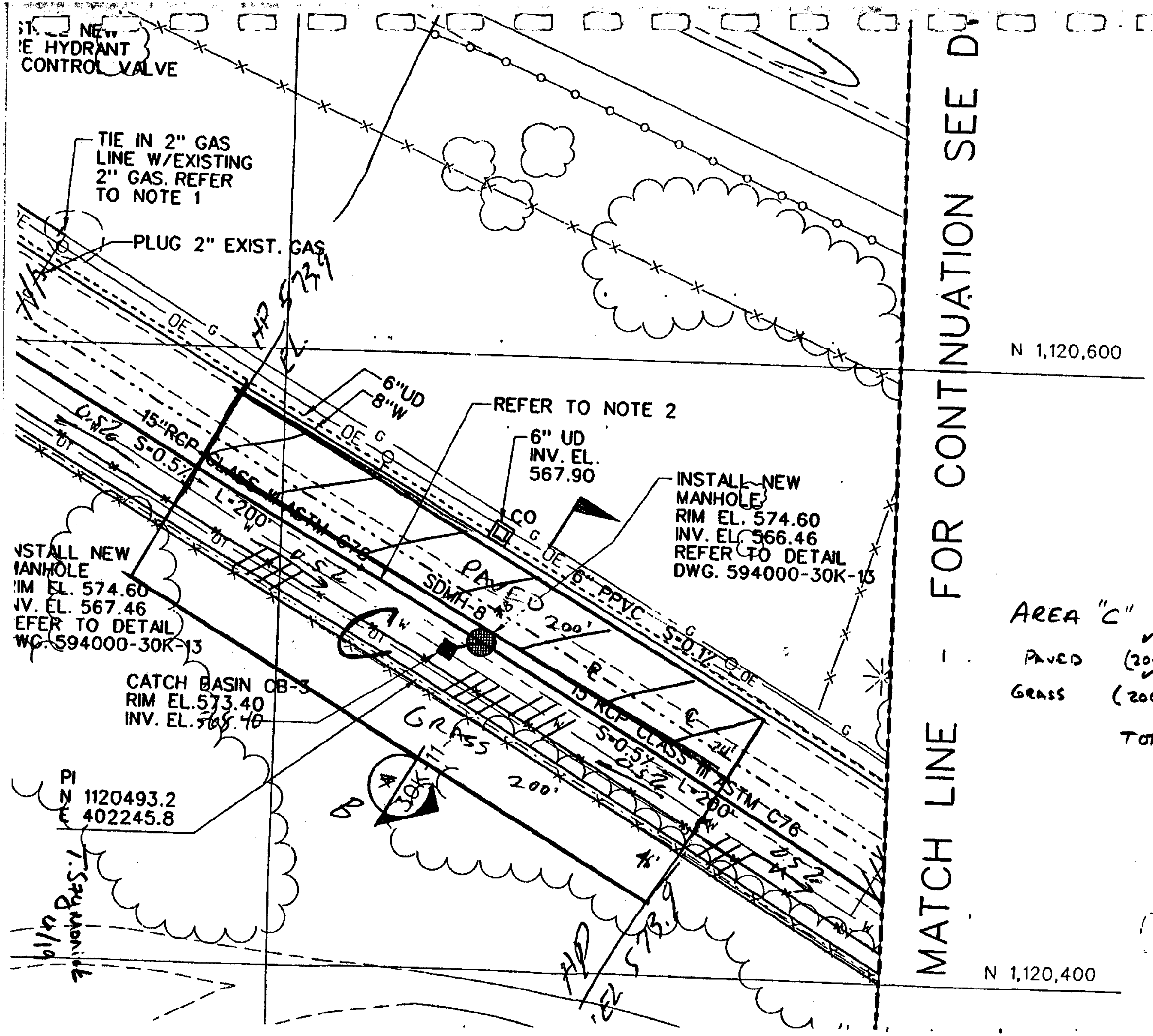




RIM EL. 575.00
INV. EL. 569.46
REFER TO DETAIL
DWG. 594000-30K-13
8" DI CLASS 52
CITY WATER
6" UD
8" W
OE
CB
CATCH BASIN CB-5
RIM EL. 572.9
INV. EL. 569.20
12" RCP CLASS III ASTM C76
S-0.5%
L-200'
6" PVC S-0.1%
SDMH 10
INSTALL NEW
MANHOLE
RIM EL. 573.35
INV. EL. 568.46
REFER TO DETAIL
DWG. 594000-30K-13
AREA
PAVED (24)(300) = 7,200
GRASSED (46)(300) = 13,800
TOTAL = 21,000
6" UD
INV. EL.
568.20
2" GAS (TO BE
RELOCATED
BY GAS CO.)
REFER TO NOTE 1
C' = (7200)(0.85)+(13800)(0.35) / 21000
C' = 0.52
CO
200'
PAVEMENT
INSTALL NEW
FIRE HYDRANT
W/CONTROL VALVE
FH
15" RCP CLASS III ASTM C76
S-0.5%
TIE IN 2" GAS
LINE W/EXISTING
2" GAS. REFER
TO NOTE 1
PLUG 2" EXIST. GAS
ABANDON
EXISTING GAS
LINE IN PLACE
ABANDON
EXISTING
UNDERGROUND
TELEPHONE LINE
IN PLACE
ABANDON
EXISTING CITY
WATER LINE
IN PLACE
℄ SLURRY WALL
(SHOWN FOR
REFERENCE)
30K-11
SDMH 9
24'
46'
100'
PAVED
REMOVE EXIST.
FIRE HYDRANT
W/CONTROL VALVE
AND RETURN TO CITY
15" RCP CLASS III ASTM C76
S-0.5%
L-200'
CATCH BASIN CB-4
RIM EL. 573.40
INV. EL. 568.40
INSTALL NEW
MANHOLE
6'
8" W
T. Szymoniok
5/19



P.1
EL 3.0
INV. EL. 569.46
REFER TO DETAIL
DWG. 594000-30K-13
8" DI CLASS 52
CITY WATER
6"x8" DI
INCREASER
REFER TO
NOTES 2 & 4
8" DI 90° ELBOW
W/THRUST BLOCK
PLUG & ABANDON
EXIST. 2" GAS
& 6" CITY
WATER LINES
NEW UTILITY POLE
(REFER TO NOTE 1)
PI
N 1120791.3
E 401728.9
CATCH BASIN CB-6
RIM EL.572.4
INV. EL. 569.40
FOR TYP. DETAIL SEE
DWG. 594000-30K-13
SDMH-11
SDMH-10
12"RCP CLASS III ASTM C76
S-0.5%
L-200'
6" PVC S-0.1%
15"RCP
S-0.5%
8"W
CATCH BASIN CB-
RIM EL.572.9
INV. EL.
569.20
24'
71'
100'
TEMPORARY
CONSTRUCTION
EASEMENT
℄ SLURRY WALL
SHOWN FOR
℄ SLURRY WALL
(SHOWN FOR
REFERENCE)
REMOVE EXIST.
FIRE HYDRANT
W/CONTROL VALVE
AND RETURN TO CITY
30K-11
Post-it™ Fax Note 7671
Date
# of pages
To Tom Szymoniak
From L Gervais
Co./Dept.
Co.
Phone #
Phone #
Fax # 505-880-2525
Fax #
AREA (A) PAVED - (240)(24) = 5760
GRASSED - (240)(46) = 11,040
TOTAL = 16,800
C' = (5760)(.85)+(11040)(
16800
C' = 0.52
T. Szymoniak
4/19

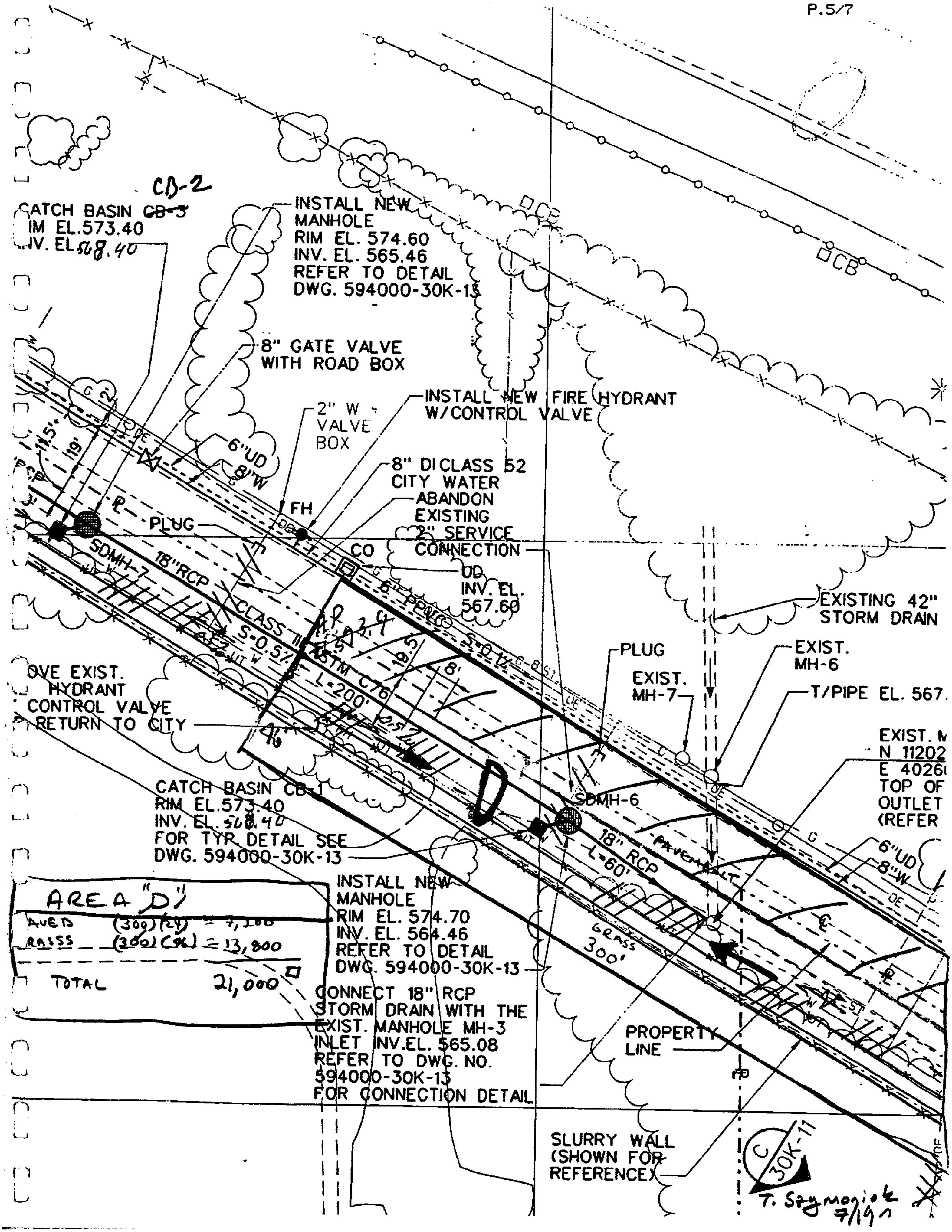


GENERA...

1. REFER TO DRAINAGE...
2. REFER TO ABBREVIAT...

AREA "C"

PAVED	(200)(24) =	4800
GRASS	(200)(46) =	9200
TOTAL		14000 $\square$

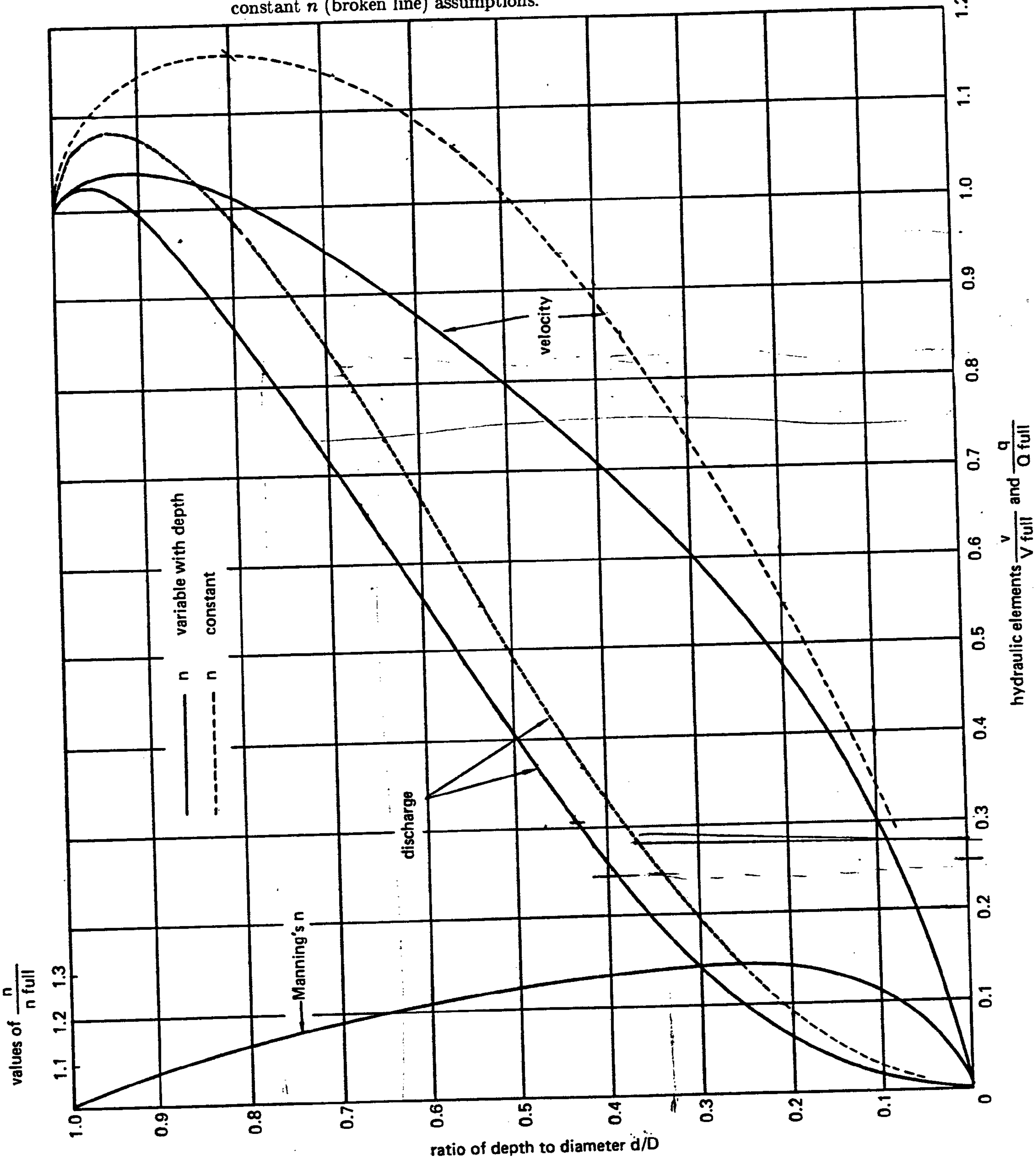

CD-2
CATCH BASIN ~~CB-3~~
IM EL.573.40
NV. EL 568.40
INSTALL NEW
MANHOLE
RIM EL. 574.60
INV. EL. 565.46
REFER TO DETAIL
DWG. 594000-30K-13
8" GATE VALVE
WITH ROAD BOX
INSTALL NEW FIRE HYDRANT
W/CONTROL VALVE
2" W
VALVE
BOX
6"UD
8"W
8" DI CLASS 52
CITY WATER
ABANDON
EXISTING
2" SERVICE
CONNECTION
FH
PLUG
CO
UD
INV. EL.
567.60
SDMH-7
18"RCP
CLASS III
ASTM C76
S-0.5%
L-200'
S-0.1%
EXISTING 42"
STORM DRAIN
PLUG
EXIST.
MH-7
EXIST.
MH-6
T/PIPE EL. 567.
OVE EXIST.
HYDRANT
CONTROL VALVE
RETURN TO CITY
EXIST. M
N 11202
E 4026
TOP OF
OUTLET
(REFER
CATCH BASIN CB-1
RIM EL.573.40
INV. EL. 568.40
FOR TYP. DETAIL SEE
DWG. 594000-30K-13
SDMH-6
18" RCP
L-60'
PAVEMENT
6"UD
8"W
INSTALL NEW
MANHOLE
RIM EL. 574.70
INV. EL. 564.46
REFER TO DETAIL
DWG. 594000-30K-13
GRASS
300'
CONNECT 18" RCP
STORM DRAIN WITH THE
EXIST. MANHOLE MH-3
INLET INV.EL. 565.08
REFER TO DWG. NO.
594000-30K-13
FOR CONNECTION DETAIL
PROPERTY
LINE
SLURRY WALL
(SHOWN FOR
REFERENCE)
C
30K-11
T. Szymoniak
7/19
AREA "D"
PAVED (300)(24) = 7,200
GRASS (300)(46) = 13,800
TOTAL 21,000

CRITICAL Circular Channel ratios

See page 5-4 for table summary.

Appendix C: Circular Channel Ratios

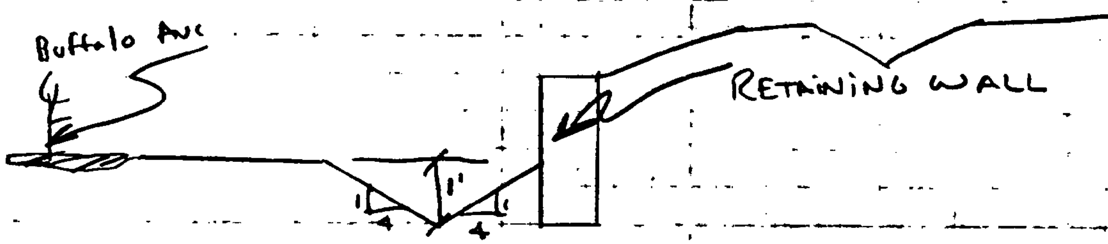
Experiments have shown that n varies slightly with depth. This figure gives velocity and flow rate ratios for varying n (solid line) and constant n (broken line) assumptions.



9/19

4. Compute velocity, depth and discharge for the swales located around the perimeter and on the east side of loadout access roadway (E1, E2, E3).

For the swale adjacent to retaining wall the following geometry has been assummed.



$z = 4$

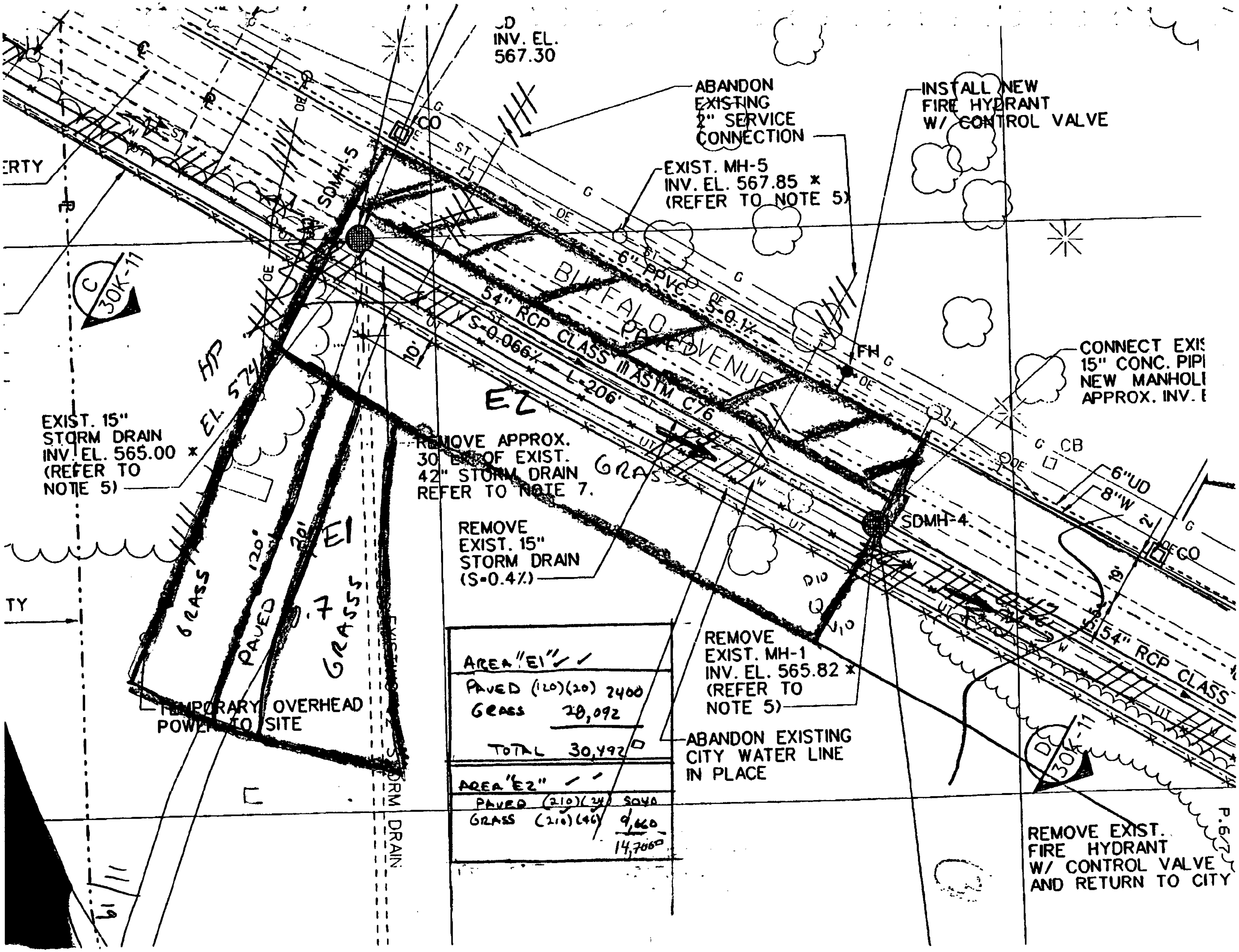
$n = 0.040$

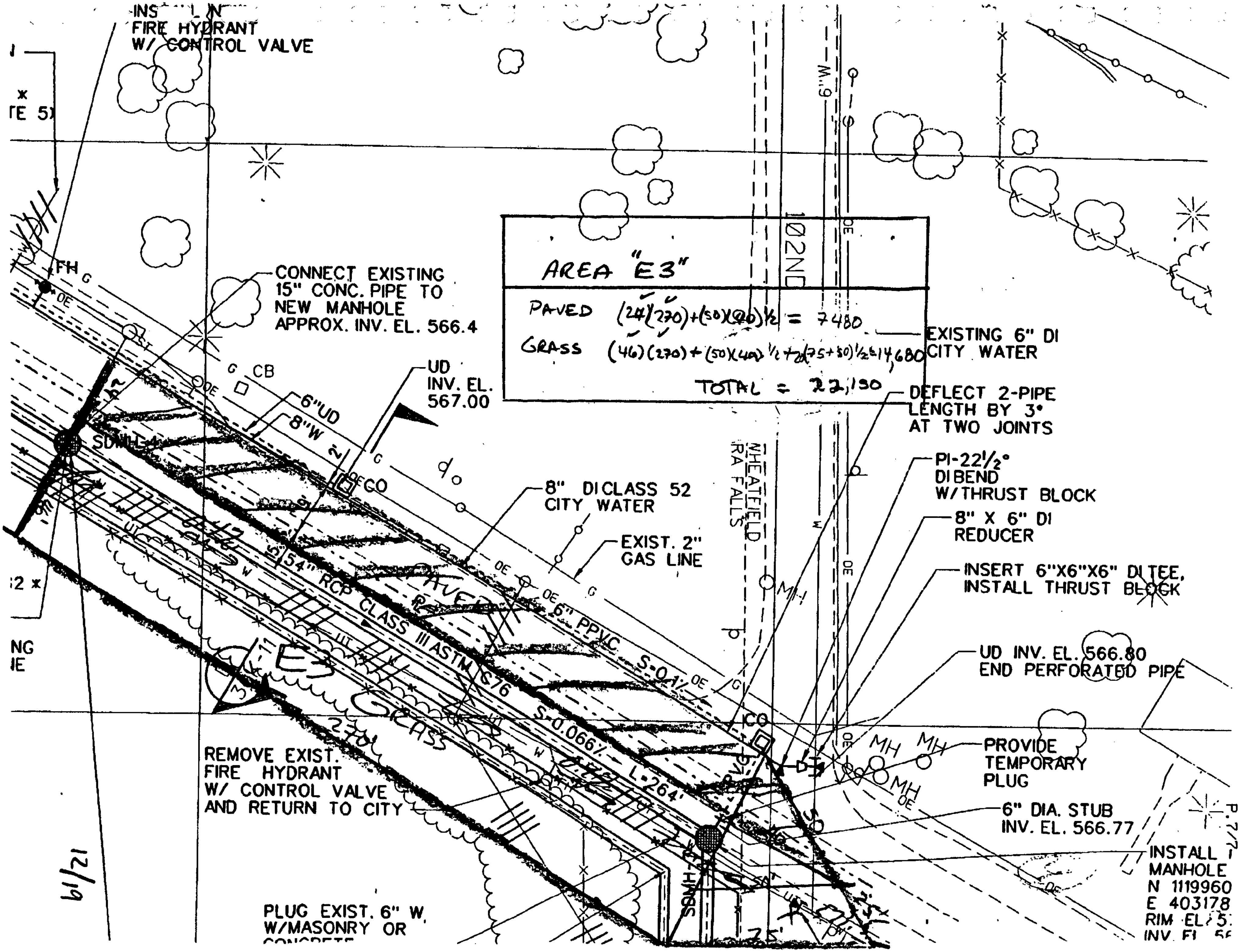
Areas are shown on pg. 11 + 12 for the "E" SWALE

Summary:

	@ Structure	Q_{10} cfs	V_{10} ft/sec	d_{10} ft.	
E1 + E2	SDMH-4	3.2	1.4	0.77	< 1' OK
E1 + E2 + E3	SDMH-3	4.8	1.5	0.92	< 1' OK

No erosion protection required since velocity is less than 2 ft/sec. Swale may overtop during 25-year storm since there is no freeboard.

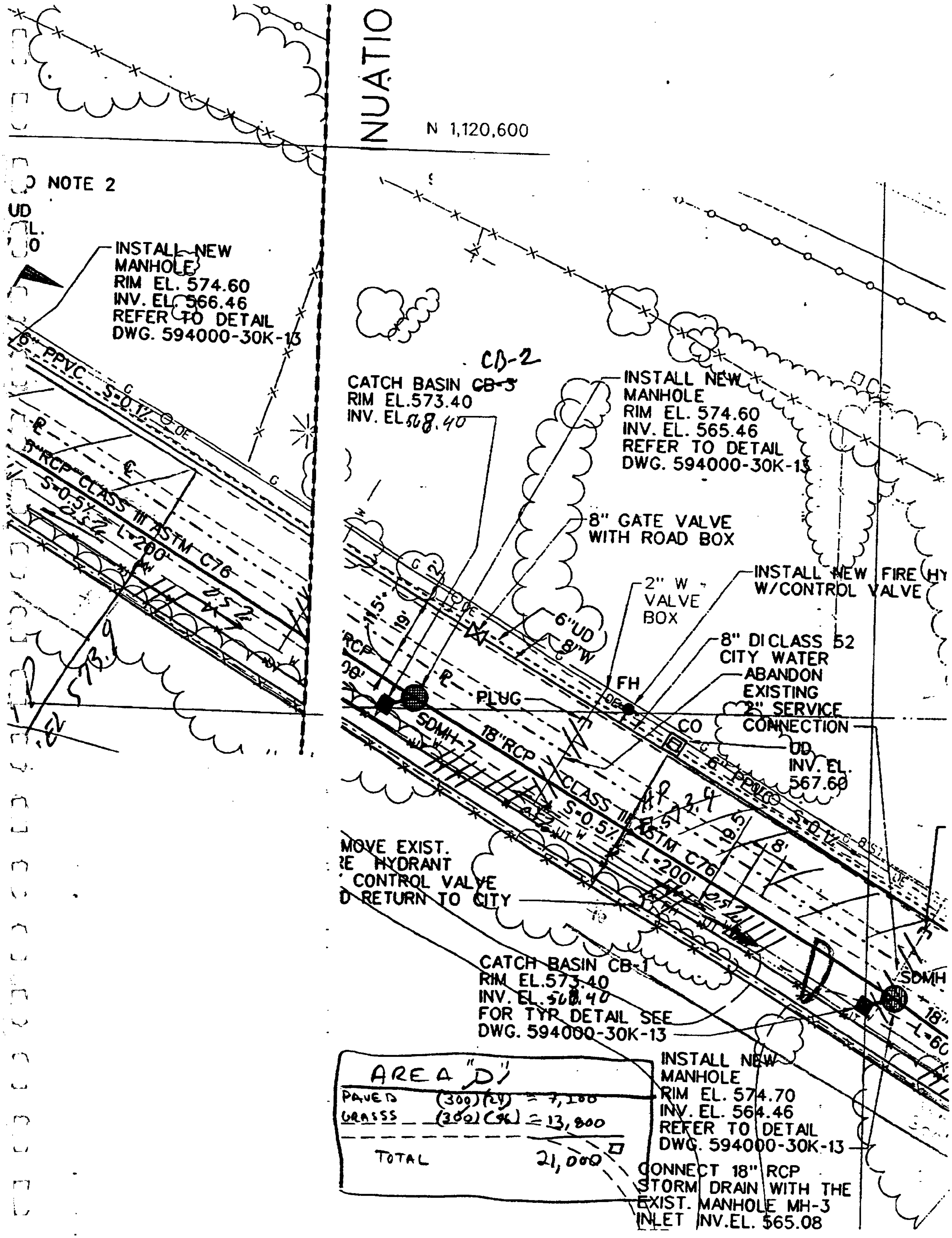

P.6/7
INV. EL.
567.30
ABANDON
EXISTING
2" SERVICE
CONNECTION
INSTALL NEW
FIRE HYDRANT
W/ CONTROL VALVE
EXIST. MH-5
INV. EL. 567.85 ×
(REFER TO NOTE 5)
SDMH-5
SDMH-4
BUFFALO AVENUE
PAVED
6" PPVC S-0.1%
54" RCP CLASS III ASTM C76
S-0.066% L-206'
FH
CO
CB
6"UD
8"W
CONNECT EXIS
15" CONC. PIPI
NEW MANHOLI
APPROX. INV. I
E2
GRASS
HP
EL 574
EXIST. 15"
STORM DRAIN
INV. EL. 565.00 ×
(REFER TO
NOTE 5)
REMOVE APPROX.
30 LF OF EXIST.
42" STORM DRAIN
REFER TO NOTE 7.
REMOVE
EXIST. 15"
STORM DRAIN
(S-0.4%)
REMOVE
EXIST. MH-1
INV. EL. 565.82 ×
(REFER TO
NOTE 5)
ABANDON EXISTING
CITY WATER LINE
IN PLACE
REMOVE EXIST.
FIRE HYDRANT
W/ CONTROL VALVE
AND RETURN TO CITY
54" RCP CLASS
30K-11
E1
.7
GRASS
PAVED
GRASS
120'
20'
TEMPORARY OVERHEAD
POWER TO SITE
EXISTING STORM DRAIN
AREA "E1"
PAVED (120)(20) 2400
GRASS 28,092
TOTAL 30,492
AREA "E2"
PAVED (210)(24) 5040
GRASS (210)(46) 9,660
14,700
11/19



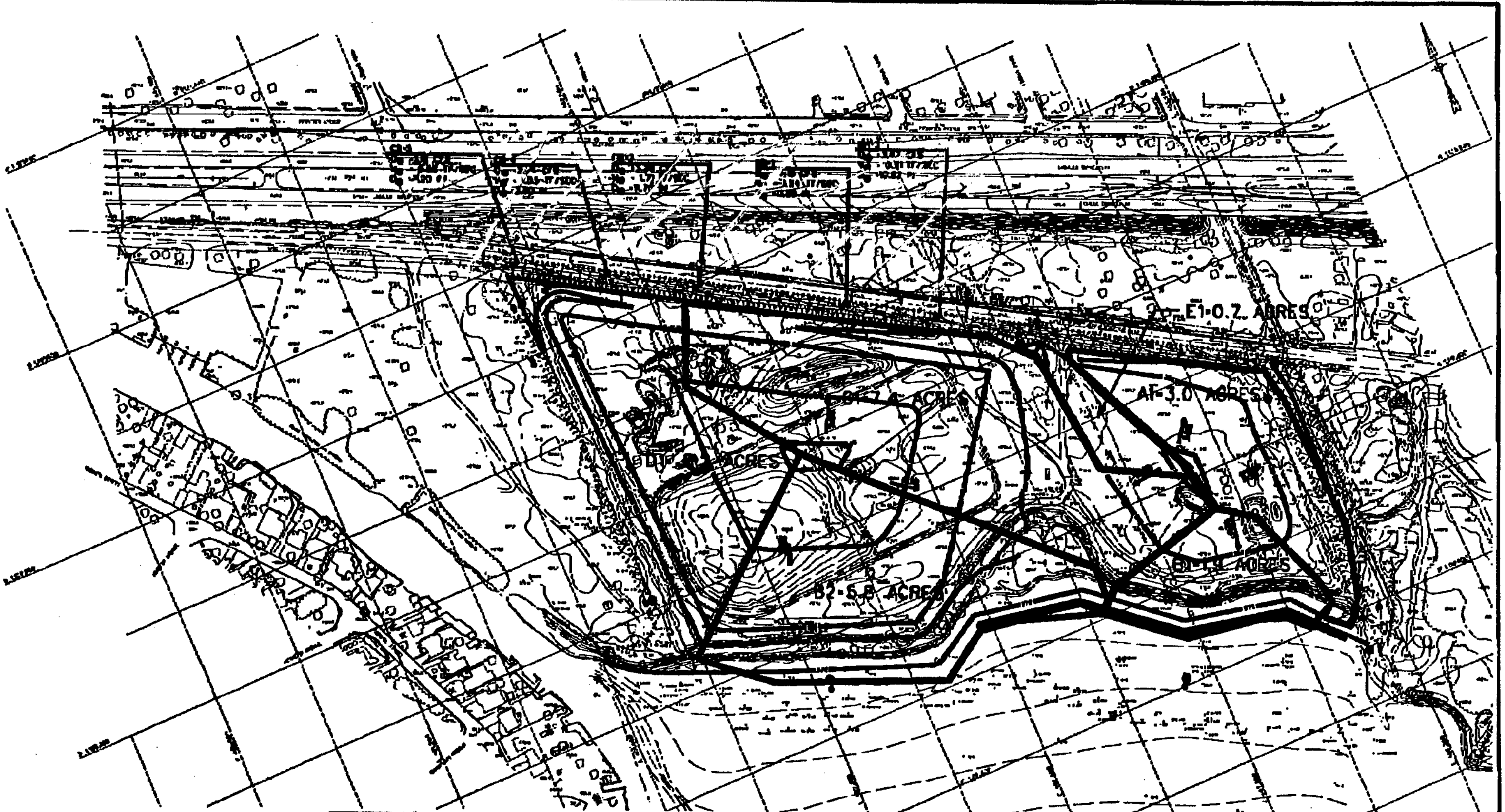
INSTALL FIRE HYDRANT W/ CONTROL VALVE
CONNECT EXISTING 15" CONC. PIPE TO NEW MANHOLE APPROX. INV. EL. 566.4
UD INV. EL. 567.00
AREA "E3"
PAVED (24)(270)+(50)(20)½ = 7480
GRASS (46)(270)+(50)(40)½+2(75+50)½ = 14,680
TOTAL = 22,150
EXISTING 6" DI CITY WATER
DEFLECT 2-PIPE LENGTH BY 3° AT TWO JOINTS
PI-22½° DI BEND W/THRUST BLOCK
8" X 6" DI REDUCER
INSERT 6"X6"X6" DI TEE, INSTALL THRUST BLOCK
UD INV. EL. 566.80 END PERFORATED PIPE
PROVIDE TEMPORARY PLUG
6" DIA. STUB INV. EL. 566.77
INSTALL MANHOLE N 1119960 E 403178
8" DI CLASS 52 CITY WATER
EXIST. 2" GAS LINE
6" PPVC S-0.1%
54" RCP CLASS III ASTM C76 S-0.066% L-264'
102ND
WHEATFIELD
RA FALLS
6"UD
8"W
CB
CO
SDMH-4
SDMH-5
MH
FH
PAVED
GRASS
270'
75'
REMOVE EXIST. FIRE HYDRANT W/ CONTROL VALVE AND RETURN TO CITY
PLUG EXIST. 6" W. W/MASONRY OR
P.7/7
12/19

RATIONAL

AREA	STRUCTURE	AREA	AREA CUM.	"C"	t MIN	LEN (Overland Flow)	I IN/HR	Q CFS	SLOPE %	DIAMETER INCH	AREA SQ FT	VFULL FT/SEC	QFULL CFS	Q/QFULL	V/VFULL	d/D	v FT/SEC	d IN
A	CB-6 TO CB-5	0.38	0.38	0.52	5		6	1.19	0.5	12	0.79	2.78	2.18	0.54	0.2	0.6	0.56	7.20
B	CB-5 TO CB-4	0.48	0.86	0.52	7.8	200	5.5	2.47	0.5	15	1.23	3.23	3.96	0.62	0.32	0.64	1.03	9.60
C	CB-4 TO CB-3	0.32	1.18	0.52	9.5	100	5.3	3.26	0.5	15	1.23	3.23	3.96	0.82	0.53	0.78	1.71	11.70
C1	CB-3 TO CB-2	0.32	1.50	0.52	11.4	200	5	3.91	0.5	15	1.23	3.23	3.96	0.99	0.53	0.78	1.71	11.70
D	CB-2 TO CB-1	0.482	1.67	0.52	13.3	200	4.8	4.18	0.5	18	1.77	3.64	6.44	0.65	0.34	0.66	1.24	11.88
D1	CB-1 TO MH-3	0	1.67	0.52	13.9	60	4	3.47	0.5	18	1.77	3.64	6.44	0.54	0.25	0.59	0.91	10.62



NUATIO
N 1,120,600
NOTE 2
UD
INSTALL NEW
MANHOLE
RIM EL. 574.60
INV. EL. 566.46
REFER TO DETAIL
DWG. 594000-30K-13
6" PPVC S-0.1%
RCP CLASS III ASTM C76
S-0.5% L-200'
CB-2
CATCH BASIN CB-3
RIM EL.573.40
INV. EL.568.40
INSTALL NEW
MANHOLE
RIM EL. 574.60
INV. EL. 565.46
REFER TO DETAIL
DWG. 594000-30K-13
8" GATE VALVE
WITH ROAD BOX
2" W
VALVE
BOX
INSTALL NEW FIRE HY
W/CONTROL VALVE
6"UD
8"W
8" DI CLASS 52
CITY WATER
ABANDON
EXISTING
2" SERVICE
CONNECTION
FH
PLUG
CO
UD
INV. EL.
567.60
SDMH-7
18"RCP CLASS III ASTM C76
S-0.5% L-200'
6" PPVC S-0.1%
MOVE EXIST.
RE HYDRANT
CONTROL VALVE
D RETURN TO CITY
CATCH BASIN CB-1
RIM EL.573.40
INV. EL.568.40
FOR TYP DETAIL SEE
DWG. 594000-30K-13
SDMH
INSTALL NEW
MANHOLE
RIM EL. 574.70
INV. EL. 564.46
REFER TO DETAIL
DWG. 594000-30K-13
CONNECT 18" RCP
STORM DRAIN WITH THE
EXIST. MANHOLE MH-3
INLET INV.EL. 565.08
AREA "D"
PAVED (300)(24) = 7,100
GRASSS (300)(46) = 13,800
TOTAL 21,000



ID	AREA ACRES	LONGEST DISTANCE OVERLAND	LONGEST DISTANCE SWALE	SWALE	INTENSITY IN/HR	10 YEAR STORM EVENT PEAK DISCHARGE cfs	VELOCITY FT/SEC	SWALE SLOPE %	SWALES DEPTH, FT
A1	3.0	200'	875	EAST	4.8	10.6	1.5	0.5%	0.85
B1	1.9	150'	-	-	~~[illegible]~~	~~[illegible]~~	-	-	-
B2	5.8	150'	-	-	~~[illegible]~~	~~[illegible]~~	-	-	-
C1	7.4	150'	1270	CENTER	3.8	27.1	1.8	0.5%	1.1
D1	3.7	150'	1260	WEST	4.0	14.8	1.6	0.5%	0.85
E	1.8	300'	-	NORTHWEST	-	4.8	1.5	0.4%	0.93

FLUOR DANIEL

OXYCHEM/OLIN
REMEDIAL DESIGN
102nd STREET LANDFILL SITE
NIAGARA FALLS, NEW YORK

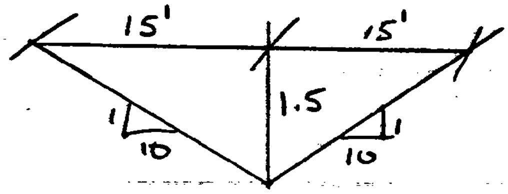
DRAINAGE SWALE PLAN

1"=60'

564000-100-03

...oi\olin\cadd\10210a03.dgn Sep. 8. 1995 09:30:49

5. Check flowrate, discharge, velocity and depth for the perimeter diversion swales. A constant ditch geometry has been determined and designed to accomodate the ten-year storm interval. The following geometry has been used for the EAST, Central and West swales.



$n = 0.040$

$S = 0.5\%$

Drainage areas are shown on the attached plan along with longest flowpath. Assume worst case for the ten-year storm frequency which yields at intensity (I) of 6.0 in/hr. Utilize the manning equation and iterate using depth.

SUMMARY

SWALE	AREA	Q_{cfs}	$V_{ft/sec}$	d_{10} ft	
EAST	3.0	13.5	1.6	0.93	< 1.5' OK
CENTRAL	7.4	33.3	1.97	1.3	< 1.5' OK
WEST	5.2	23.4	1.80	1.14	< 1.5' ok

Provide tapered riprap (4-8") at the exit of the swales unto the bulkhead. Because velocities are less than 2 ft/sec no additional permanent erosion protection is required.

STORM DRAINAGE

Revise computations to account for travel time and time of concentration at outlet point. Determine time of concentration using the overland flow and the flow velocity (assumed) in the ditch.

	Overland	time ①	Ditch length	time ② min	TOTAL min.	Intensity in/hr.
EAST	200'	2.1	976	8.2	10.3	4.8
CENTRAL	150'	1.6	1270	10.6	12.1	3.8
WEST	150'	1.6	1210	10.1	11.7	4.0

① * Use overland flow @ 5% of 1.6 ft/sec, see attached figure).

② Use a velocity of 2.0 ft/sec for the ditches

Compute velocity, discharge and depth for the swales usings mannings equations. Use a mannings roughness of 0.040 and a runoff coefficient of 0.75

SUMMARY 10YR - STORM Event

	AREA	I in/hr	Q cfs	V ft/sec	d ft	
EAST	3.0	4.8	10.8	1.5	0.85	< 1.5 OK
CENTRAL	7.4	3.8	21.1	1.8	1.1	< 1.5 OK
WEST	5.2	4.0	14.8	1.6	0.95	

Ditches have adequate capacity and erosion protection then computed by dividing the total overland flow length by the average velocity.

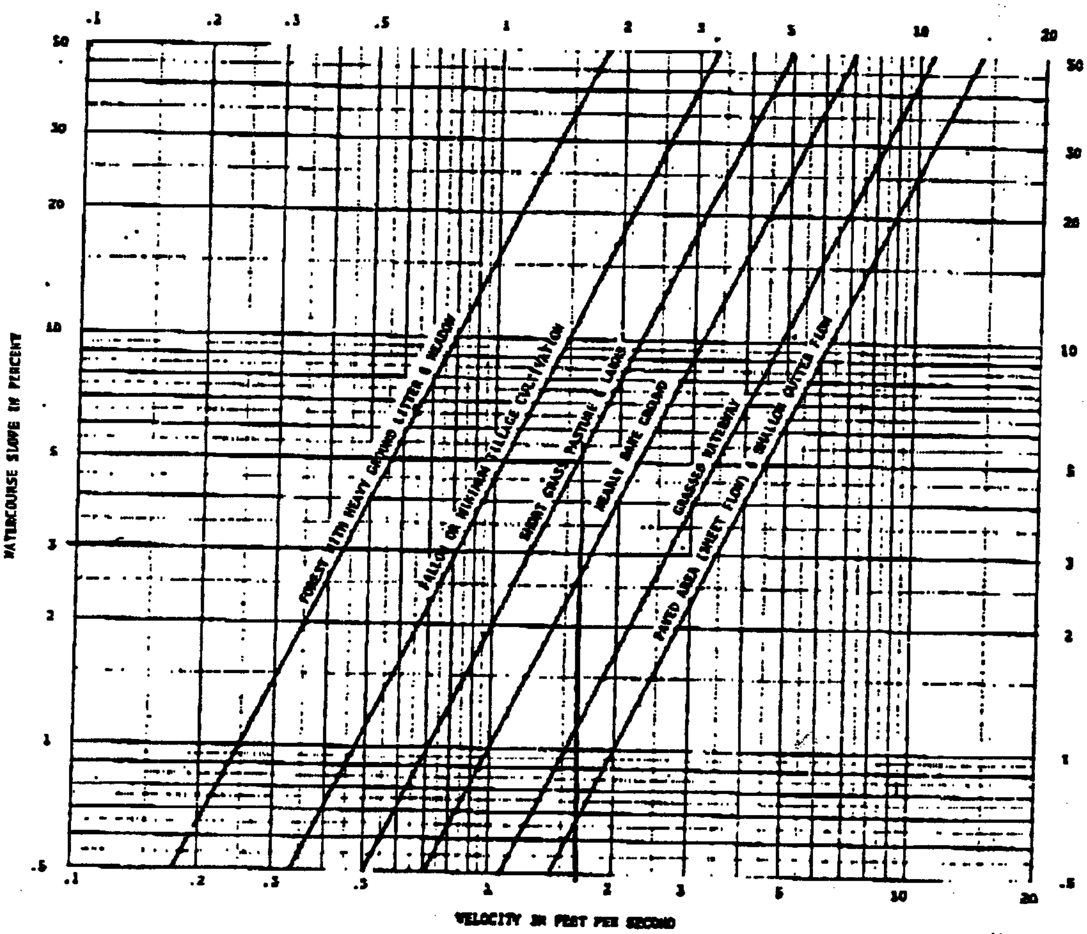


Figure 3-1.--Average velocities for estimating travel time for overland flow.

Storm sewer or road gutter flow

Travel time through the storm sewer or road gutter system to the main open channel is the sum of travel times in each individual component of the system between the uppermost inlet and the outlet. In most cases average velocities can be used without a significant loss of accuracy. During major storm events, the sewer system may be fully taxed and additional overland flow may occur, generally at a significantly lower velocity than the flow in the storm sewers. By using average conduit sizes and an average slope (excluding any vertical drops in the system), the average velocity can be estimated using Manning's formula.

Since the hydraulic radius of a pipe flowing half full is the same as when flowing full, the respective velocities are equal. Travel time may

SOURCE: URBAN Hydrology for SMALL Watersheds, Technical Release NO. 55

SWALE(EAST)			
Determine the design flowrate using the rational formula (Q=CIA)			
Runoff Coeffecient	0.75		
Drainage Area, acres	3		
Intensity, in/hr	4.8		
Flowrate, cfs	10.8		
Manning Equation for a Trapezoidal Ditch			
Iterate using depth			
Y, feet	0.85		1
Base, ft	0		
Z, ft	10		
Area	7.225		
n	0.04		
S	0.005		
Hydraulic Radius	0.422891		
Area	7.225		
Velocity, Ft/sect	1.483983	0.04	
Flow,cfs	10.72178		0.884881

17/19

SWALE(CENTRAL)			
Determine the design flowrate using the rational formula (Q=CIA)			
Runoff Coeffecient	0.75		
Drainage Area, acres	7.4		
Intensity, in/hr	3.8		
Flowrate, cfs	21.09		
Manning Equation for a Trapezoidal Ditch			
Iterate using depth			
Y, feet	1.1		1
Base, ft	0		
Z, ft	10		
Area	12.1		
n	0.04		
S	0.005		
Hydraulic Radius	0.54727		
Area	12.1		
Velocity, Ft/sect	1.762294	0.04	
Flow,cfs	21.32376		0.984881

18/19

SWALE(WEST)			
Determine the design flowrate using the rational formula (Q=CIA)			
Runoff Coeffecient	0.75		
Drainage Area, acres	5.2		
Intensity, in/hr	3.8		
Flowrate, cfs	14.82		
Manning Equation for a Trapezoidal Ditch			
Iterate using depth			
Y, feet	0.95		1
Base, ft	0		
Z, ft	10		
Area	9.025		
n	0.04		
S	0.005		
Hydraulic Radius	0.472643		
Area	9.025		
Velocity, Ft/sect	1.598204	0.04	
Flow,cfs	14.42379		0.984881

19/19

APPENDIX A.6

Cofferdam Stability Analysis (for Reference Only)

A.6 BULKHEAD AND COFFERDAM STABILITY ANALYSES

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A.6 BULKHEAD AND COFFERDAM STABILITY ANALYSES

A.6.1 Introduction and Summary

The stability analyses were performed using the University of Texas Analysis of Slopes, Version 2 (UTEXAS2) computer program. The stability of the bulkhead was analyzed under various conditions, including end of construction, long term, seismic loading (0.09 seismic coefficient), and rapid drawdown of the river from the 100-year flood level. The stability of the cofferdam, which will be a temporary structure, was analyzed only for the end-of-construction condition. Three cross-sections were considered in the analysis of the bulkhead stability, representing the range of geologic conditions along the bulkhead alignment. The geologic profile which generally resulted in the lowest factors of safety for the bulkhead stability was used for the cofferdam stability analysis. Table A.6-1 summarizes the results of the bulkhead and cofferdam stability analyses.

A.6.2 Embankment Configurations

The bulkhead and cofferdam configurations used in the stability analyses were adopted in the design drawings (Appendix D). Side slopes for both structures are two horizontal to one vertical. The bulkhead consists of a compacted embankment with a soil-bentonite slurry wall. Landfill material rests against the entire height of the bulkhead. The surface of the landfill, which consists of two feet of composite cap material, is sloped at five percent. The cofferdam has geomembrane on the up-gradient slope. A geomembrane or other impermeable barrier is assumed to be placed over the mudline, underneath the cofferdam to reduce seepage through the cofferdam. The bulkhead and cofferdam cross-sections are shown in Figure A.6-1.

A.6.3 Geologic Profile and Geotechnical Parameters

Three cross-sections were considered in the analysis of the bulkhead stability, representing the range of geologic conditions along the bulkhead alignment. The locations of these cross-sections are shown by Figure A.6-2. The geologic profile at cross-section number 3, which generally resulted in the lowest factors of safety for the bulkhead stability, was used for the cofferdam stability analysis. Figure A.6-3 consists of a geologic profile along the bulkhead alignment compiled from boring logs and showing the results of field tests as well as the locations and depths of soil samples collected, the types of laboratory analysis performed on the samples, and the summarized analysis results. The boring logs and most of the detailed geotechnical field and laboratory test data is contained in the Predesign Field Activity Report (20 November 1992). More recent additional laboratory test results are also included in the Figure A.6-3 compilation. This new laboratory test data is contained in its entirety in Section A.8 of this Appendix.

Based on the boring logs and geotechnical test results, the following soil deposits were identified beneath the bulkhead alignment:

- o Alluvium sediments
- o Clay
- o Ablation till
- o Glacial till.

As shown by Figure A.6-3, the clay layer is present only at cross-section number 2, and the ablation till exists only at cross-sections number 1 and 3.

In addition to these soil layers, three types of placed soils are modeled for the stability analyses, including:

- o Embankment material (bulkhead and cofferdam)
- o Landfill material (bulkhead only)
- o Soil-bentonite slurry wall (bulkhead only).

Table A.6-2 summarizes the shear strength parameters and total unit weights adopted for each of the above types of soil for use in the stability analyses. The basis for adopting these parameters is discussed in the following paragraphs:

Alluvium Sediments. The shear strength of the alluvium sediments, which consist of silty sand, is characterized by an effective friction angle of 37 degrees based on the range of consolidated undrained (CU) triaxial shear test data, and typical values that have been obtained for similar types of soil. A total unit weight of 125 pounds per cubic foot (pcf) was calculated based on the dry density and moisture content data.

Clay. Under conditions of relatively rapid loading (i.e., end of construction, seismic loading, and rapid drawdown) the shear strength of the clay layer is characterized as an undrained shear strength (S_u) as a function of the effective vertical confining pressure (p'). CU triaxial tests were performed at three effective consolidation pressures; the first is roughly equal to the in-situ p', the second is twice the in-situ p', and the third is four times the in-situ p'. The S_u/p' ratios obtained (see Table A.6-3) indicate that the clay is currently overconsolidated. For end-of-construction conditions, the clay is assumed to have insufficient time to consolidate under the load of the newly placed embankment, so that it remains at its present overconsolidation ratio (OCR). The adopted end-of-construction S_u/p' therefore corresponds with the CU test results for the in-situ p' consolidation pressure, the S_u at any depth level being determined without the benefit of the added p' due to the embankment weight. For long-term rapid loading conditions (i.e. seismic and rapid drawdown), it is assumed that the clay has reached a normally consolidated state due to the embankment surcharge. For these conditions, the adopted S_u/p' value corresponds with the CU test results obtained when the effective consolidation pressure was four times the the current in-situ p'. For long-term static stability analysis, the clay's shear strength is characterized by an effective friction angle of 15 degrees, based on the CU test results. The total unit weight of the clay is estimated at 110 pcf, based on typical values for this type of soil.

Ablation Till. The deposition history of the ablation till is assumed to be similar to that of the clay, and therefore the OCR of the ablation till is roughly equal to that of the clay. Based on the unconsolidated undrained (UU) triaxial shear test, the current S_u/p' of the ablation till is about 0.4, indicating that this material may be somewhat stronger than the clay. This is supported by the standard penetration test (SPT) N-values. The adopted strength and unit weight values are therefore slightly higher than those for the clay.

Glacial Till. This material is considerably stronger than the overlying soils based on the results of the UU tests. Because of its relative strength, this soil is not critical to the stability analyses; all of the conceivable slope failure surfaces pass above this layer.

Embankment Material. For the bulkhead, local borrow sources will be used which tend to consist mostly of silty sand. This material will be compacted during placement. The cofferdam will be constructed by dumping a clean gravel. The adopted strength and unit weight parameters for these materials are based on typical values generally easily achieved with these types of materials. The adopted parameters are the same for the clean gravel and the silty sand because the typically higher strength of clean gravels is in this case probably offset by the fact that this material will be dumped into place, whereas the silty sand will be compacted.

Landfill Material. The landfill material behind the bulkhead will consist of excavated alluvium sediments which will be compacted during placement. As a conservative estimate of the strength of this material, its geotechnical parameters are equated to those of the in-situ alluvium sediments.

Soil-Bentonite Slurry Wall. The relatively low shear strength values adopted for this material represent lower bound conditions applicable during construction of the slurry wall. These conditions will exist for a relatively short duration. Over the long term, the strength of the slurry wall is expected to increase significantly. However, due to uncertainty in the actual eventual strength of the slurry wall, the lower bound values are retained for the long term analysis conditions.

A.6.4 Analysis Methodology and Results

The University of Texas Analysis of Slopes - Version 2 (UTEXAS2) computer program, which was used for these analyses, was developed by Stephen Wright of the Civil Engineering Department at the University of Texas in Austin. This program is capable of locating critical failure surfaces and computing factors of safety using a variety of analysis procedures. The procedure recommended by the program's author, Spencer's Method, was used for all analyses.

The program can search for circular and non-circular critical failure surfaces. Non-circular failure surfaces through the weakest layers (slurry wall, clay or ablation till) were investigated for the end-of-construction and seismic loading cases. Circular failure surfaces were investigated for all analyses. None of the non-circular failure surfaces resulted in computed factors of safety lower than those for the critical circular failure surface.

The program searches for critical failure circles based on the center coordinates and radius that must be input for an initial trial circle. The center coordinates and radius are then incrementally modified by the program in a systematic manner, and the new factor of safety calculated each time, enabling the critical failure circle to be eventually determined. The initial trial circle must be defined with a certain degree of realism and accuracy to ensure that the actual critical circle is identified by the program. For all but the rapid drawdown analysis, the critical failure circle was initially assumed to be a relatively deep circle tangent to a point within the underlying weak layer (clay or ablation till), and the initial trial circle was input accordingly. This initial assumption was proved correct by the results of the analyses. For the rapid drawdown condition, initial trial circles were specified both deep and shallow. The rapid drawdown analysis results indicated that the shallow failure mode was the most critical. The rapid drawdown analysis simulated the sudden lowering of the river level from the 100-year flood elevation to the mean water elevation.

Both total and effective stress analyses were performed. Total stress strength parameters were used for rapid loading conditions (i.e., end of construction, seismic and rapid drawdown) in undrained soil layers (i.e., slurry wall, clay, ablation till, and glacial till). Effective stress parameters were used for long term static stability analyses, and for free-draining materials (embankment, landfill and alluvium). The program allows pore pressures to be defined and used selectively for specified layers within a profile, so that a combined total and effective analysis can be done, as required per the drainage characteristics of individual layers.

Table A.6-1 summarizes the computed factors of safety. The critical failure surface computed for each condition analyzed is shown in Figures A.6-4 through A.6-16.

Table A.6-1 Summary of Stability Analyses

Embankment Analyzed	Cross-Section	Condition Analyzed	Factor of Safety
Bulkhead	Cross-Section No. 1 (OLIN Property)	End of Construction	2.04
		Long Term Stability	2.48
		Rapid Drawdown (From 100-yr. Flood Level)	1.26
		Seismic Loading (0.09 Seimic Coefficient)	1.67
	Cross-Section No. 2 (East End of OXYCHEM Property)	End of Construction	2.25
		Long Term Stability	2.63
		Rapid Drawdown (From 100-yr. Flood Level)	1.26
		Seismic Loading (0.09 Seismic Coefficient)	1.62
	Cross-Section No. 3 (West End of OXYCHEM Property)	End of Construction	1.54
		Long Term Stability	2.16
		Rapid Drawdown (From 100-yr. Flood Level)	1.26
		Seismic Loading (0.09 Seismic Coefficient)	1.27
Cofferdam		End of Construction	1.43

Table A.6-2 Geotechnical Parameters Used for Stability Analyses

Soil Layer	Total Unit Weight (pcf)	Soil Strength Parameters		
		End of Construction Stability	Long Term Rapid Drawdown & Seismic Stability	Long Term Static Stability
Alluvium Sediments	125	Effective Phi = 37°		
Clay	110 (Est.)	S_u/p' = 0.30	S_u/p' = 0.20	Effective Phi = 15°
Ablation Till	115 (Est.)	S_u/p' = 0.35	S_u/p' = 0.25	Effective Phi = 20°
Glacial Till	130 (Est.)	S_u = 2,000 psf		Effective Phi = 40° (Est.)
Embankment Material	130 (Est.)	Effective Phi = 40° (Est.)		
Landfill Material	125 (Est.)	Effective Phi = 37° (Est.)		
Soil-Bentonite Slurry Wall	100 (Est.)	S_u/p' = 0.15 (Est.)		Effective Phi = 8° (Est.)

EXPLANATION OF SYMBOLS AND ABBREVIATIONS:

pcf = pounds per cubic foot
psf = pounds per square foot
Est. = estimated parameter (no laboratory data was available)
Phi = angle of internal friction
S_u = undrained shear strength
p' = effective vertical overburden pressure

Table A.6-3 Consolidated Undrained Triaxial Shear Test Results for Clay Samples

Effective Consolidation Pressure	Sample No.	Effective Friction Angle	S_u/p'
In-Situ p'	27-3	26°	0.29
	28-1	26°	0.40
	28-2[1]	38°	0.70
2 x In-Situ p'	28-1	22°	0.26
	28-2[1]	32°	0.48
4 x In-Situ p'	27-3	16°	0.17
	28-1	15°	0.21
	28-2[1]	26°	0.25

[1] This sample was collected at or near the clay/glacial till interface, and is probably not representative of the clay layer.

FORM E50 (Rev. 8-91)

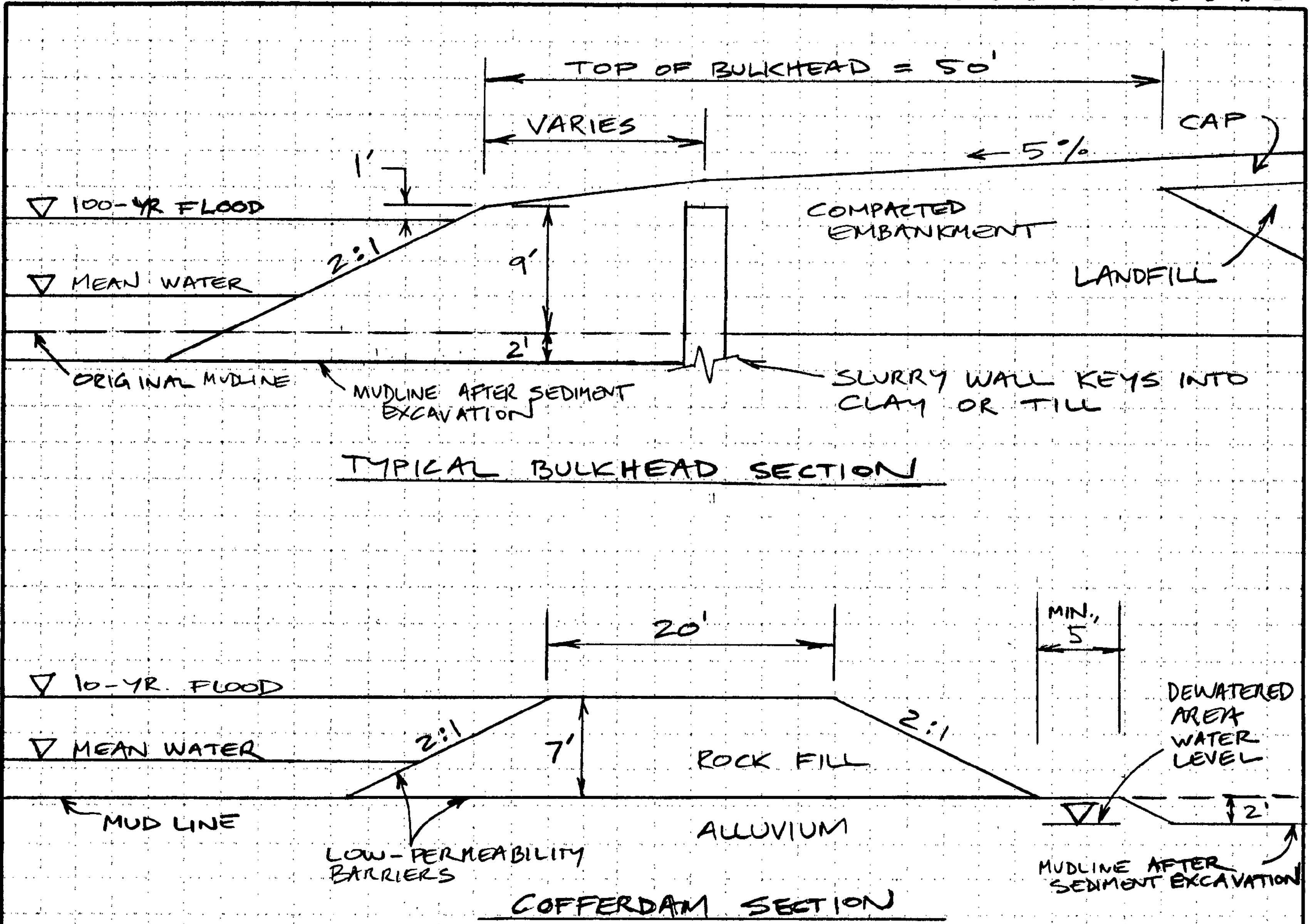


FIGURE A.6-1 BULKHEAD & COFFERDAM CONFIGURATIONS

FORM E50 (Rev. 8-91)

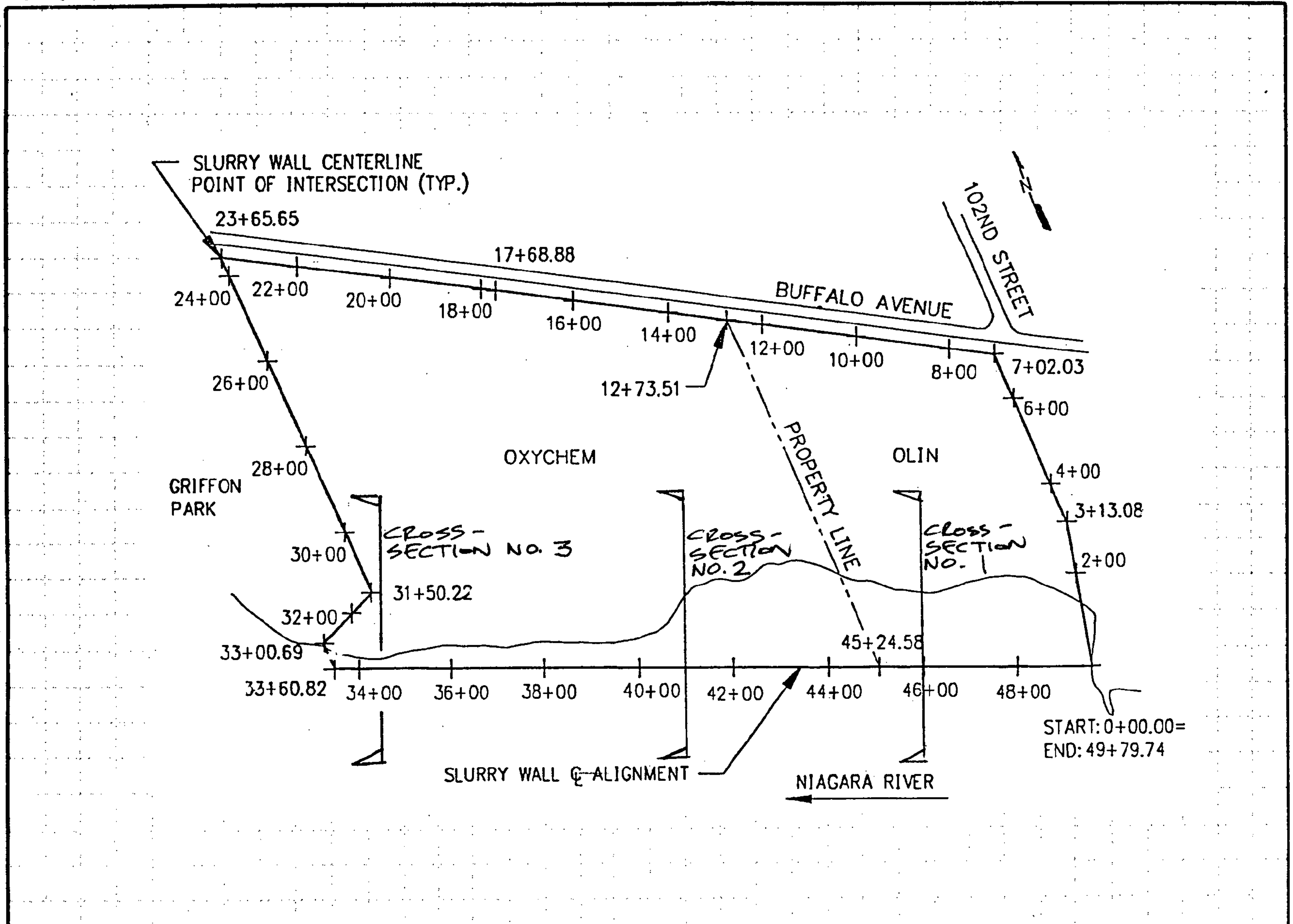
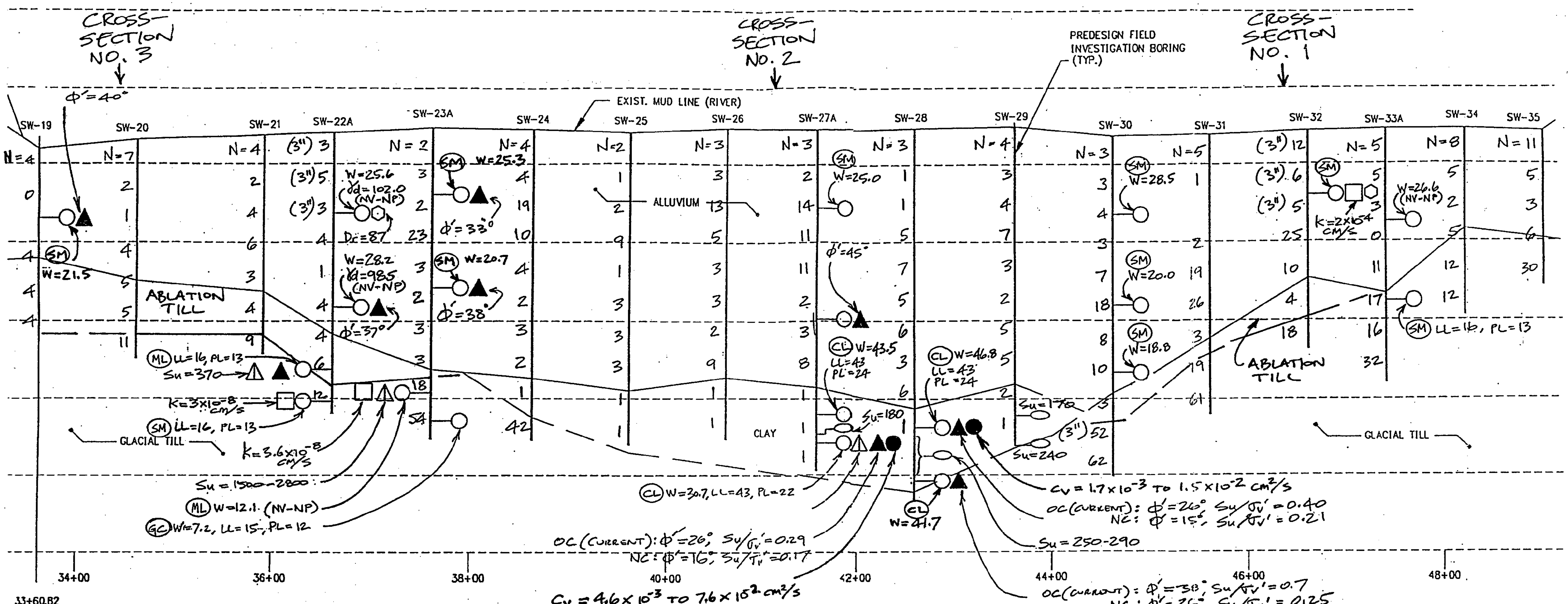


FIGURE A.6-2 CROSS-SECTION LOCATIONS



UNLESS OTHERWISE SPECIFIED ALL UNITS ARE IN POUNDS AND FEET

SOIL TESING LEGEND:

- ○ INDEX PROPERTIES
- □ HYDRAULIC CONDUCTIVITY
- △ UU TRIAXIAL
- ▲ CIUC TRIAXIAL
- ⬡ RELATIVE DENSITY
- △ UNCONFINED COMPRESSION
- ● CONSOLIDATION
- ⬭ VANE SHEAR

HORIZONTAL SCALE 100'

VERTICAL SCALE 5'

FIGURE A.6-3

GEOLOGIC PROFILE ALONG BULKHEAD ALIGNMENT

FIGURE A.6-4 BULKHEAD, OLIN PROPERTY, CROSS-SECTION NO. 1

END OF CONSTRUCTION

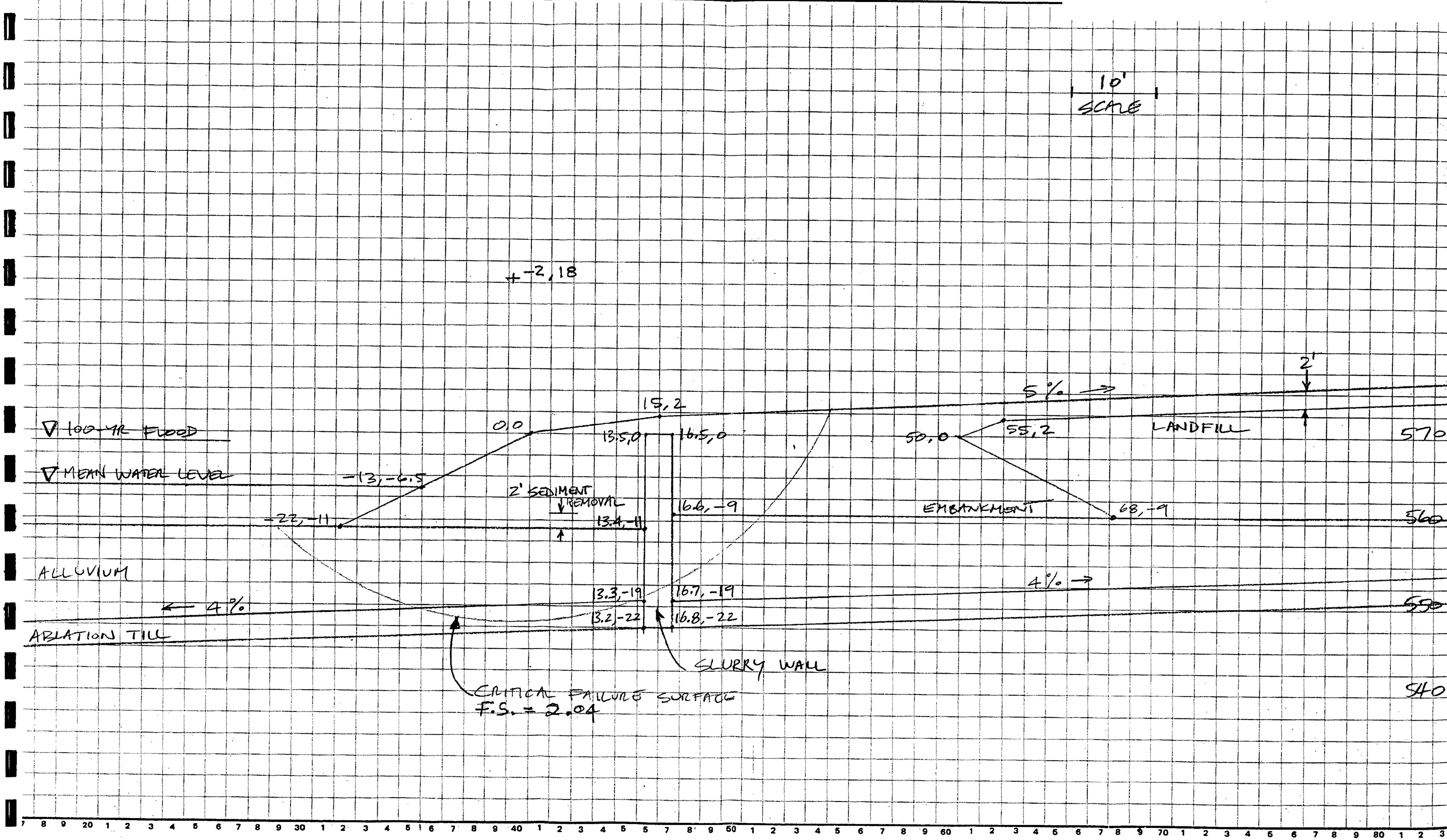


FIGURE A.6-5 BULKHEAD, OLIN PROPERTY, CROSS-SECTION NO. 1

LONG TERM

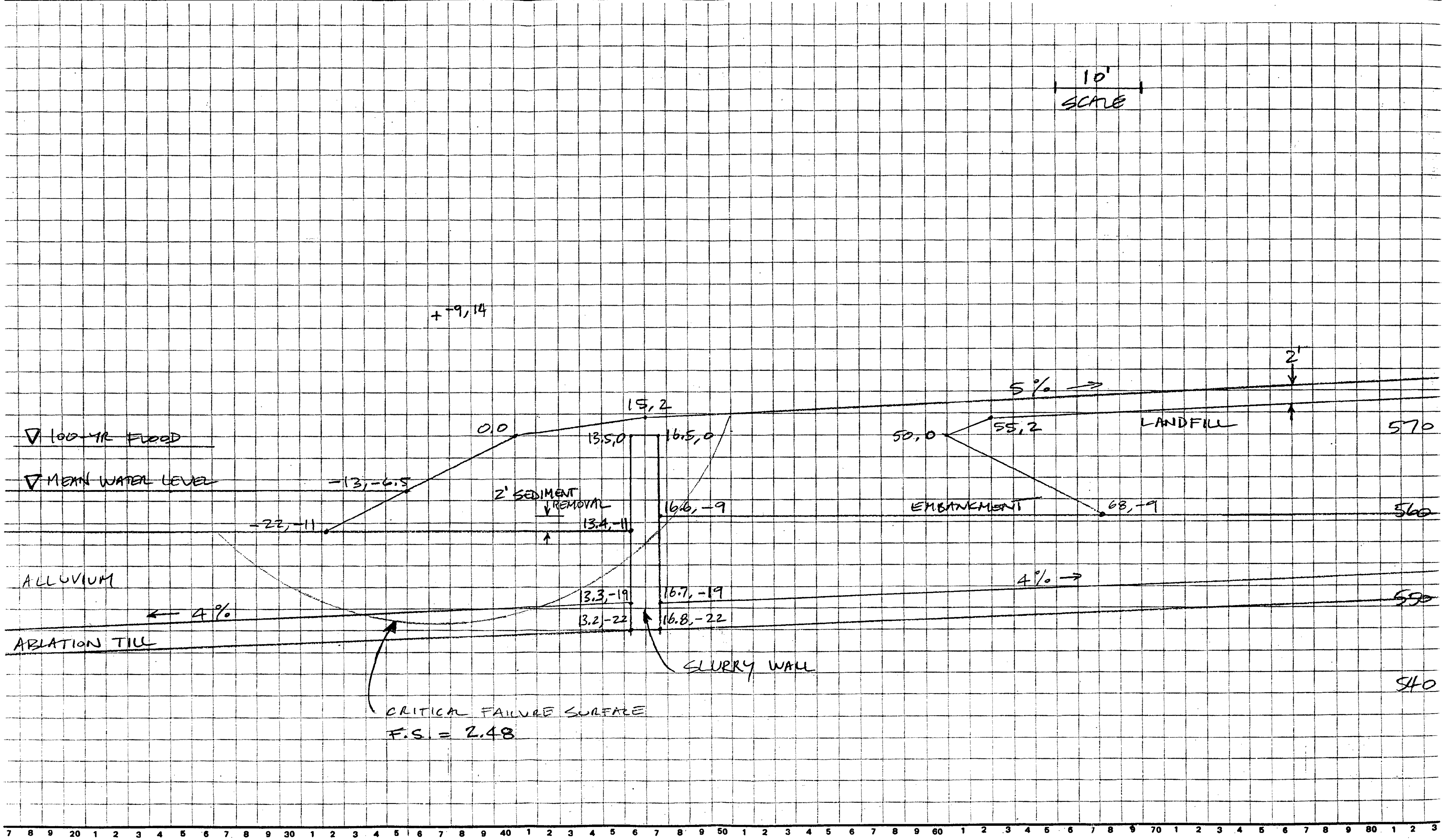


FIGURE A.6-6 BULKHEAD, OLIN PROPERTY, CROSS-SECTION No. 1

RAPID DRAWDOWN

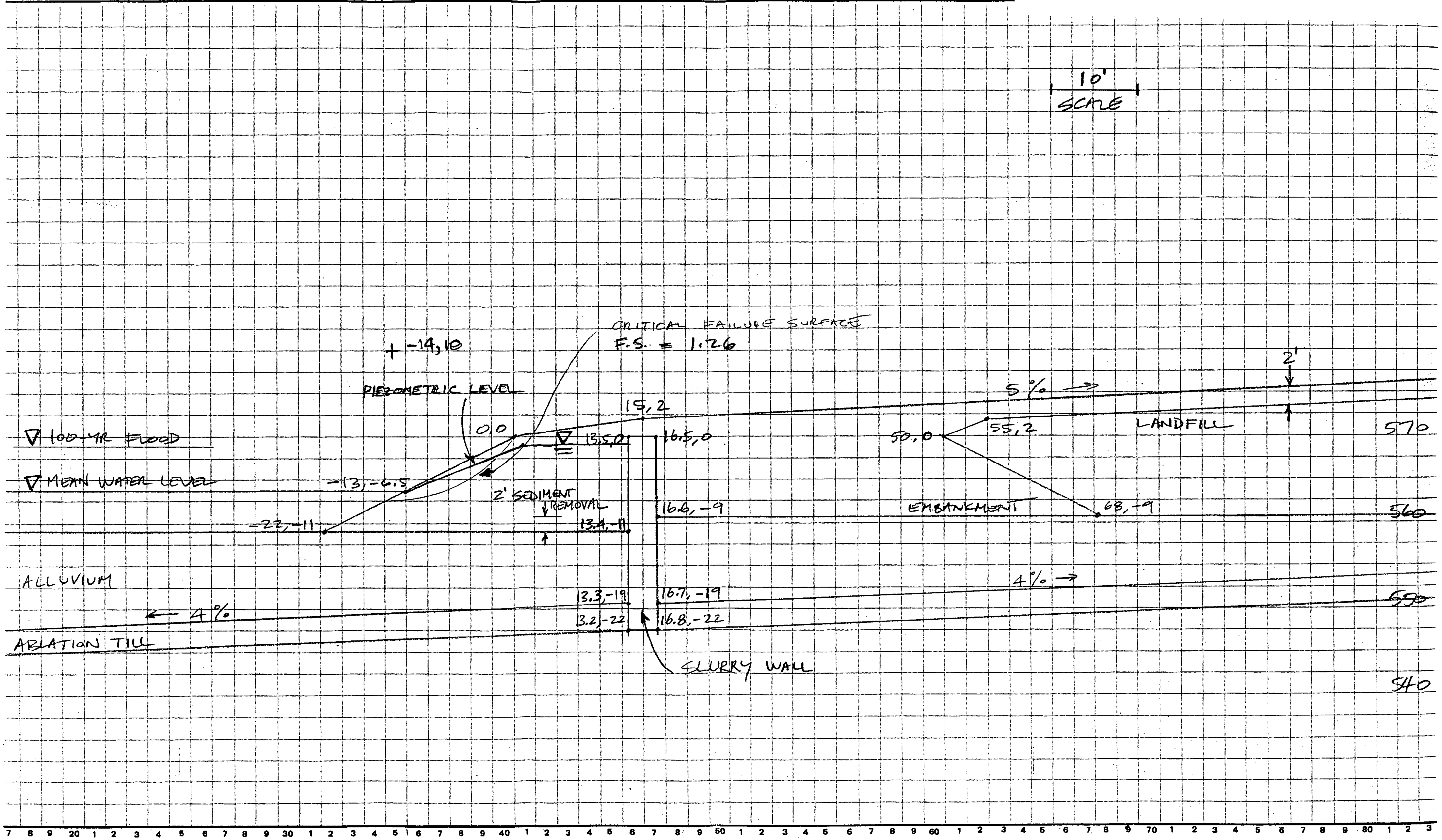


FIGURE A.6-7 BULKHEAD, OLIN PROPERTY, CROSS-SECTION NO. 1

SEISMIC (0.09 SEISMIC COEFF.)

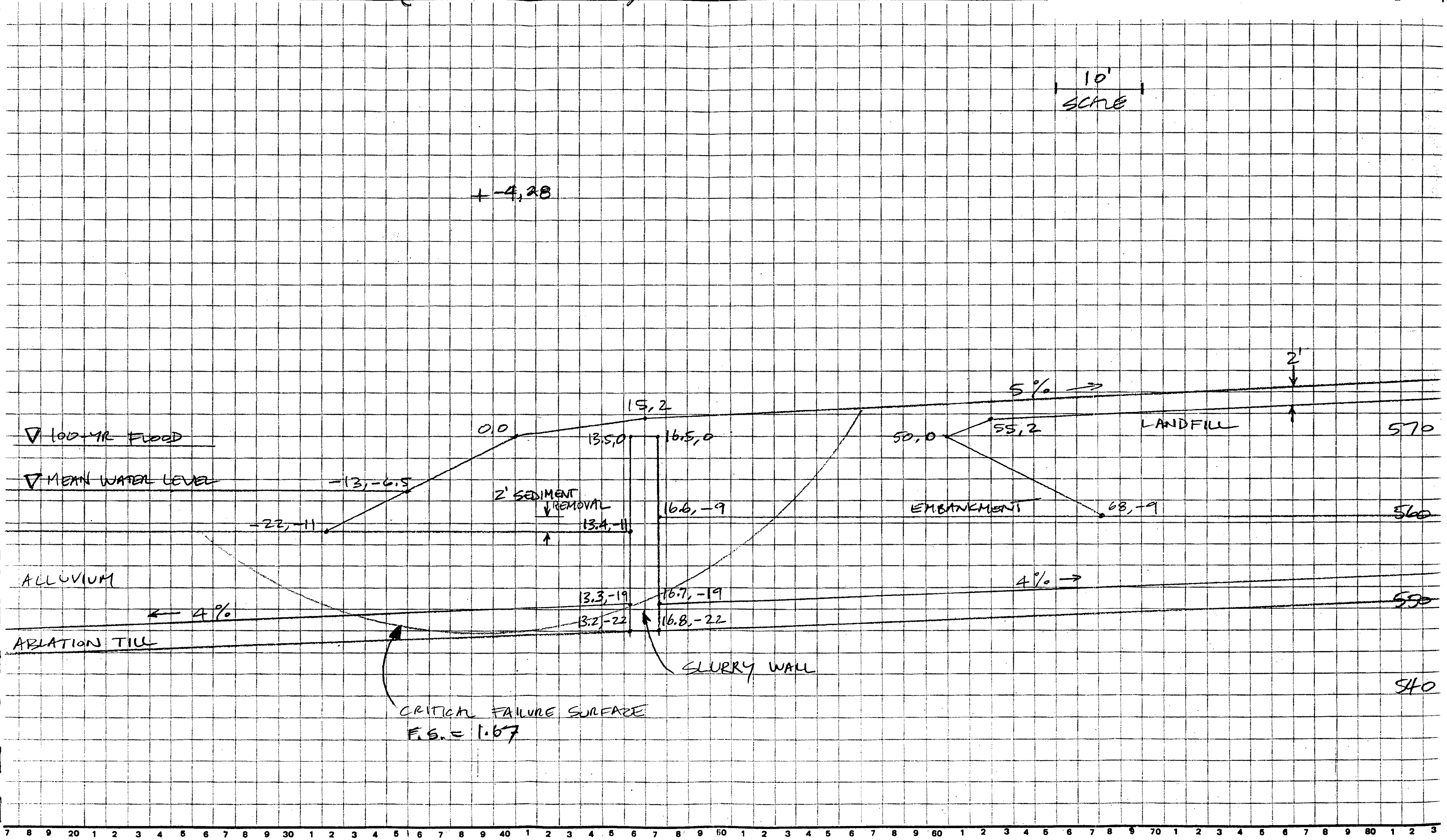


FIGURE A.6-8 BULKHEAD, EAST END OF OXYCHEM PROPERTY, CROSS-SECTION NO. 2, END OF CONSTRUCTION

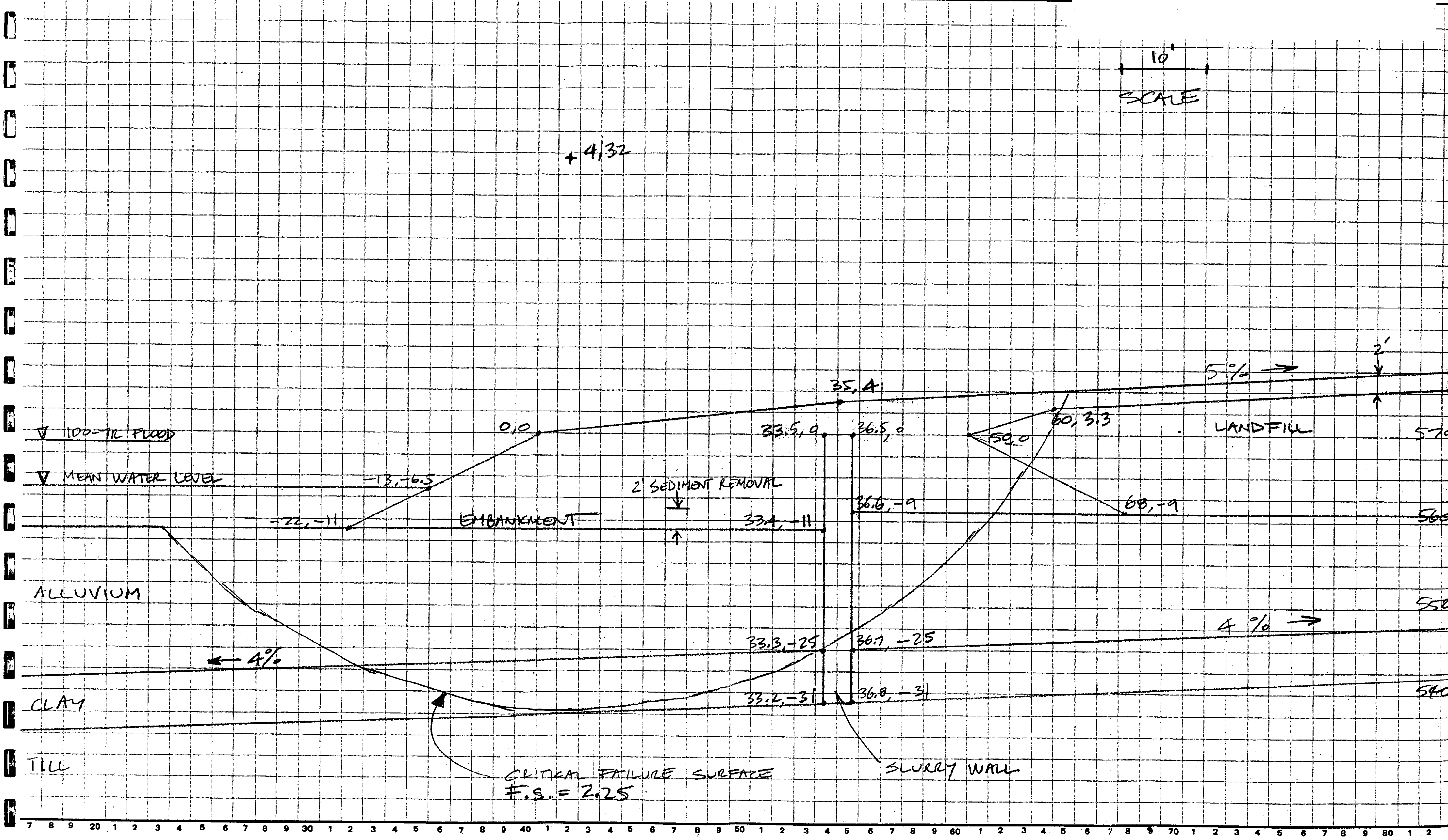


FIGURE A.6-9 BULKHEAD, EAST END OF OXYCHEM PROPERTY, CROSS-SECTION NO. 2

LONG TERM

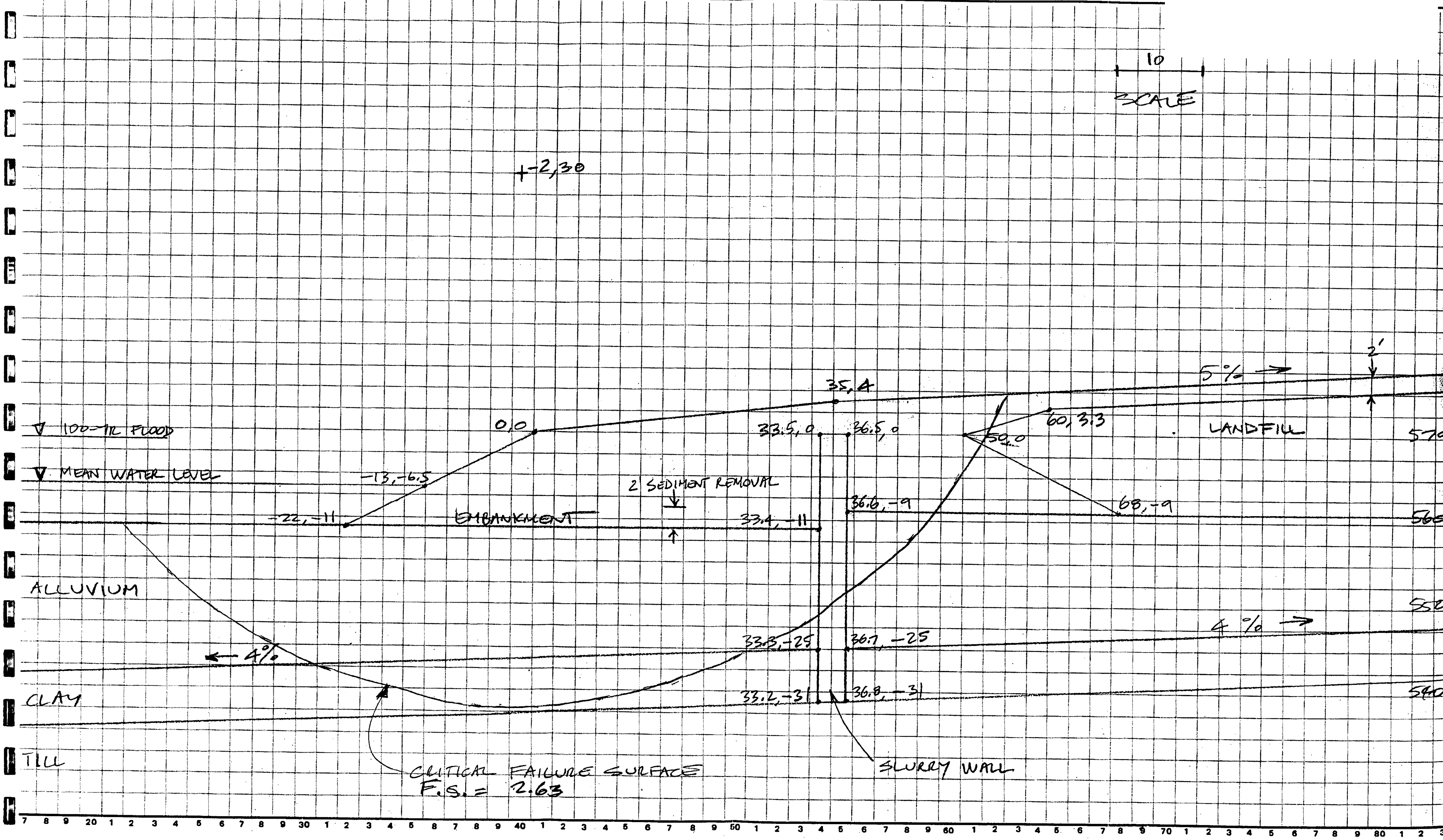


FIG. A.6-10 BULKHEAD, EAST END OF OXYCHEM PROPERTY, CROSS-SECTION NO. 2

RAPID DRAWDOWN

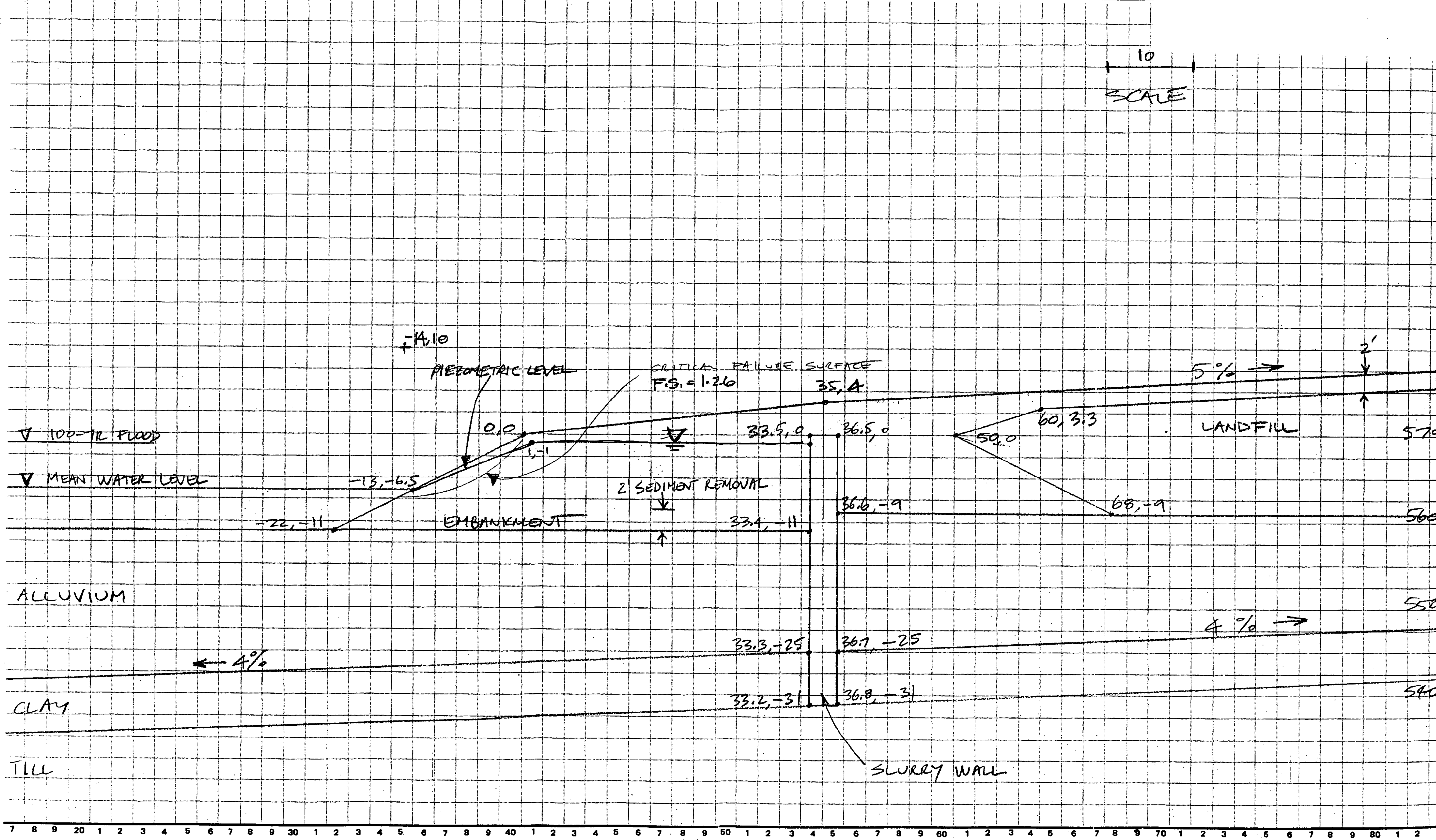


FIG. A.6-11 BULKHEAD, EAST END OF OXYCHEM PROPERTY, CROSS-SECTION NO. 2

SEISMIC (0.09 SEISMIC COEFFICIENT)

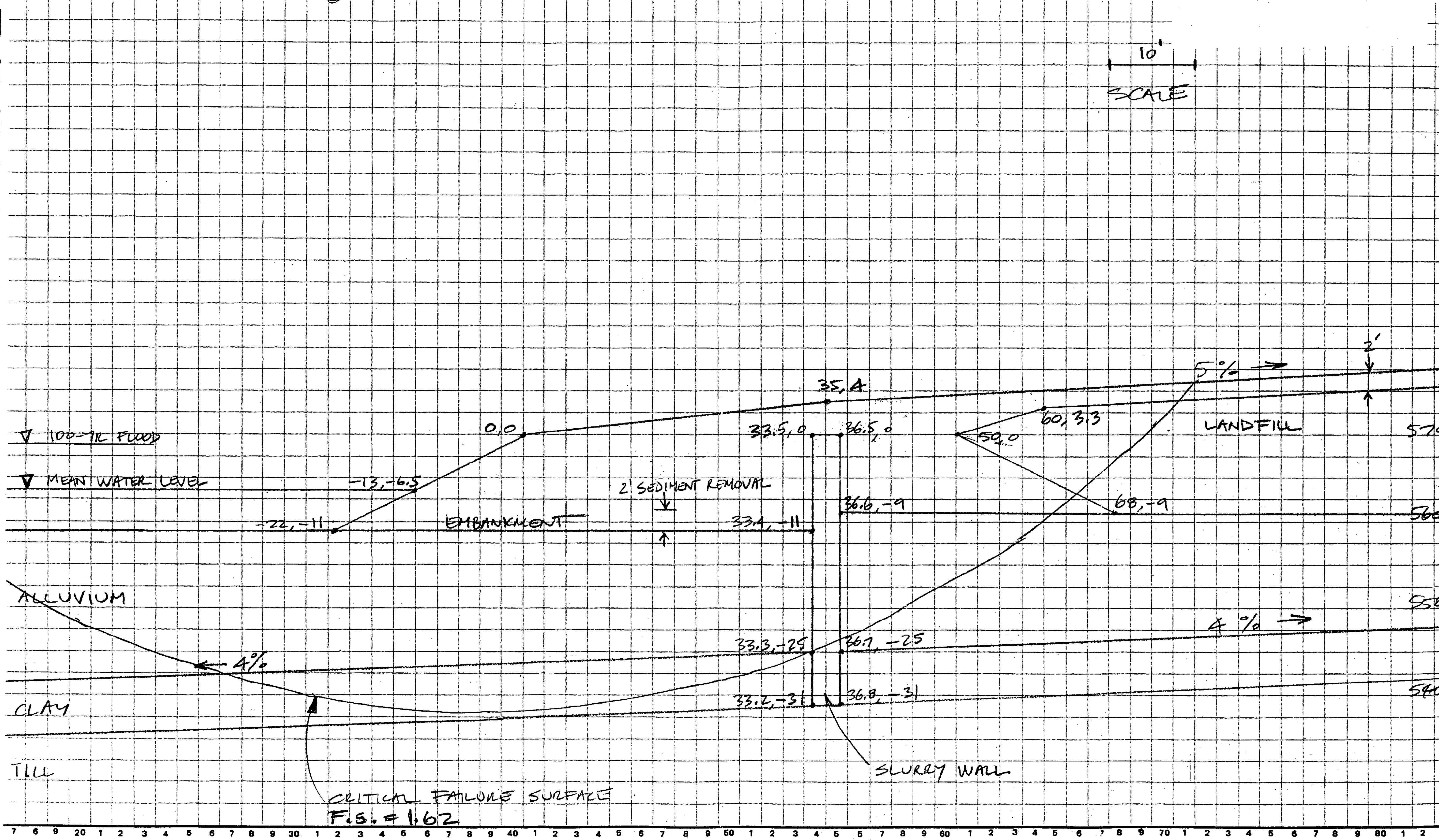


FIGURE A.6-12 BULKHEAD, WEST END OF OXYCHEM PROPERTY, CROSS-SECTION NO. 3
END OF CONSTRUCTION

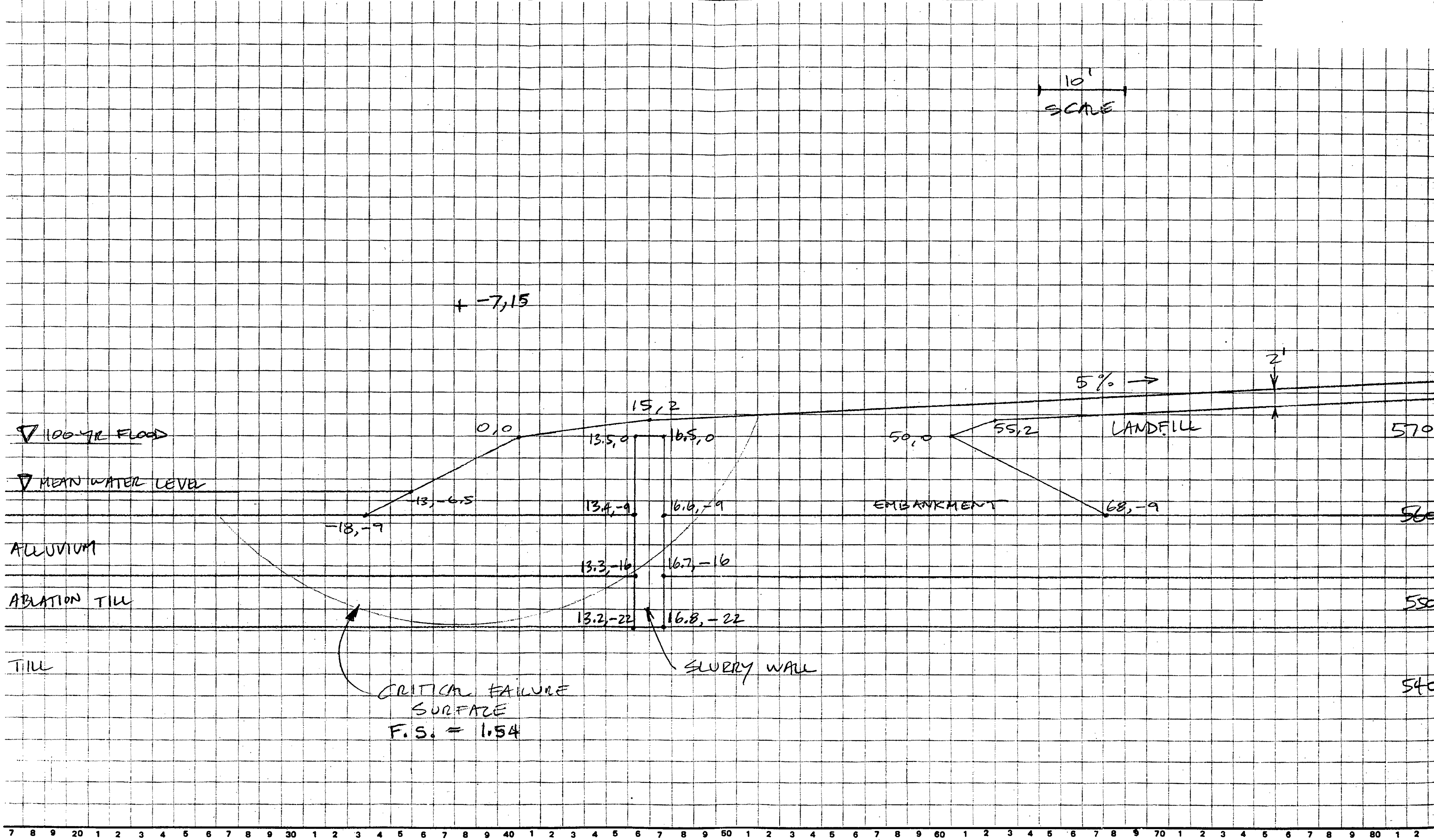


FIGURE A.6-13 BULKHEAD, WEST END OF OXYCHEM PROPERTY, CROSS-SECTION NO. 3

LONG TERM

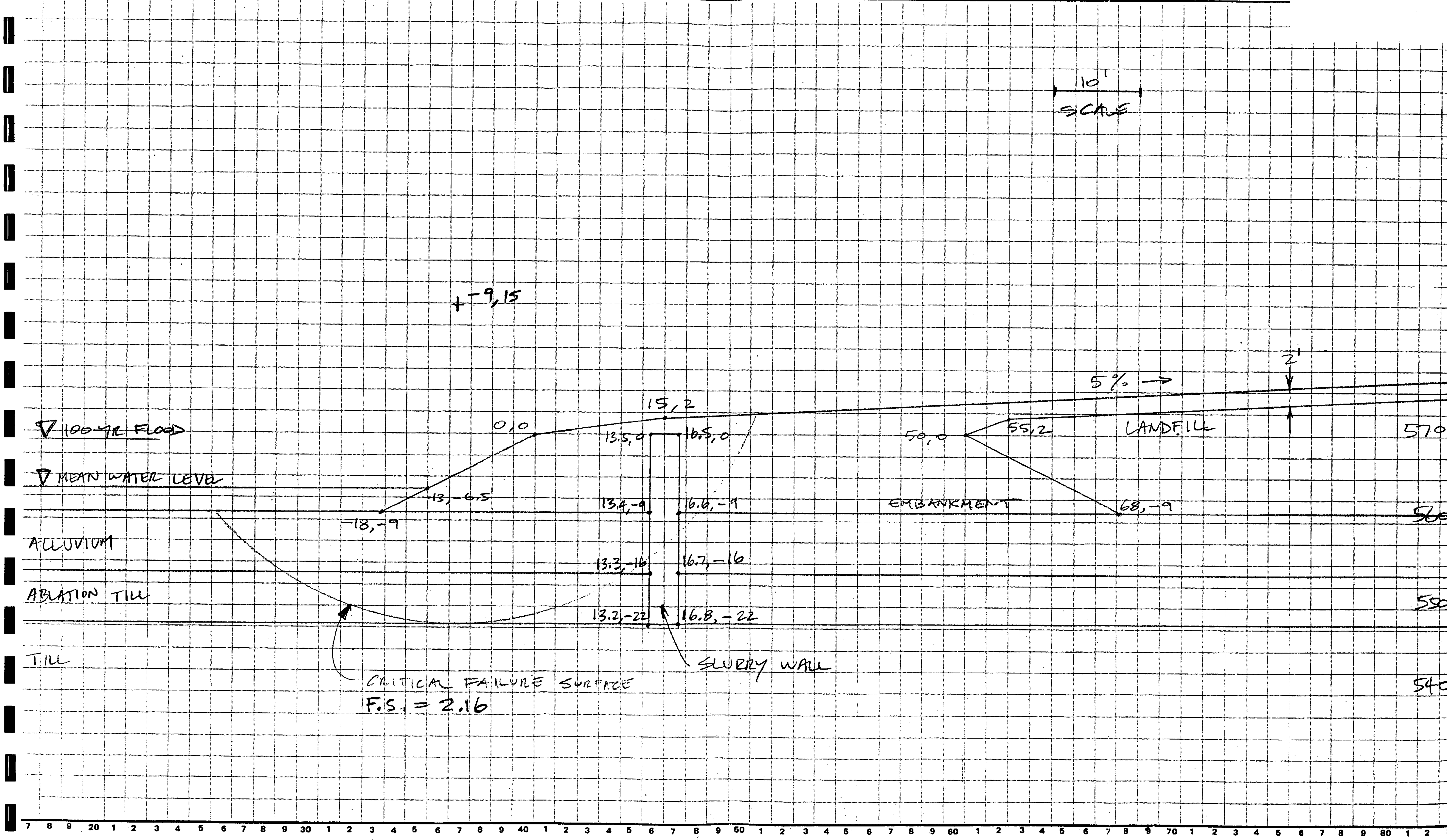


FIGURE A.6-14 BULKHEAD, WEST END OF OXYCHEM PROPERTY, CROSS-SECTION NO. 3

RAPID DRAWDOWN

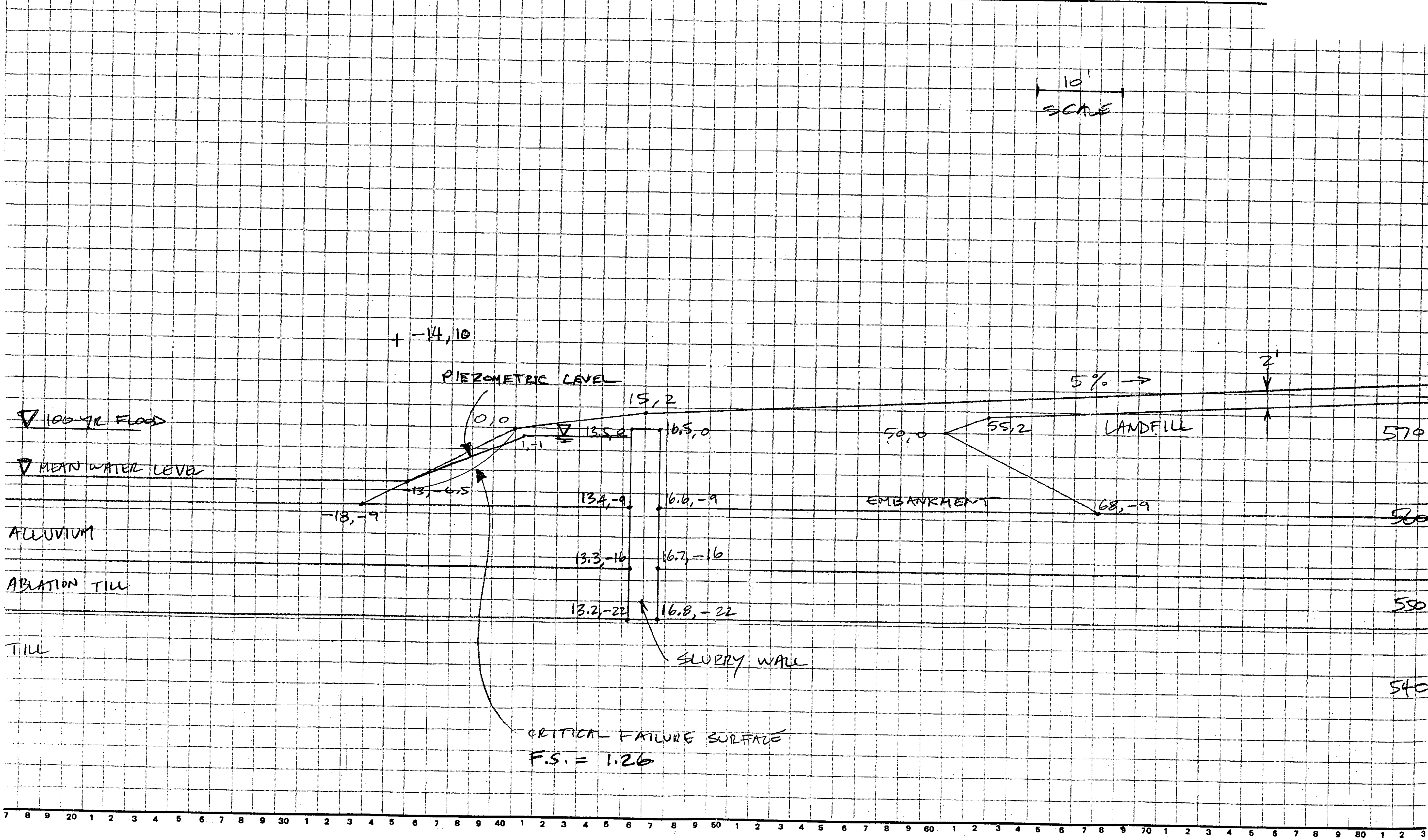


FIGURE A.6-15 BULKHEAD, WEST END OF OXYCHEM PROPERTY, CROSS-SECTION NO. 3

SEISMIC (0.09 SEISMIC COEFF.)

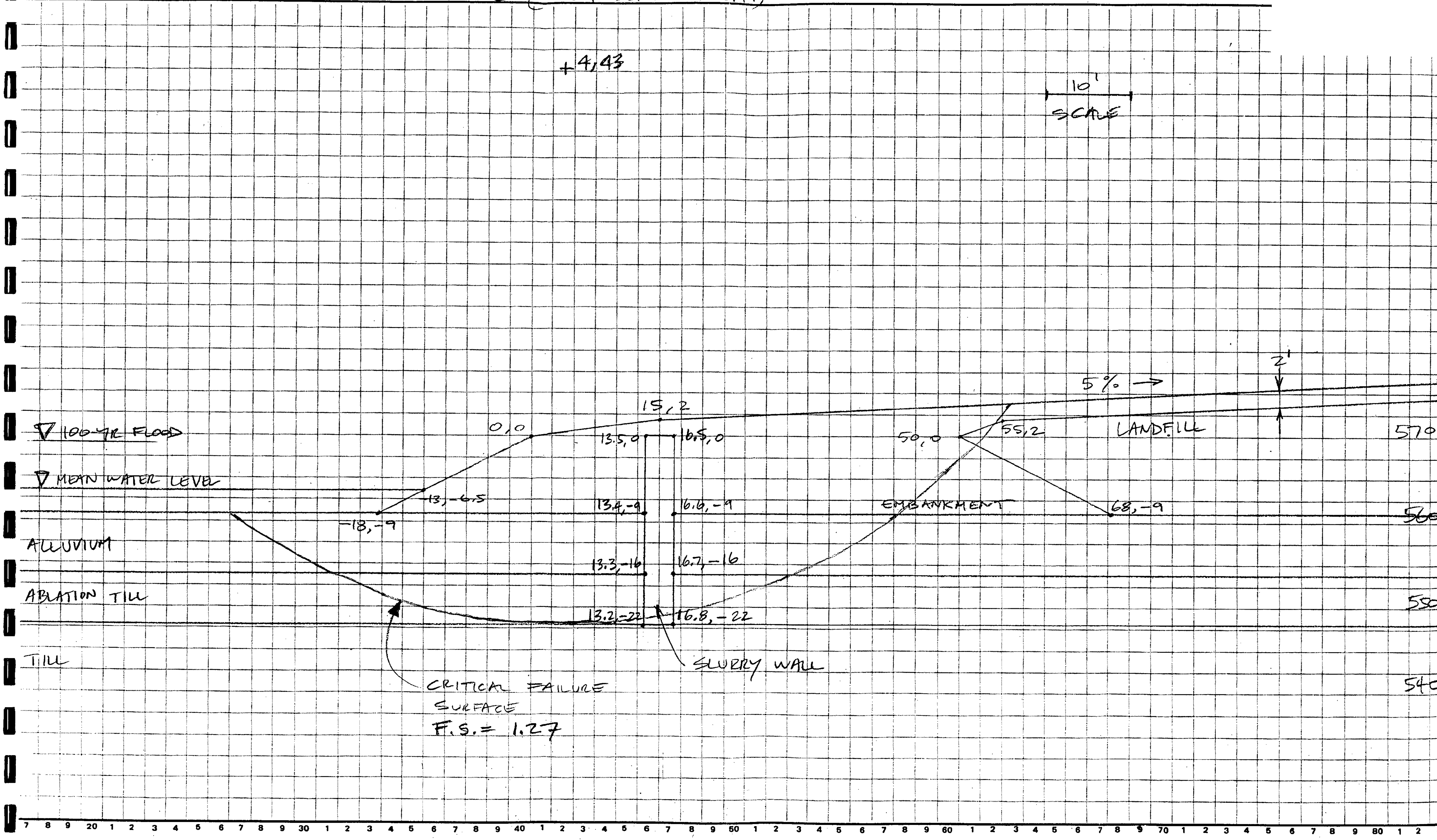
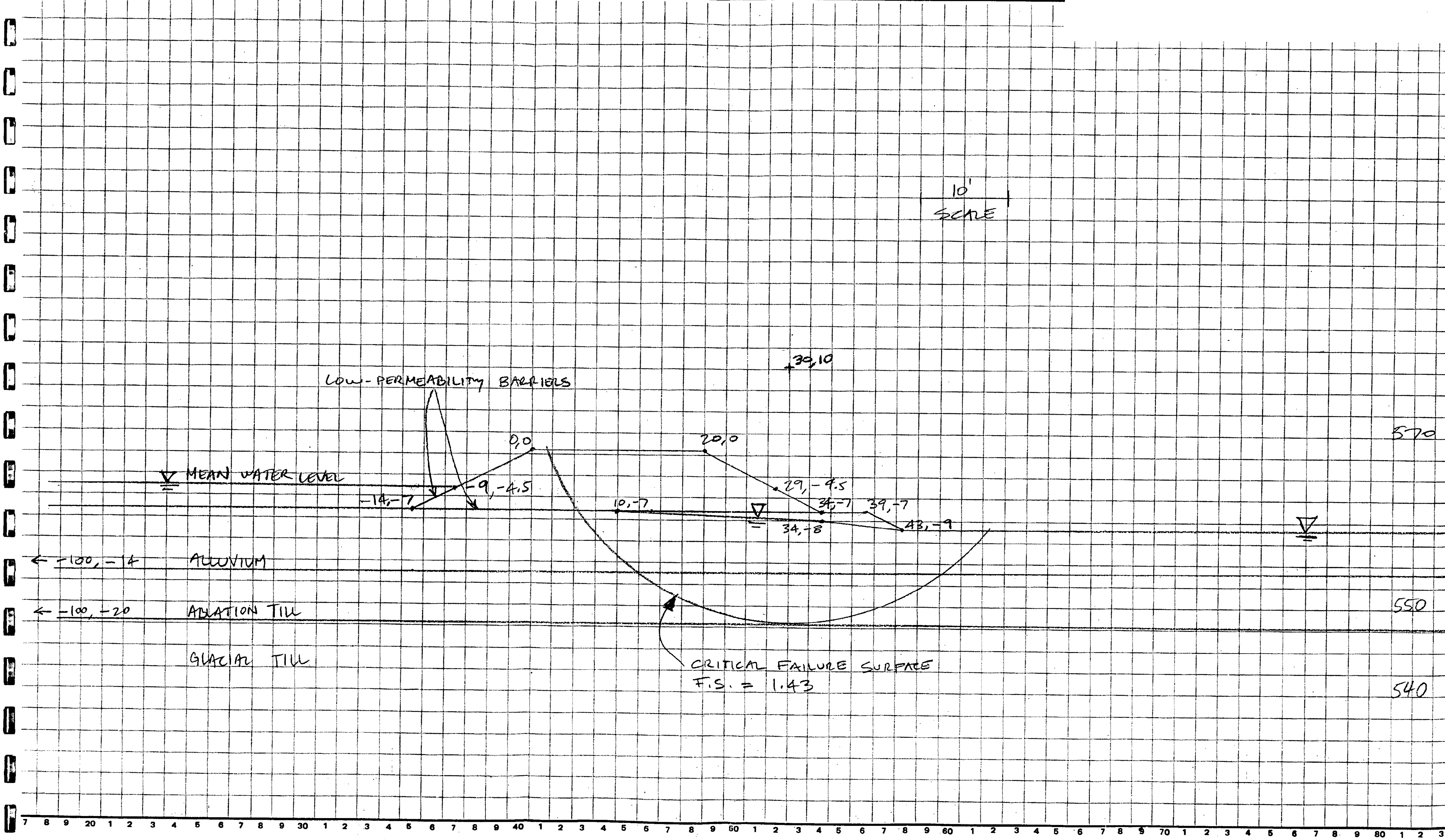


FIGURE A.6-16 COFFERDAM, CROSS-SECTION NO. 3

END OF CONSTRUCTION



APPENDIX A.7

Seepage Analysis

FORM E50 (Rev. 8-91)

ESTIMATE OF
SEEPAGE BENEATH COFFERDAM,

$Q = \dfrac{mKH}{n}$ WHERE m = NO. OF FLOW PATHS $\doteq 3$

n = NO. OF POTENTIAL DROPS $\doteq 10$

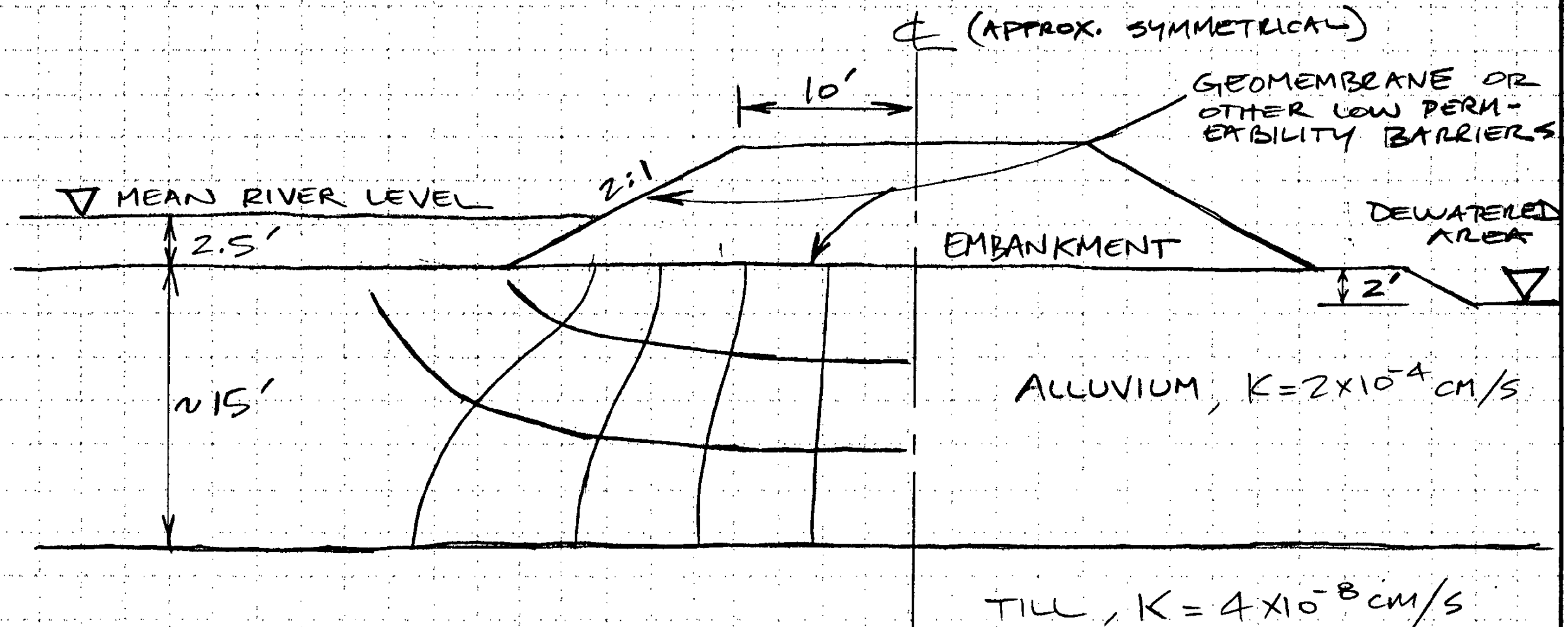
K = HYDRAULIC CONDUCTIVITY $= 2 \times 10^{-4}$ cm/s

$= 6.6 \times 10^{-6}$ ft/s

H = HEAD DROP = 4.5 ft

$Q \doteq 9 \times 10^{-6}$ ft³/s PER FOOT LENGTH

$\doteq$ 0.004 gpm PER FOOT LENGTH



APPENDIX A.8

Settlement Calculations

102ND STREET LANDFILL SITE
NIAGARA FALLS, N.Y.
REMEDIAL DESIGN

LANDFILL SETTLEMENT CALCULATIONS

The objective of the landfill settlement calculations is to determine potential settlement that may occur at the 102nd Street Landfill Site after placement of the final cap. The calculated settlement is evaluated to determine the impact on the integrity of the cap and on the storm water drainage pathways.

Settlement calculations were done at six locations to generate a settlement profile from north to south across the landfill. The profile traverses the central mound of the three mounds that will make up the final grades of the Site. A geologic section along the settlement point profile is shown in Figure 1. The geologic profile is based upon profiles and borings from the RI and borings from the Pre-design Field Activities Program.

The soil units of the site were each assigned values for density, void ratio and compression index. These values are provided in Table 1. The values are based upon site-specific data, wherever available. The data available for this Site is summarized in Table 2.

The assumptions that were made to complete the settlement calculations are:

1. The settlement is based upon one dimensional consolidation formulas.

2. Initial void ratio (e_o) for the imported fill and existing fill was not available from the laboratory data. The value used was based upon representative values from Peck, Hanson & Thornburn (P,H&T), Table 1.4, p.13, for loose, mixed-grain sandy soils and dense mixed-grain sandy soils and then interpolated to moderately dense mixed-grain sandy soils.

3. Unit weights for the imported fill were selected based upon the unit weights for dense and loose mixed-grain sandy soils from Table 1.4 of P,H&T and interpolated to moderately dense mixed grain sandy soils. Unit weights of alluvium, onshore clay and offshore clay were taken from available Site data.

4. The ratio of PI/74 for cohesive materials was used to provide a reasonable indication of the compression index. In P,H&T; it is recommended that 0.009*(LL-10) is closely correlated to the compression index. Both of these approaches were used. All compression index values fell within the range of 0.31 to 0.35 for the clay, using both equations, which compared well with the laboratory data available.

5. The water table was assumed to be at the top of the alluvium at the six locations selected to calculate the settlement of the landfill. This assumption is consistent with the historical water table data of the Site from the RI Report.

The formula used to calculate settlements was:

$$S=C_c/(1+e_o)*H*Log(1+\hat{}P/P_o),$$

Where: S=settlement
C_c=Compression Index
e_o=initial void ratio
^P=increased effective stress
P_o=initial effective stress
H=thickness of soil unit

From Peck, Hanson & Thornburn, p. 63, 1974.

Results of the settlement calculations at the six locations and the resulting slope profile (Figure 2) are attached. The available data indicates settlements of up to one foot will take place on the southern portion of the Site. The required minimum grades of three percent are maintained on the slopes even with up to one foot of settlement.

The variable settlement of the landfill will result in some local steepening and flattening of the slopes. However the final slopes determined from the calculations and profile will not change the storm water runoff paths.

TABLE 1

Parameters Used in Settlement Calculations

Soil Unit	Density (pcf)	Compression Index (C_c) (dimensionless)	Void Ratio (e_o) (dimensionless)
Cap	135	N/A	N/A
Imported Fill	105	0.03	0.65
Existing Fill	105	0.03	0.6
Alluvium	118	0.1	0.71
Onshore Clay	117	0.3	1.3
Offshore Clay	99	0.4	1.5

Abbreviations:

pcf - pounds per cubic foot
psf - pounds per square foot
ft - foot

The following table is a summary of geotechnical testing results from the PFA that are relevant to the landfill settlement calculations and used to determine the parameters values of the settlement calculations.

TABLE 2

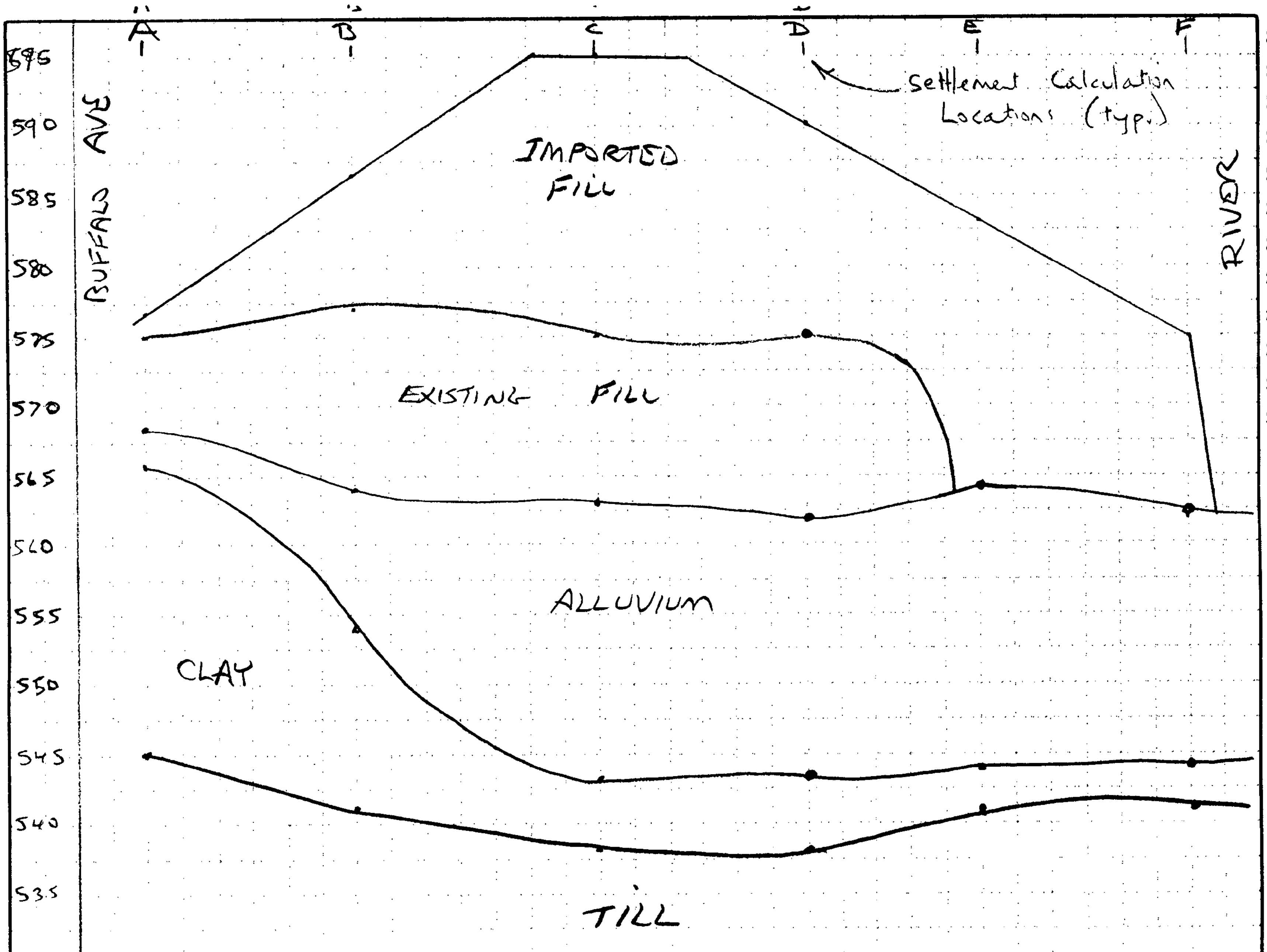
Geotechnical Laboratory Testing Results

BORING NUMBER	DEPTH	SOIL TYPE	WET DENSITY (pcf)	VOID RATIO	Cc
SW-03	4-6	FILL	120.4		
SW-07	10-12	CLAY	118.2		
SW-10	10-12	CLAY	115.4		
SW-22	4-6	ALLUVIUM	118.0	0.75	
SW-22	10-12	ALLUVIUM	116.1	0.65	
SW-37	4-6	ALLUVIUM	116.8		
SW-37	19-21	CLAY	105.3	1.32	0.30
SW-37	21-23	TILL	134.6	0.34	
SW-38	0-2	ALLUVIUM	114.3		
SW-38	12-14	TILL	139.9		
SW-38	10-12	TILL	139.5		
SW-39	0-2	ALLUVIUM	123.2	0.73	0.10
SW-28A	18.5-20.5	CLAY	92.1	1.7	0.53

102nd Street Landfill Site
Niagara Falls, N.Y.

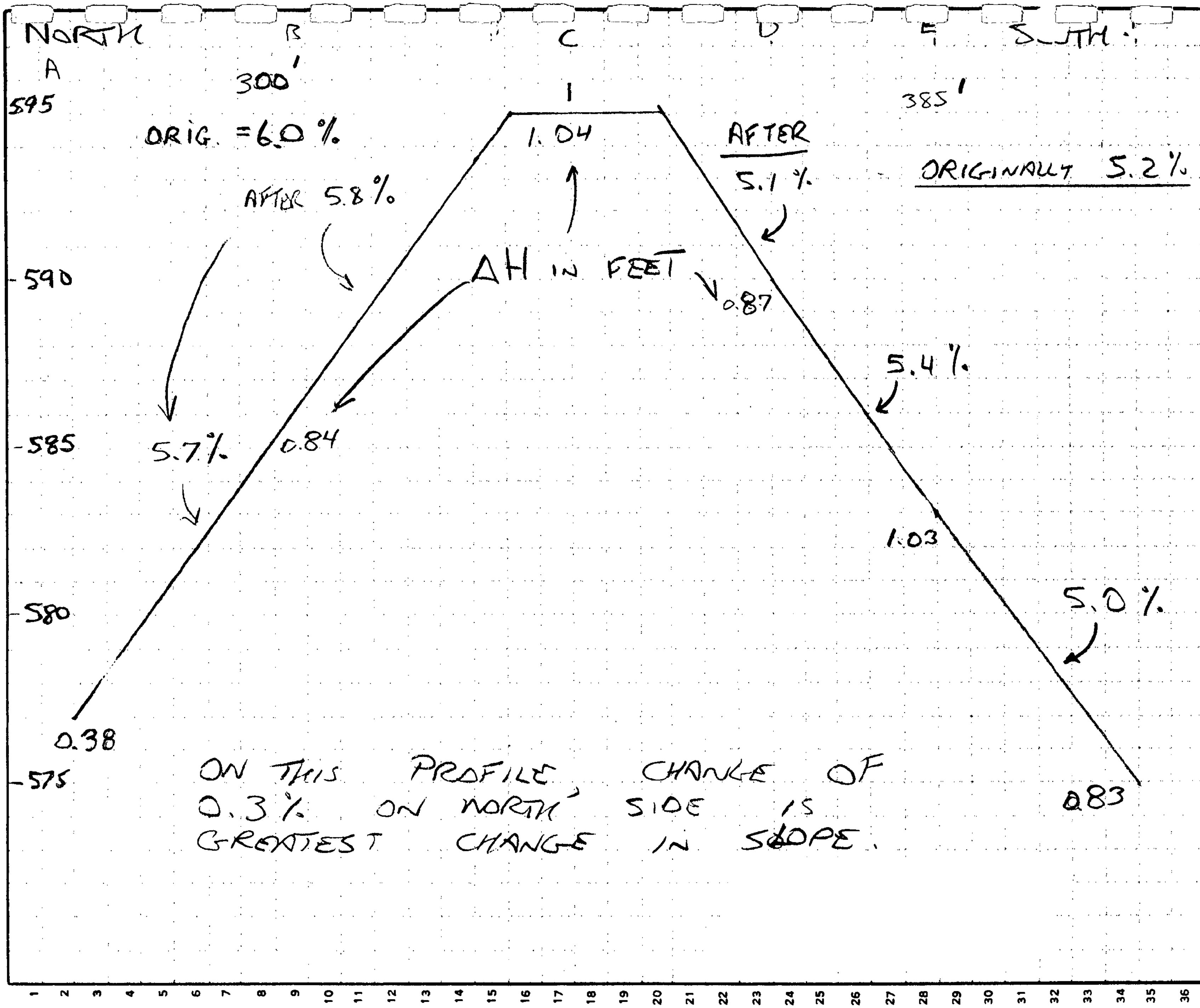
Landfill Settlement Calculations

		Location A	Location B	Location C	Location D	Location E	Location F
thickness of cap	(ft)	2	2	2	2	2	2
cap load	(psf)	270	270	270	270	270	270
top of imported fill	(ft)	575	585	593	588	581	573
thickness of import fill	(ft)	0	8	18	13	17	11
imported fill load	(psf)	0	840	1890	1365	1785	1155
top of existing fill	(ft)	575	577	575	575	564	562
thickness of existing fill	(ft)	7	13	12	13	0	0
existing fill load	(psf)	298.2	553.8	511.2	553.8	0	0
top of alluvium	(ft)	568	564	563	562	564	562
thickness of alluvium	(ft)	2	10	20	19	20	18
alluvium load	(psf)	111.2	556	1112	1056.4	1112	1000.8
top of onshore clay	(ft)	566	554	543	543	541	541
thickness of onshore clay	(ft)	21	13	5	5	0	0
onshore clay load	(psf)	1146.6	709.8	273	273	0	0
top of offshore clay	(ft)	545	541	538	538	544	544
thickness of offshore clay	(ft)	0	0	0	0	3	3
offshore clay load	(psf)	0	0	0	0	109.8	109.8
top of till	(ft)	545	541	538	538	541	541
Settlement – imported fill	(ft)	0.000	0.031	0.036	0.034	0.035	0.033
Settlement – existing fill	(ft)	0.059	0.171	0.219	0.205	0.000	0.000
Settlement – alluvium	(ft)	0.029	0.215	0.562	0.444	0.786	0.616
Settlement – onshore clay	(ft)	0.289	0.415	0.227	0.187	0.000	0.000
Settlement – offshore clay	(ft)	0.000	0.000	0.000	0.000	0.212	0.178
TOTAL SETTLEMENT	**(ft)**	**0.4**	**0.8**	**1.0**	**0.9**	**1.0**	**0.8**



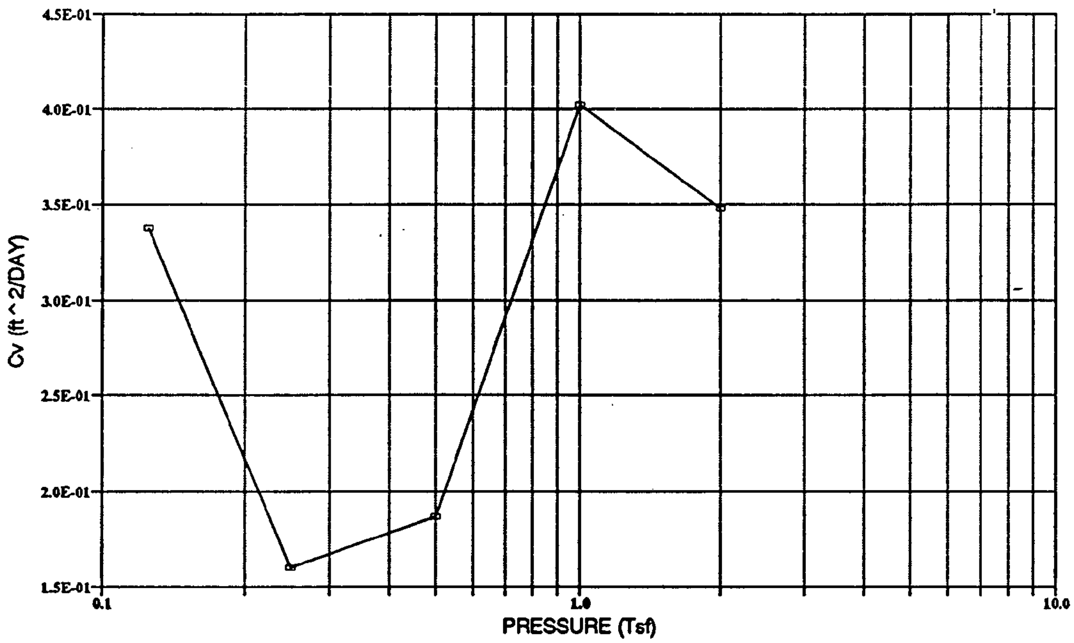
GEOLOGIC PROFILE - SETTLEMENT ANALYSIS

FORM E50 (Rev. 8-91)



FORM E50 (Rev. 8-91)

ONE-DIMENSIONAL CONSOLIDATION
ASTM D 2435



PRESSURE (tsf)	COEFFICIENT OF CONSOLIDATION	
	Cv(cm ^ 2/sec)	(ft ^2/day)
0.00	0.00E+00	0.00
0.05	1.49E-02	1.38
0.13	3.64E-03	0.34
0.25	1.72E-03	0.16
0.50	2.01E-03	0.19
1.00	4.33E-03	0.40
2.00	3.74E-03	0.35
4.00	4.16E-03	0.39
2.00	7.42E-03	0.69
1.00	2.80E-03	0.26
2.00	5.25E-03	0.49

PARAMETERS	
LIQUID LIMIT:	48
PLASTIC LIMIT:	23
PLASTICITY INDEX:	25
SPECIFIC GRAVITY:	2.74
USCS CLASSIFICATION:	CL
DESCRIPTION:	Reddish gray SILTY CLAY, moist, plastic

SAMPLE DEPTH:	18.5'-20.5'
DATE RECEIVED:	10/27/92
CONSOLIDOMETER No.:	#1

PROJECT NAME:	FLUOR DANIEL/LAB GEOTECH/NY
PROJECT NO.:	23594000-9-05K
SAMPLE NO.:	SW28A UD1

GOLDER ASSOCIATES INC.
MT. LAUREL, NJ

APPENDIX A.9

Pre-design Field Activities

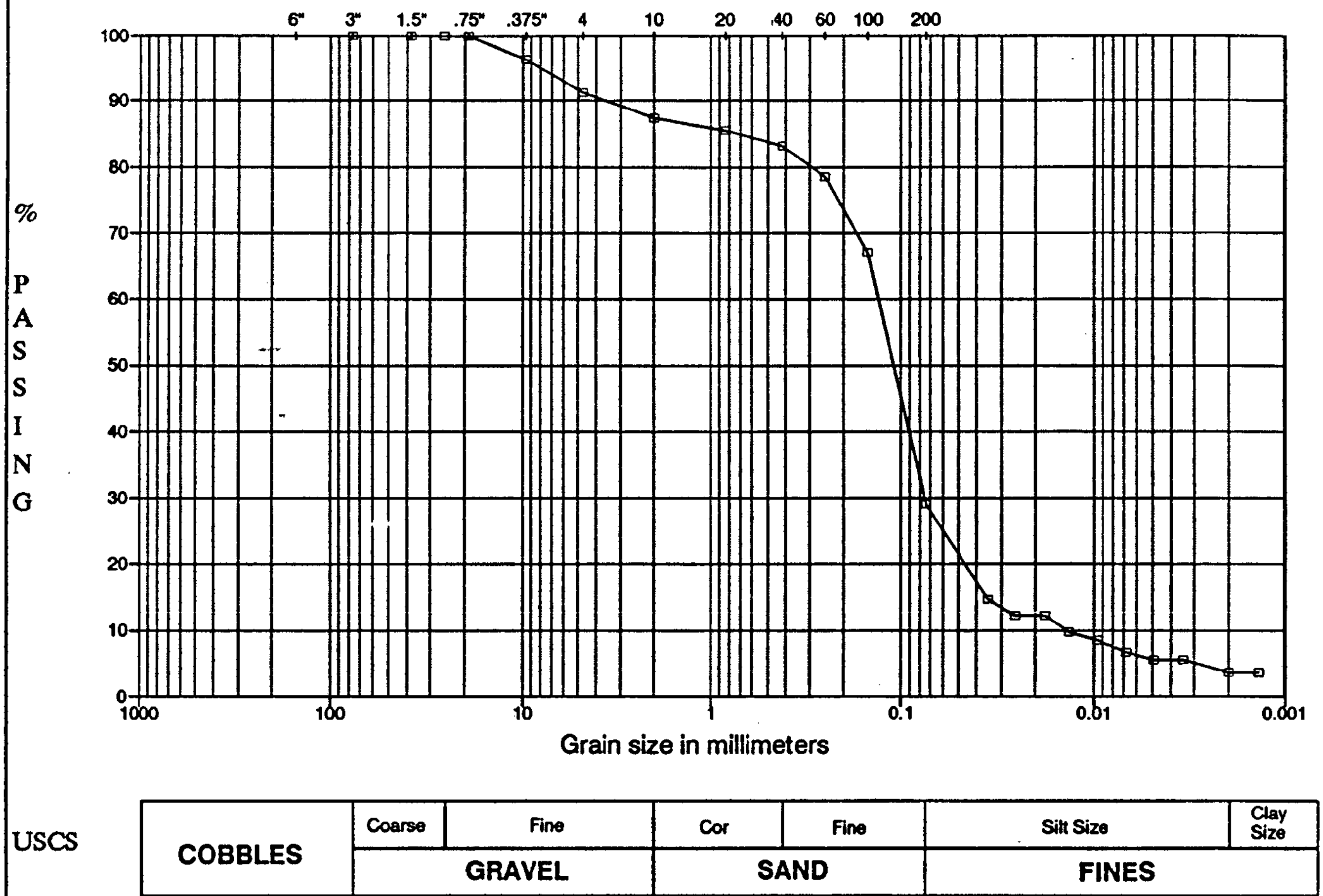
Additional Geotechnical Test Results

The following test results are from soil samples collected during the Predesign Field Activity (PFA). The tests contained herein were assigned by field geologists/geotechnical engineers as a result of conditions encountered at the Site. These test assignments were not part of the original set contained in the Geotechnical Sampling and Testing Plan (1992).

Boring	Depth (Feet)	Test
SW-19	4-6	Particle size distribution
SW-23	4-6	Particle size distribution
SW-23	10-12	Particle size distribution
SW-28	22-23.6	Particle size distribution
SW-23	10-12	CIUC
SW-23*	4-6	CIUC
SW-28	22-23.6	CIUC
SW-23	16-18	UU
SW-36	14-16	UU
SW-27	17-19	Consolidation
SW-28A	18.5-20.5	Consolidation

* Previously submitted in September 28, 1992 attachment to PFAR.

PARTICLE SIZE DISTRIBUTION ASTM D421 AND D422
US STANDARD SIEVE OPENING SIZES



TECH: TK
DATE: 12/7/92
CHECKED: DMW
REVIEWED: RH

SAMPLE ID	W%	LL	PL	PI	Gs	DESCRIPTION
SW-19 UD-1	21.5				2.58	Dark gray SITLY SAND moist, nonplastic
Sample Type: S.T.		Date Tested: 11/22/92				

FLUOR DANIEL/LAB GEOTECH/NY
23594000-9-05K

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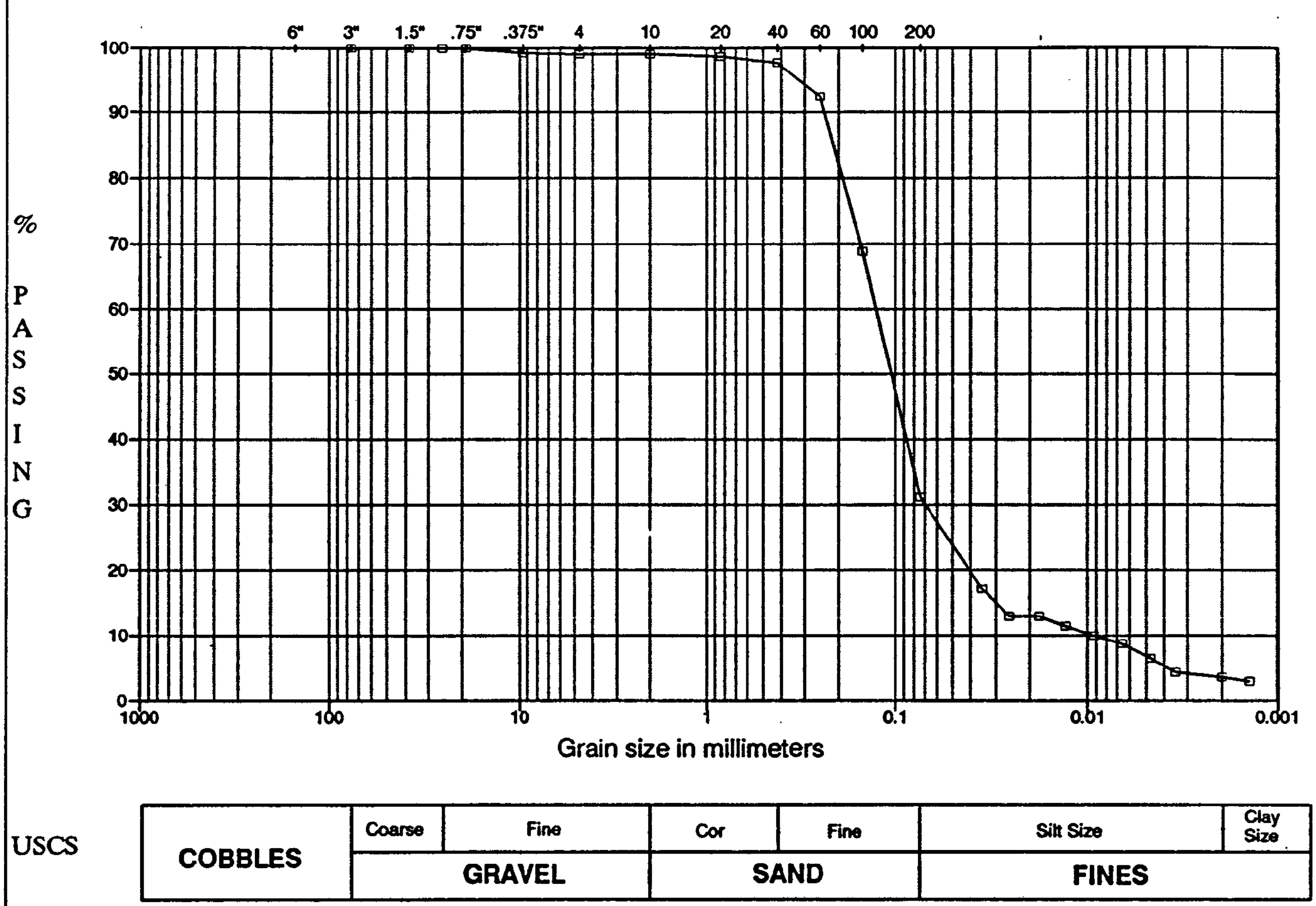
PARTICLE SIZE DISTRIBUTION ASTM D421 AND D422
US STANDARD SIEVE OPENING SIZES



TECH: CH
DATE: 12/7/92
CHECKED: DWW
REVIEWED: RJ

SAMPLE ID	W%	LL	PL	PI	Gs	DESCRIPTION
SW-23 UD-1 4' - 6'	25.3				2.54	Dark gray SILTY SAND moist, nonplastic
Sample Type: S.T.			Date Tested: 12/7/92			

FLUOR DANIEL/LAB GEOTECH/NY
23594000-9-05K

GOLDER ASSOCIATES INC.
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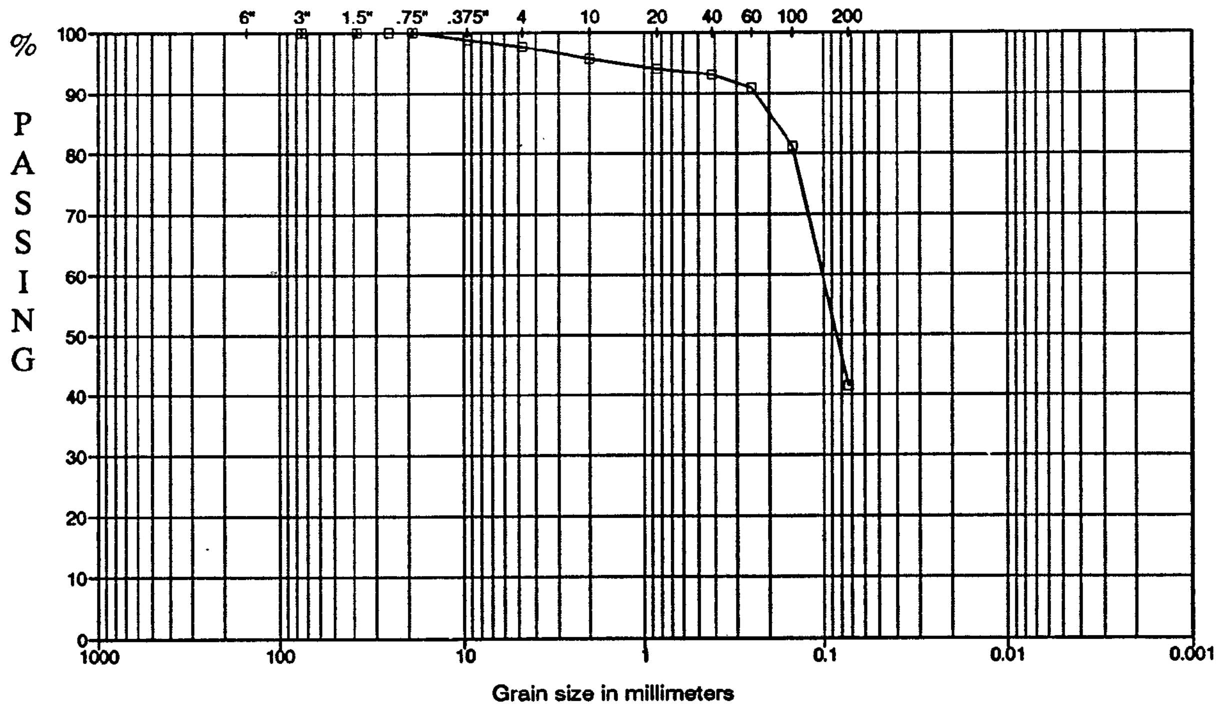
PARTICLE SIZE DISTRIBUTION ASTM D-421 AND 422

US STANDARD SIEVE OPENING SIZES



COBBLES	GRAVEL		SAND		FINES (Silt or Clay)
	Cor	Fine	Cor	Fine	

SAMPLE ID	W%	LL	PL	PI	Gs	DESCRIPTION
SW-23	20.7				2.51	Weak red
SS-1	DATE TESTED: 12/5/92					SILTY SAND
10'-12'						moist, nonplastic
TECHNICIAN: TK	DATE: 12/7/92				CHECKED: Pwm	REVIEWED: PJ

FLUOR DANIEL/LAB GEOTECH/NY
23594000-9-05K

GOLDER ASSOCIATES INC.
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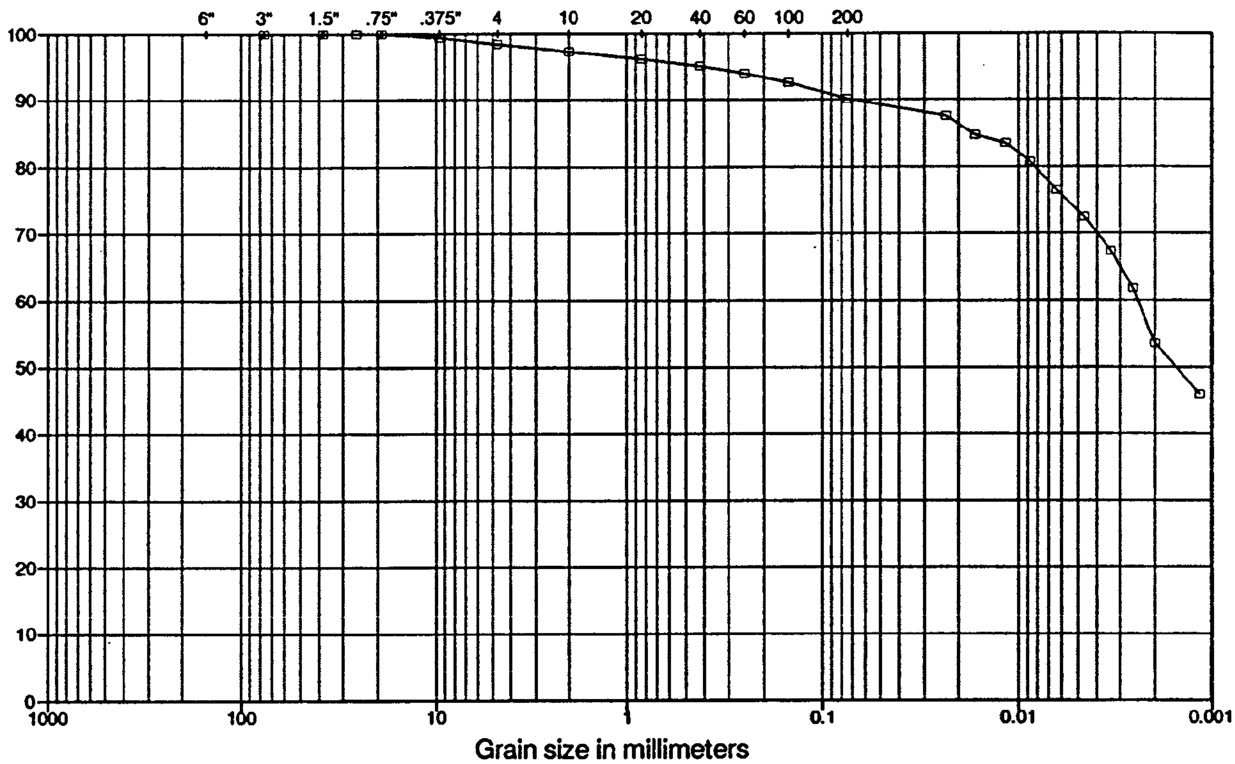
PARTICLE SIZE DISTRIBUTION ASTM D421 AND D422
US STANDARD SIEVE OPENING SIZES



USCS							
	COBBLES	Coarse	Fine	Cor	Fine	Silt Size	Clay Size
		GRAVEL		SAND		FINES	

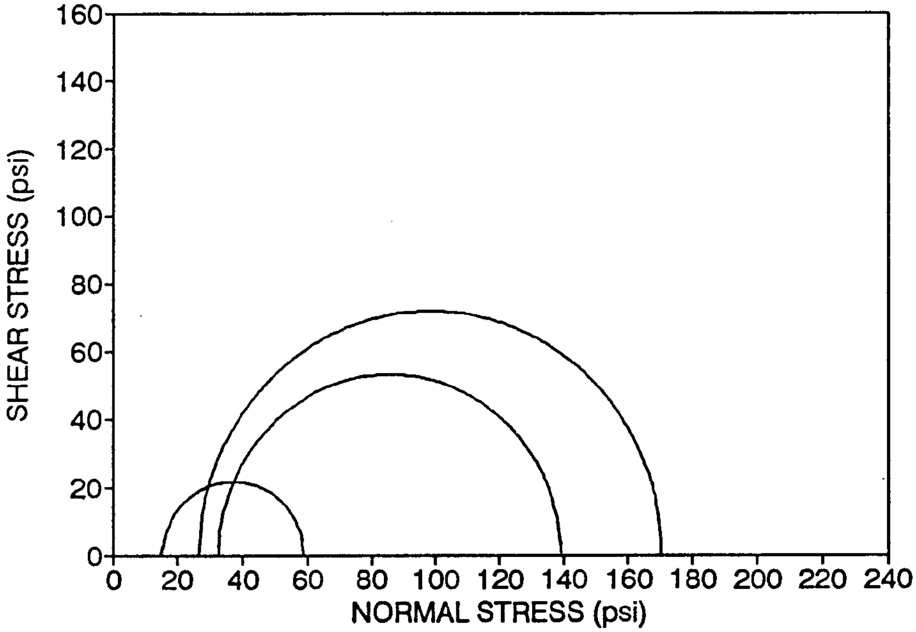
TECH: RDD/CH
DATE: 12/7/92
CHECKED:
REVIEWED:

SAMPLE ID	W%	LL	PL	PI	Gs	DESCRIPTION
SW-28 UD-2 22'- 23.6'	41.7				2.59	Reddish brown SILTY CLAY moist, plastic
Sample Type: S.T.		Date Tested: 12/7/92				

FLUOR DANIEL/LAB GEOTECH/NY
23594000-9-05K

GOLDER ASSOCIATES INC.
MT. LAUREL,NJ

MOHR STRESS CIRCLES
ASTM D4767



SAMPLE DESCRIPTION:

EFFECTIVE CONSOLIDATION PRESS (psi)	8.2	16.5	33
INITIAL MOIST DENSITY (pcf)	120.4	125.8	130.8
INITIAL WATER CONTENT, %	20.80%	22.44%	21.19%

SOIL PARAMETERS	
LL	
PL	
PI	
Gs	

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/N
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-23 SS-1

Split Spoon Sample

CHECKED
REVIEWED

GOLDER ASSOCIATES INC
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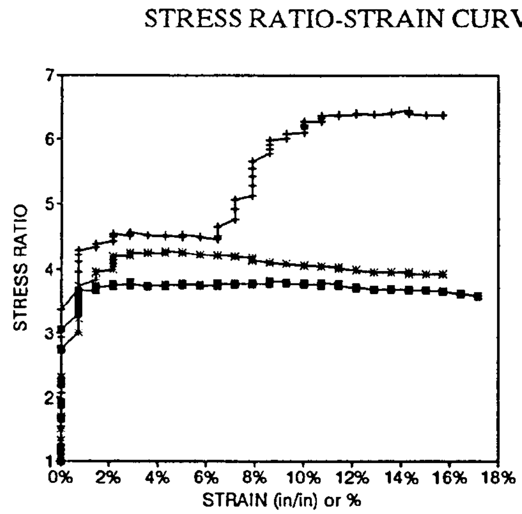
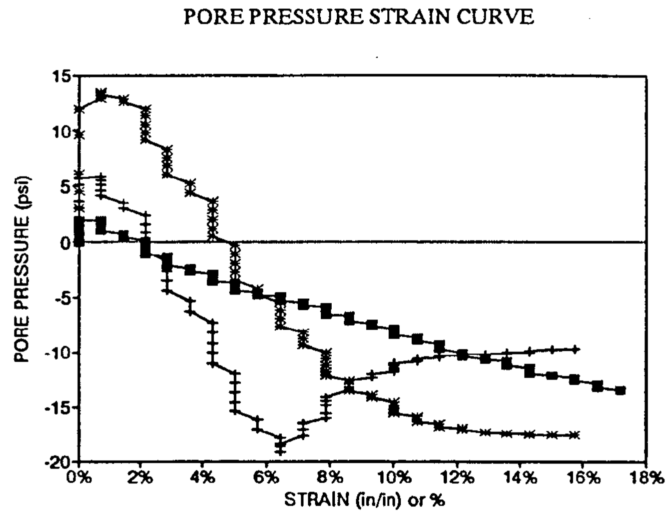
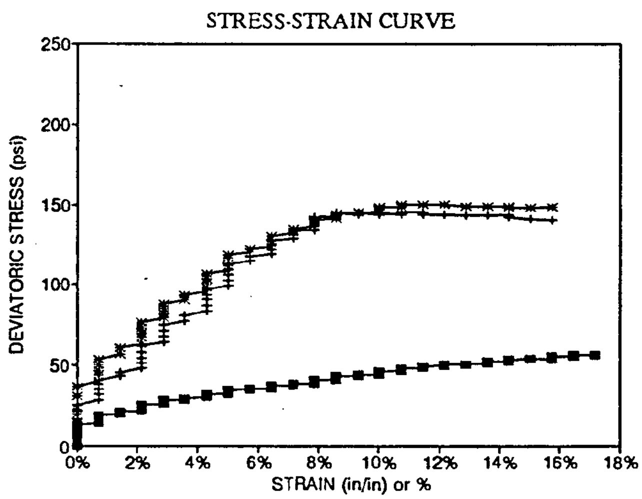
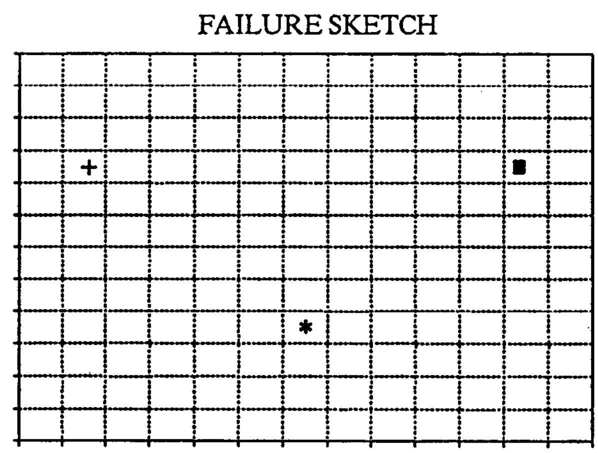
CONSOLIDATED UNDRAINED WITH PORE PRESSURE
ASTM D4767









SAMPLE DESCRIPTION:	
SOIL PARAMETERS	
LL	
PL	
PI	
Gs	

EFFECTIVE CONSOLIDATION PRESS (psi)	8	17	33
INITIAL MOIST DENSITY (pcf)	120.4	125.8	130.8
INITIAL WATER CONTENT (%)	20.80%	22.44%	21.19%
STRAIN RATE (%/min)	0.36	0.35	0.36

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/NY
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-23 SS-1

Split Spoon Sample

CHECKED: [illegible]
REVIEWED: [illegible]

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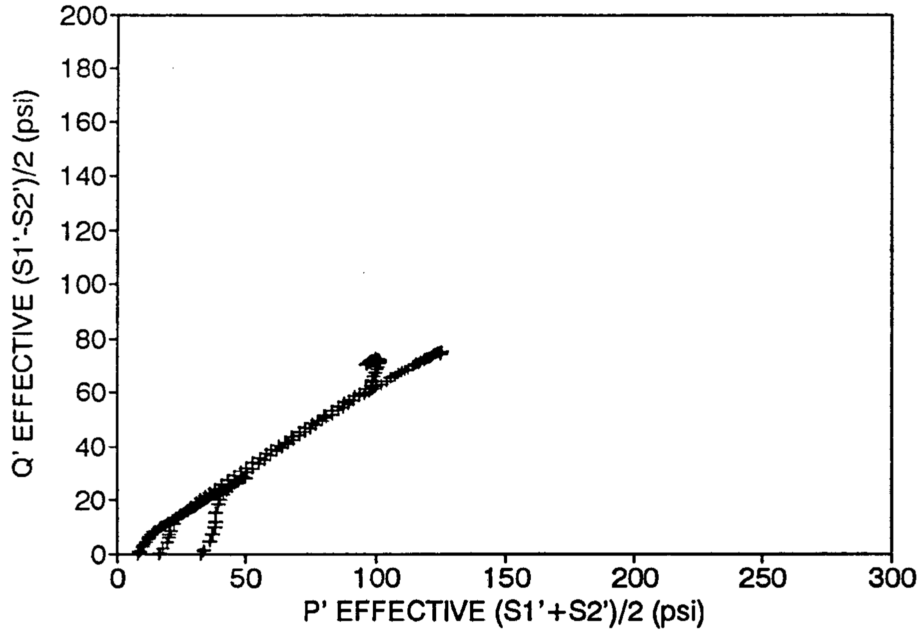
TRIAXIAL STRESS PATH
ASTM D4767



SAMPLE DESCRIPTION:

EFFECTIVE CONSOLIDATION PRESS (psi)	8.2	16.5	33
INITIAL MOIST DENSITY (pcf)	120.4	125.8	130.8
INITIAL WATER CONTENT,%	20.80%	22.44%	21.19%

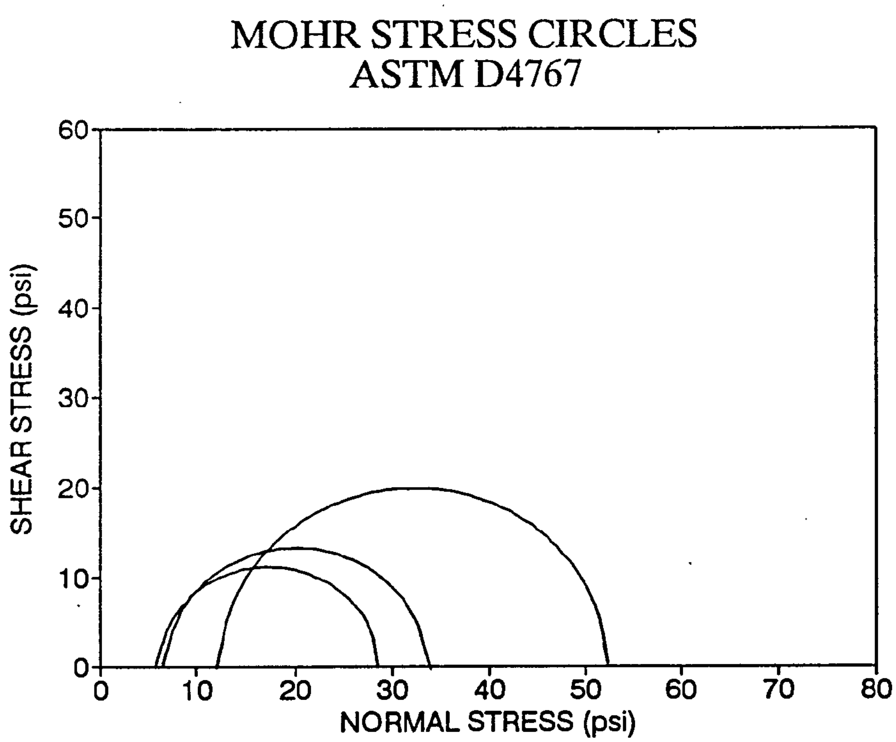
SOIL PARAMETERS	
LL	
PL	
PI	
Gs	

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/N
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-23 SS-1

Split Spoon Sample

CHECKED: MJM
REVIEWED: PJ

GOLDER ASSOCIATES INC
MT.LAUREL,NJ



SAMPLE DESCRIPTION:

EFFECTIVE CONSOLIDATION PRESS (psi)	6	12	24
INITIAL MOIST DENSITY (pcf)	117.4	118.2	118.4
INITIAL WATER CONTENT, %	25.55%	27.50%	28.20%

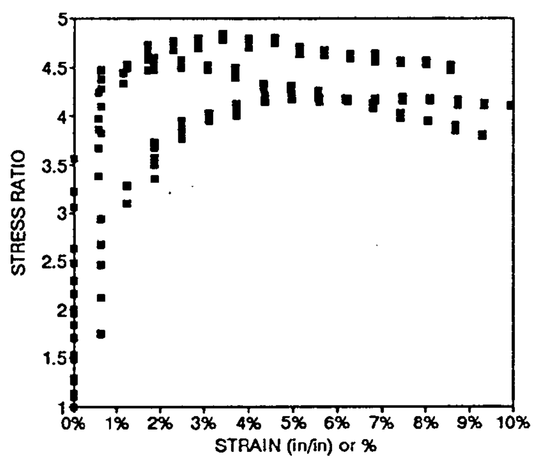
SOIL PARAMETERS	
LL	
PL	
PI	
Gs	

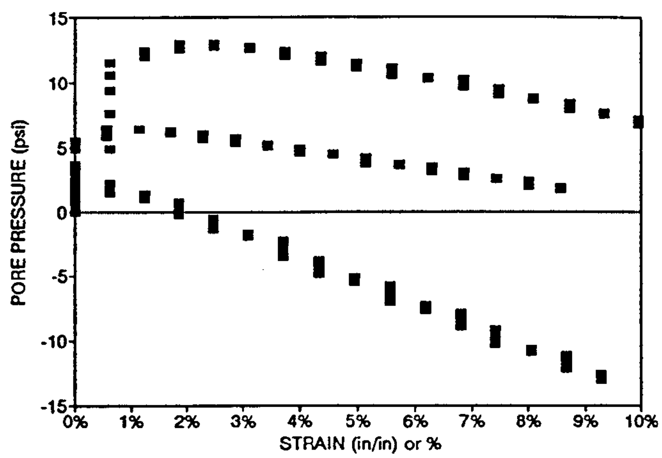
PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/N
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-23 UD-1

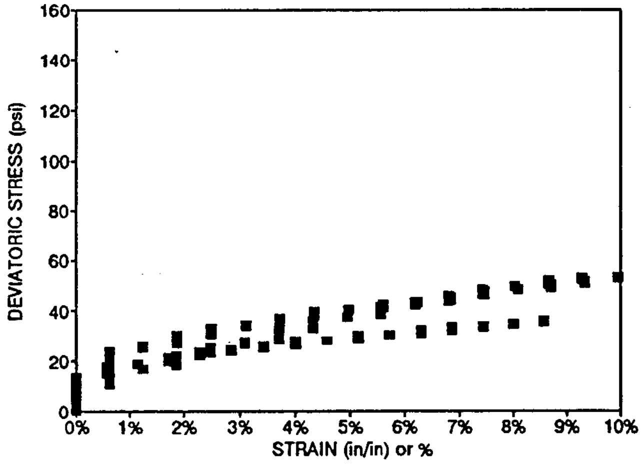
CHECKED
REVIEWED

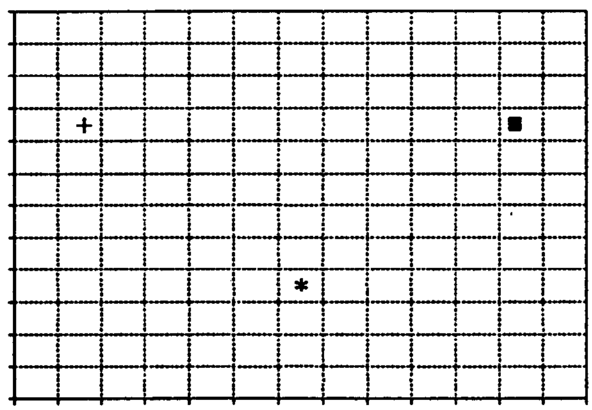
GOLDER ASSOCIATES INC
MT.LAUREL,NJ

CONSOLIDATED UNDRAINED WITH PORE PRESSURE
ASTM D4767









SAMPLE DESCRIPTION:	
SOIL PARAMETERS	
LL	
PL	
PI	
Gs	

EFFECTIVE CONSOLIDATION PRESS (psi)	6	12	24
INITIAL MOIST DENSITY (pcf)	117.4	118.2	118.4
INITIAL WATER CONTENT (%)	25.55%	27.50%	28.20%
STRAIN RATE (%/min)	0.31	0.29	0.31

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/NY
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-23 UD-1

CHECKED: MJM
REVIEWED: RJF

GOLDER ASSOCIATES INC.
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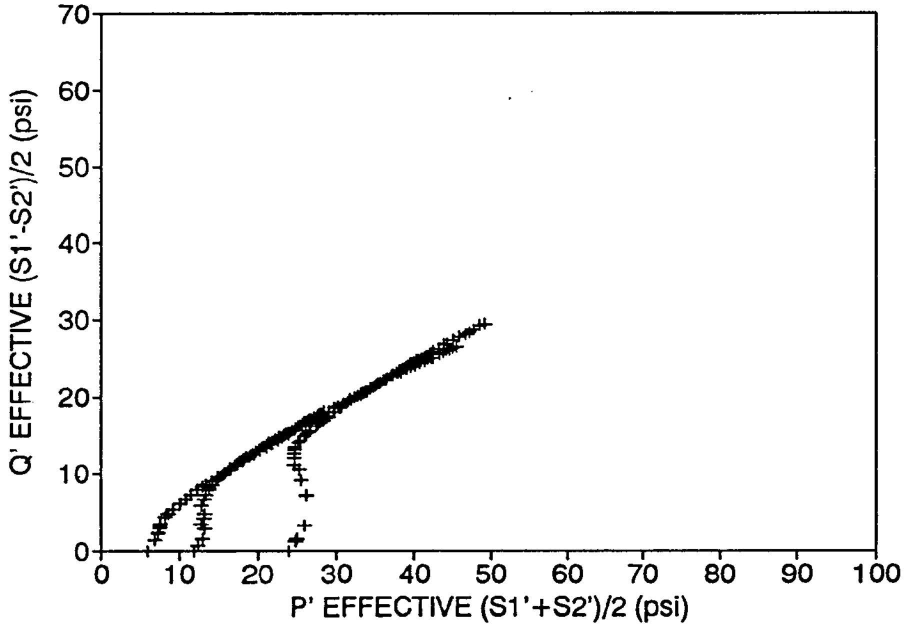
TRIAXIAL STRESS PATH
ASTM D4767



SAMPLE DESCRIPTION:

EFFECTIVE CONSOLIDATION PRESS (psi)	6	12	24
INITIAL MOIST DENSITY (pcf)	117.4	118.2	118.4
INITIAL WATER CONTENT,%	25.55%	27.50%	28.20%

SOIL PARAMETERS	
LL	
PL	
PI	
Gs	

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/N
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-23 UD-1

CHECKED: MJM
REVIEWED RJ

GOLDER ASSOCIATES INC
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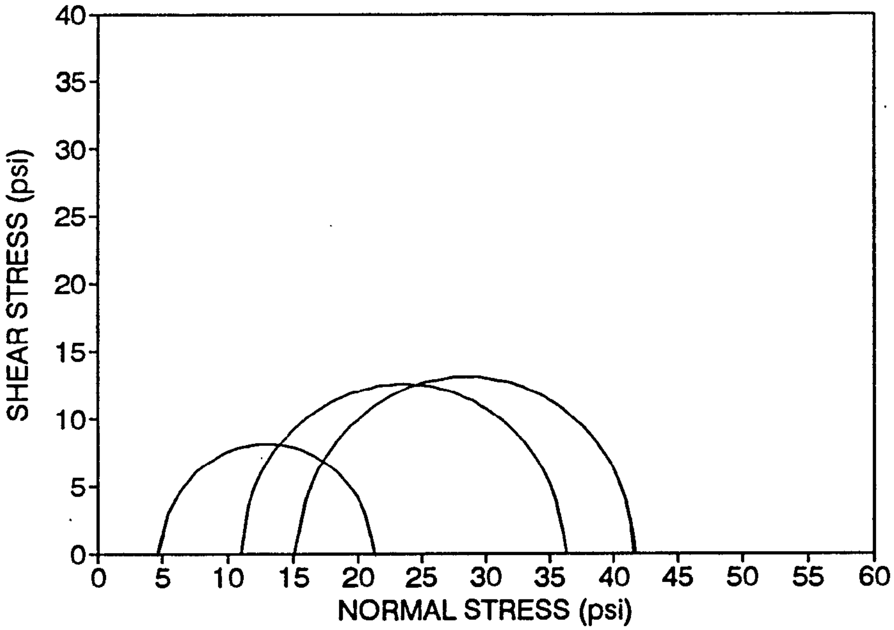
MOHR STRESS CIRCLES
ASTM D4767



SAMPLE DESCRIPTION:

EFFECTIVE CONSOLIDATION PRESS (psi)	12.2	25.5	51
INITIAL MOIST DENSITY (pcf)	119.1	119.9	114.7
INITIAL WATER CONTENT, %	22.88%	33.10%	27.70%

SOIL PARAMETERS	
LL	
PL	
PI	
Gs	

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/N
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-28 UD-2

CHECKED MM

REVIEWED PH

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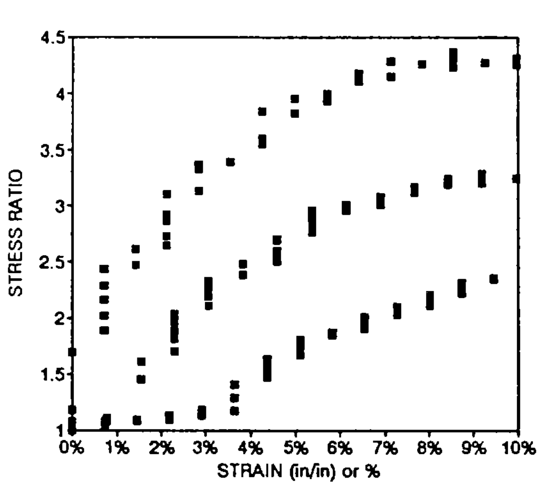
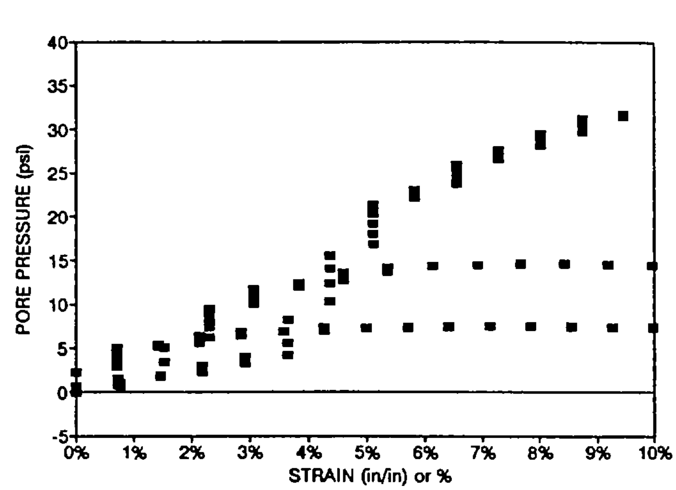
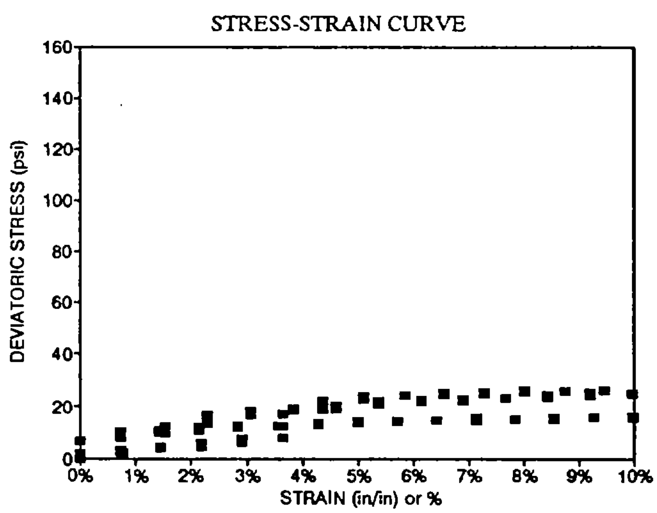
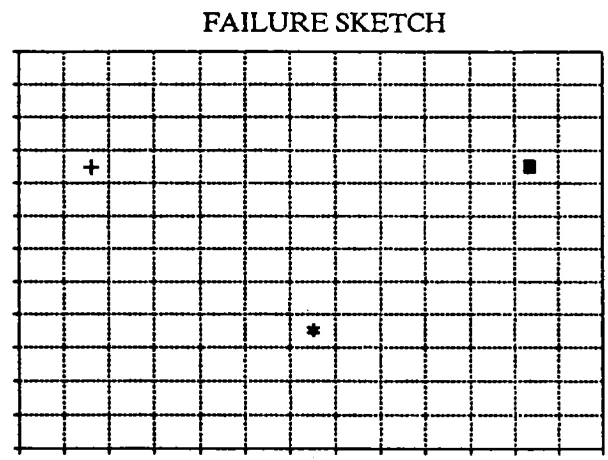
CONSOLIDATED UNDRAINED WITH PORE PRESSURE
ASTM D4767









SAMPLE DESCRIPTION:	
SOIL PARAMETERS	
LL	
PL	
PI	
Gs	

EFFECTIVE CONSOLIDATION PRESS (psi)	12	26	51
INITIAL MOIST DENSITY (pcf)	119.1	119.9	114.7
INITIAL WATER CONTENT (%)	22.88%	33.10%	27.70%
STRAIN RATE (%/min)	0.36	0.38	0.36

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/NY
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-28 UD-2

CHECKED:
REVIEWED:

GOLDER ASSOCIATES INC.
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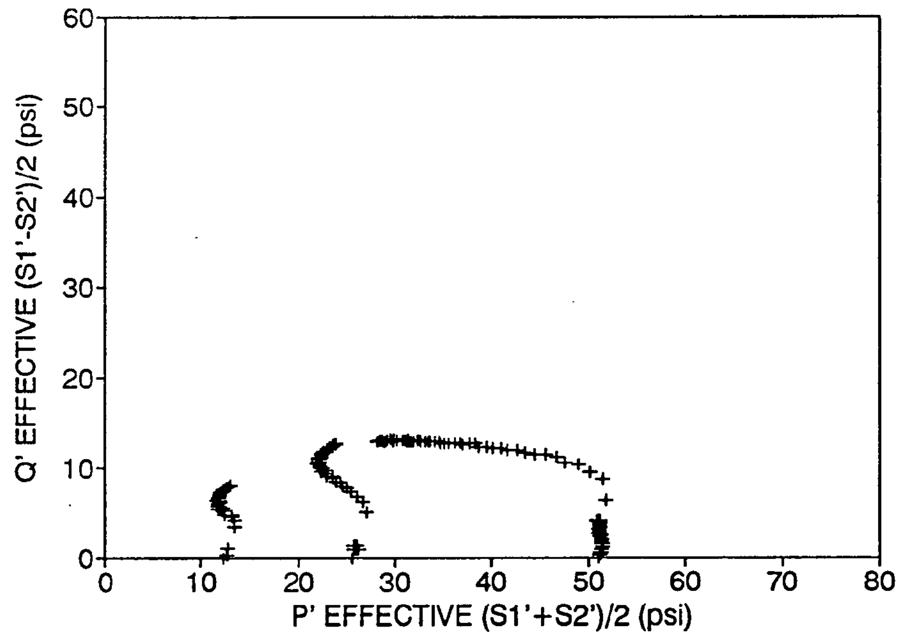
TRIAXIAL STRESS PATH
ASTM D4767



SAMPLE DESCRIPTION:

EFFECTIVE CONSOLIDATION PRESS (psi)	12.2	25.5	51
INITIAL MOIST DENSITY (pcf)	119.1	119.9	114.7
INITIAL WATER CONTENT,%	22.88%	33.10%	27.70%

SOIL PARAMETERS	
LL	
PL	
PI	
Gs	

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/N
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-28 UD-2

CHECKED:
REVIEWED

GOLDER ASSOCIATES INC
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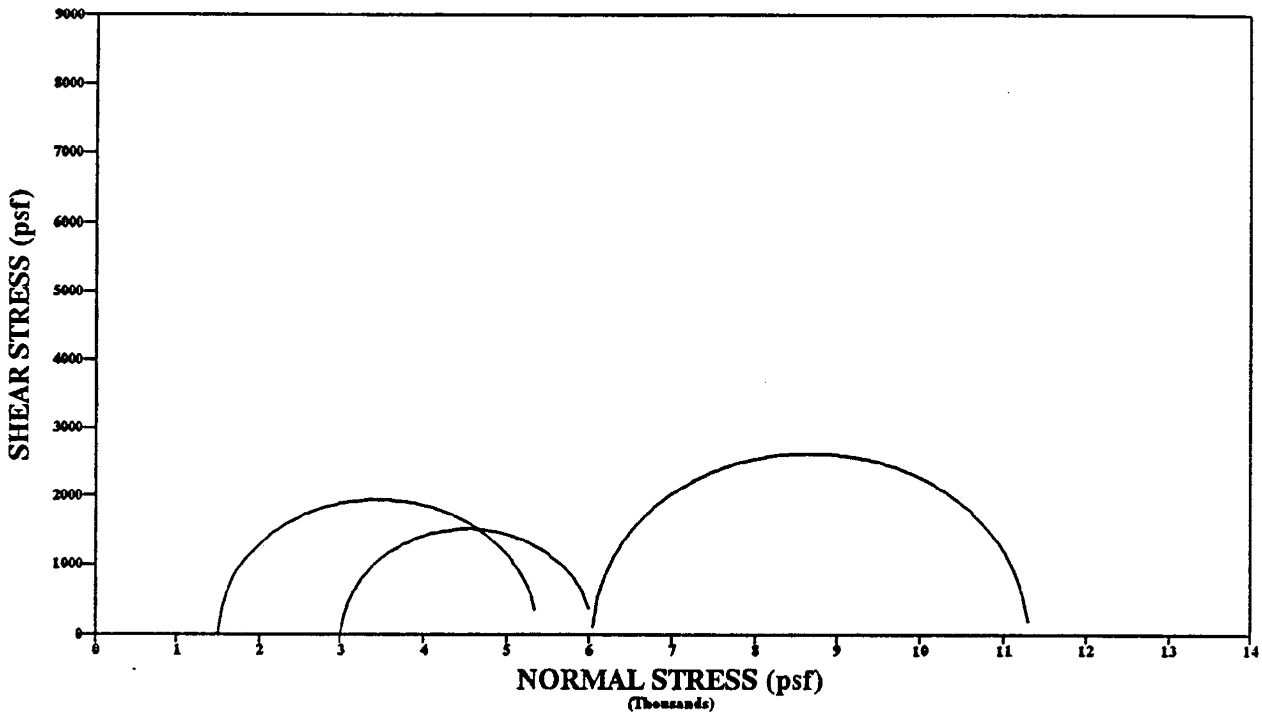
MOHR STRESS CIRCLES



SAMPLE DESCRIPTION:

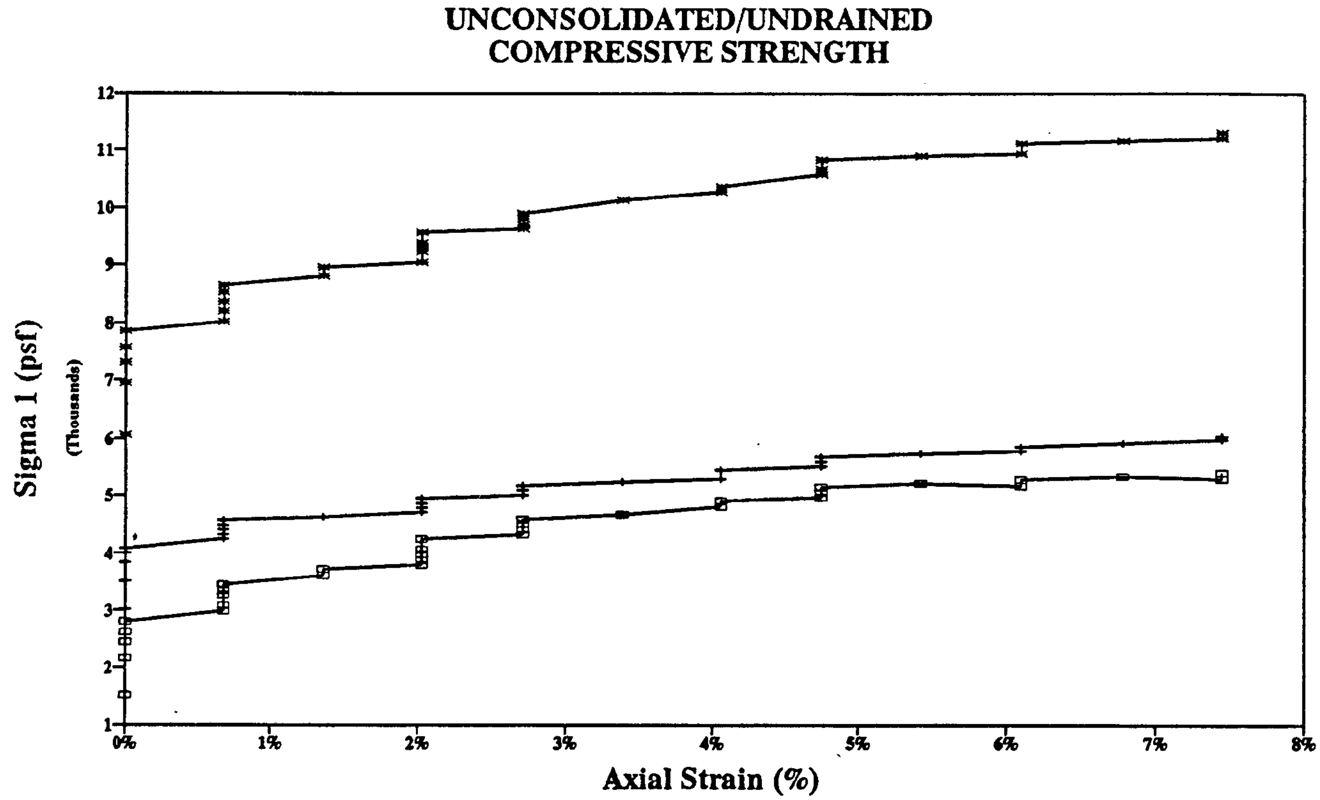
* TOTAL STRENGTH PARAMETERS	EFFECTIVE CONSOLIDATION PRESS (psi	11	21	42
0 =	INITIAL MOIST DENSITY (pcf)	150.65	146.40	143.15
c =	INITIAL WATER CONTENT, %	11.10%	11.40%	11.96%

		SOIL PARAMETERS	
PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/NY	LL	NP
PROJECT NUMBER:	23594000-9-05K	PL	NP
SAMPLE ID:	SW-23 UD-2 16'-18'	PI	NP
DELIVERED MOISTURE	11.40	Gs	

NP=NOT PERFORMED CHECKED

REVIEWED

GOLDER ASSOCIATES INC
MT.LAUREL,NJ



SAMPLE DESCRIPTION:	
SOIL PARAMETERS	
LL	NP
PL	NP
PI	NP
Gs	

NP= NOT PERFORMED

EFFECTIVE CONSOLIDATION PRESS (psi)	11	21	42
INITIAL MOIST DENSITY (pcf)	150.7	146.4	143.1
INITIAL WATER CONTENT (%)	11.10%	11.40%	11.96%
STRAIN RATE (%/min)	0.34	0.33	0.33

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-23 UD-2 16'-18'

CHECKED: 11/3/97
REVIEWED:

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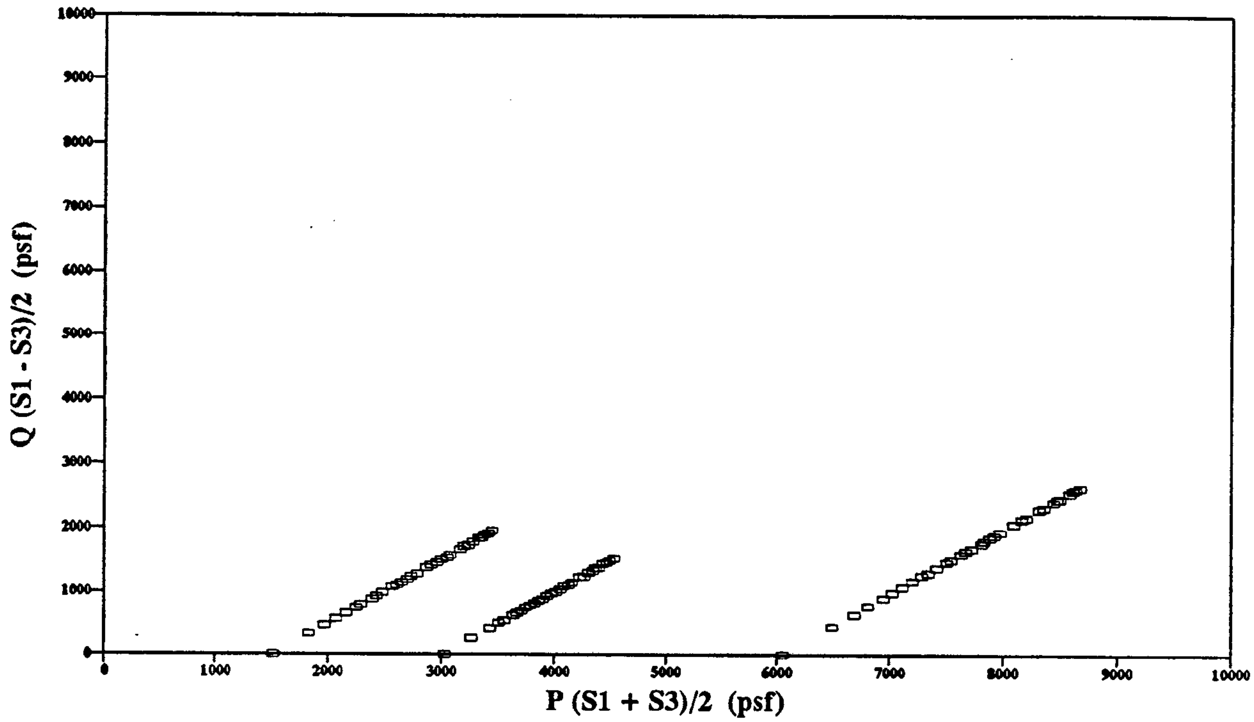
TRIAXIAL STRESS PATH



SAMPLE DESCRIPTION:

P,Q TOTAL STRESS PARAMETERS	
alpha =	
a =	

EFFECTIVE CONSOLIDATION PRESS (ps	11	21	42
INITIAL MOIST DENSITY (pcf)	150.65	146.40	143.15
INITIAL WATER CONTENT,%	11.10%	11.40%	11.96%

SOIL PARAMETERS	
LL	NP
PL	NP
PI	NP
Gs	

NP=NOT PERFORMED

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/N
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-23 UD-2 16'-18'

CHECKED: 7/27/[illegible]
REVIEWED [illegible]

GOLDER ASSOCIATES INC
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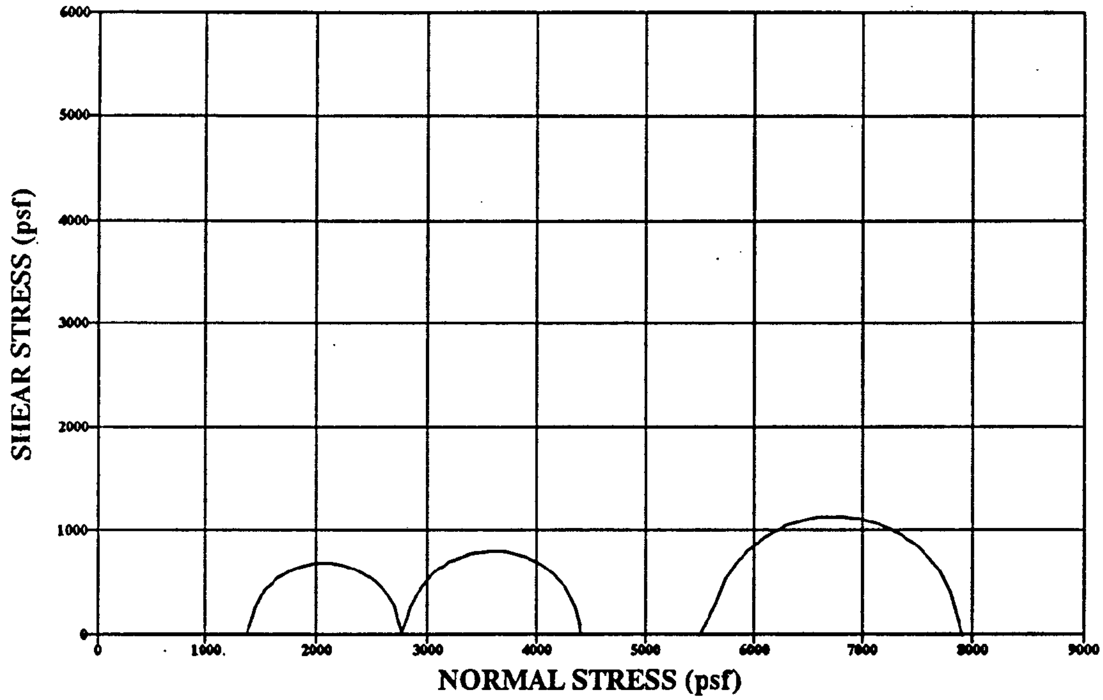
MOHR STRESS CIRCLES



SAMPLE DESCRIPTION: Reddish brown Silty Sand, Gravel, moist, low plastic

EFFECTIVE CONSOLIDATION PRESS (psi	10	20	39
INITIAL MOIST DENSITY (pcf)	158.17	151.09	157.62
INITIAL WATER CONTENT, %	11.97%	12.10%	11.23%

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/NY
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-36 UD-3 14'-16'
DELIVERED MOISTURE	10.30

SOIL PARAMETERS	
LL	NP
PL	NP
PI	NP
Gs	

NP=NOT PERFORMED CHECKED [signature]
REVIEWED [signature]

GOLDER ASSOCIATES INC
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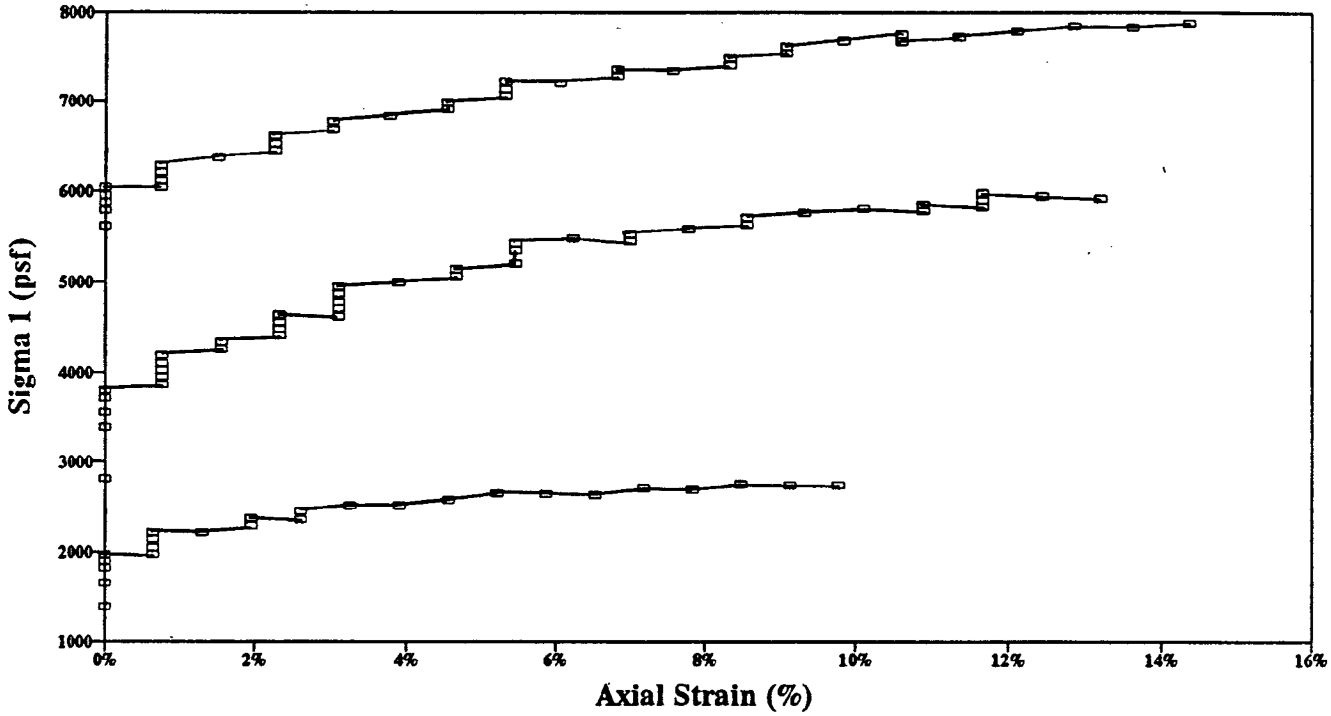
UNCONSOLIDATED/UNDRAINED COMPRESSIVE STRENGTH



SAMPLE DESCRIPTION: Reddish brown Silty Sand, Gravel, moist, low plastic	
SOIL PARAMETERS	
LL	NP
PL	NP
PI	NP
Gs	

NP = NOT PERFORMED

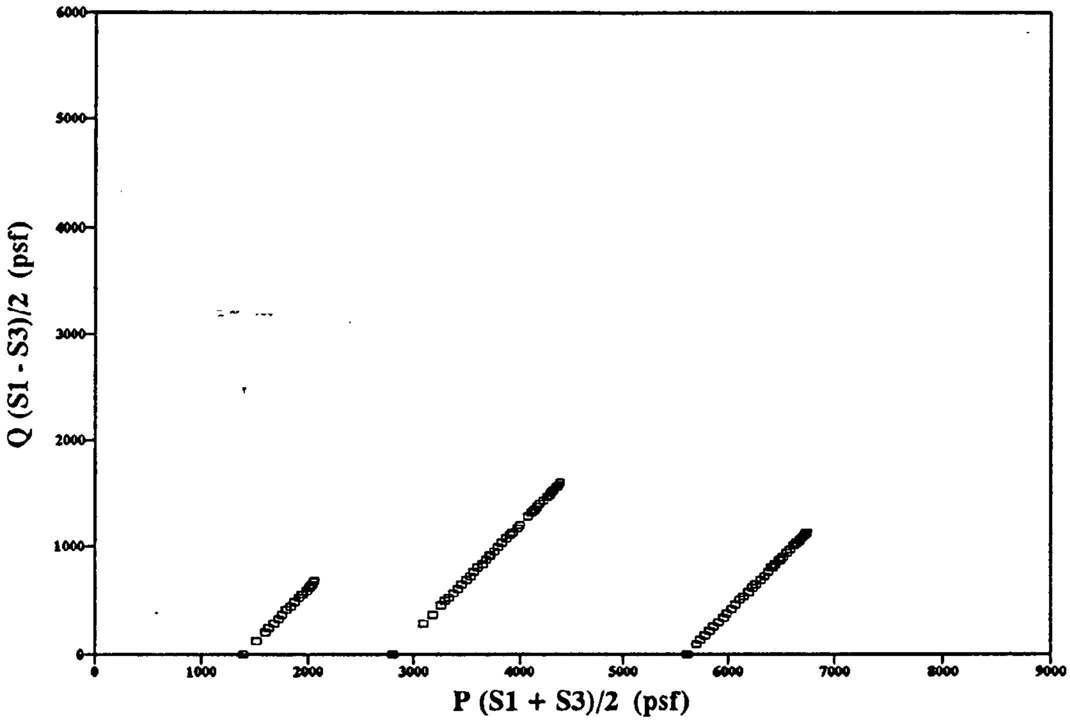
EFFECTIVE CONSOLIDATION PRESS (psi)	10	20	39
INITIAL MOIST DENSITY (pcf)	158.2	151.1	157.6
INITIAL WATER CONTENT (%)	11.97%	12.10%	11.23%
STRAIN RATE (%/min)	0.33	0.39	0.38

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-36 UD-3 14'-16'

CHECKED: [signature]
REVIEWED: [signature]

GOLDER ASSOCIATES INC
MT.LAUREL,NJ

TRIAXIAL STRESS PATH



SAMPLE DESCRIPTION: Reddish brown Silty Sand, Gravel, moist, low plastic

EFFECTIVE CONSOLIDATION PRESS (ps	10	20	39
INITIAL MOIST DENSITY (pcf)	158.17	151.09	157.62
INITIAL WATER CONTENT,%	11.97%	12.10%	11.23%

SOIL PARAMETERS	
LL	NP
PL	NP
PI	NP
Gs	

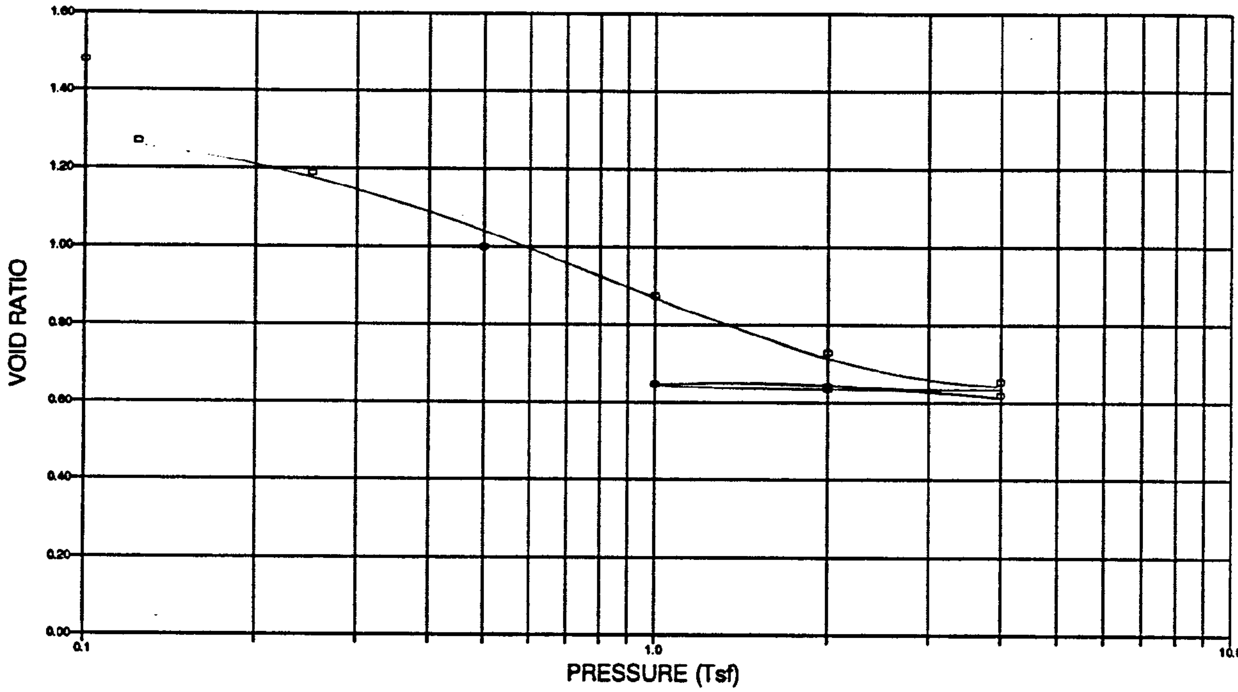
NP=NOT PERFORMED

PROJECT TITLE:	FLUOR DANIEL/LAB GEOTECH/N
PROJECT NUMBER:	23594000-9-05K
SAMPLE ID:	SW-36 UD-3 14'-16'

CHECKED:
REVIEWED

GOLDER ASSOCIATES INC
MT.LAUREL,NJ

ONE-DIMENSIONAL CONSOLIDATION
ASTM D 2435



			INITIAL	FINAL	PARAMETERS	
Diameter (in)	2.50		INITIAL	FINAL		
Height of specimen (in)	0.81	Total Heights(in)	0.81	0.53	LIQUID LIMIT:	48
Area of specimen (in ^ 2)	4.92	Height of solids(in)	0.327	0.327	PLASTIC LIMIT:	23
Volume of specimen (in ^ 3)	3.99	Height of voids(in)	0.483	0.203	PLASTICITY INDEX:	25
Water Content (Avg) from Trimmings	38.34%	Height of water(in)	0.356	0.221	SPECIFIC GRAVITY:	2.84
Specimen Wt (wet,g)	103.54	Void ratio	1.479	0.621	USCS CLASSIFICATION:	CL
Specimen Wt (dry,g)	74.84	Degree of saturation	73.61%	109.12%	DESCRIPTION:	Reddish gray
Water Wt (g)	28.70	Dry unit weight (pcf)	71.48	109.32		SILTY CLAY,
Unit weight of water (pcf)	62.40	Wet unit weight (pcf)	98.88	135.41		moist, plastic

PROJECT NAME:	FLUOR DANIEL/LAB GEOTECH/NY	SAMPLE DEPTH:	17'-19'
PROJECT NO.:	23594000-9-05K	DATE RECEIVED:	10/27/92
SAMPLE NO.:	SW27 UD2	CONSOLIDOMETER No.:	#1

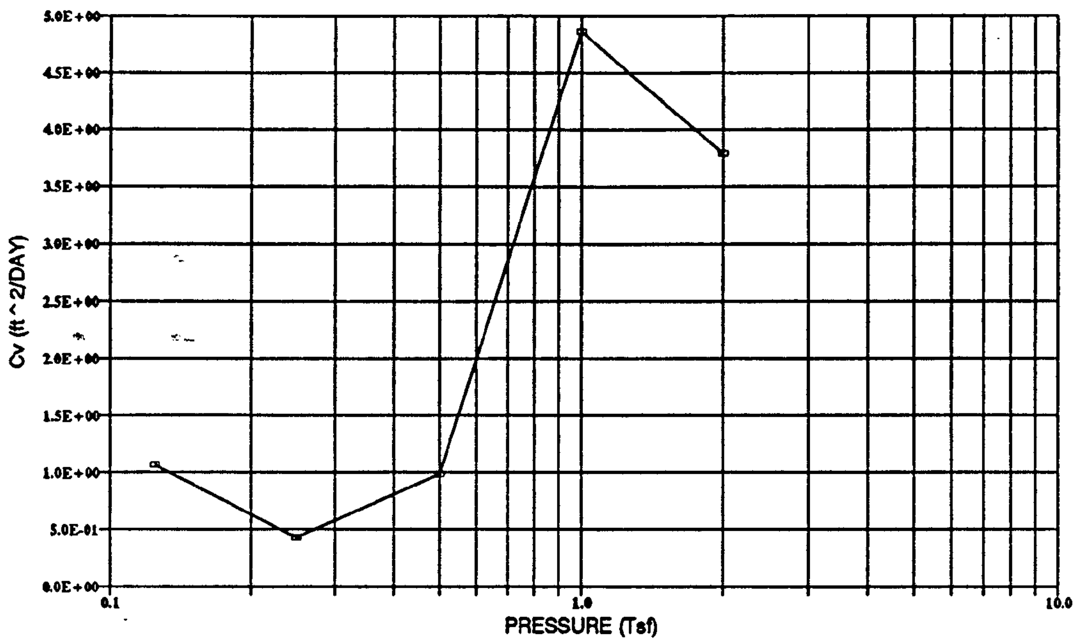
NOTES: The sample description is based upon NYDOT standard description.
Initial and final saturation values were backcalculated based upon assumed value of specific gravity.

GOLDER ASSOCIATES INC.
MT. LAUREL, NJ

Disclaimer:

e-log p curve reflects apparent disturbance of the sample, as noted on the sample extrusion sheet attached.

ONE-DIMENSIONAL CONSOLIDATION
ASTM D 2435

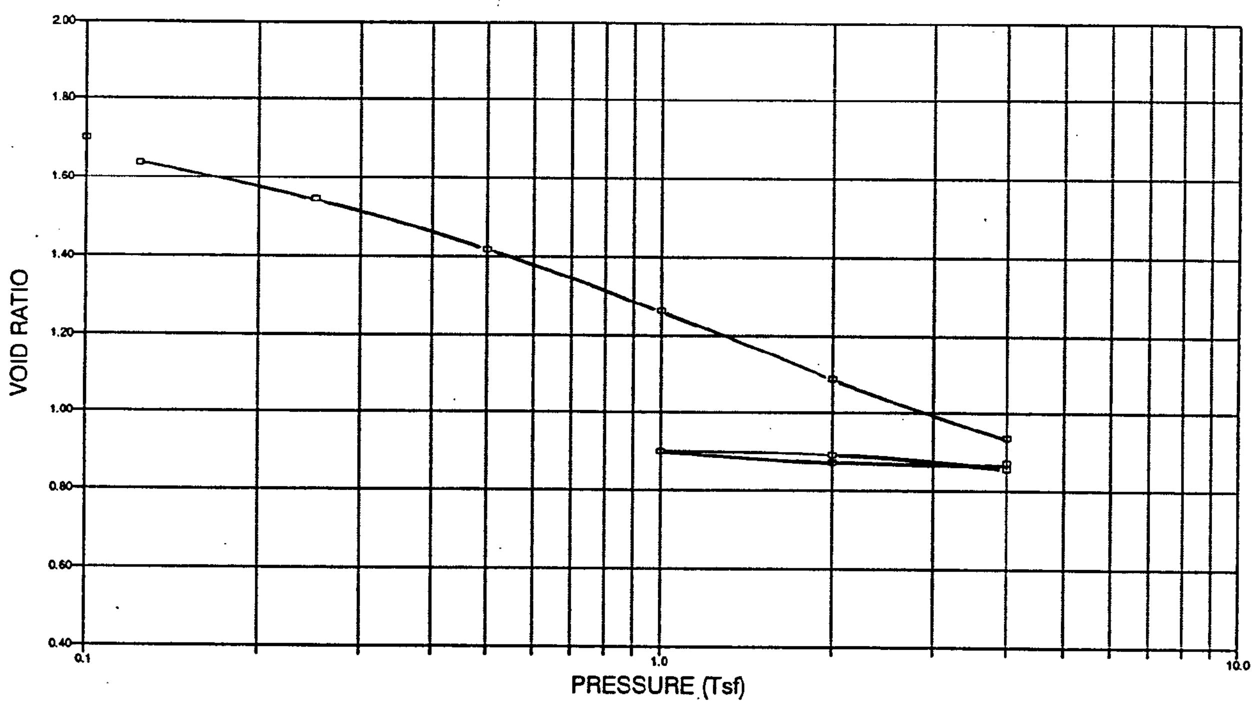


PRESSURE (tsf)	COEFFICIENT OF CONSOLIDATION	
	Cv(cm ^ 2/sec)	(ft ^ 2/day)
0.00	0.00E+00	0.00
0.05	4.75E-03	0.44
0.13	1.14E-02	1.06
0.25	4.56E-03	0.42
0.50	1.05E-02	0.97
1.00	5.23E-02	4.86
2.00	4.07E-02	3.79
4.00	7.56E-02	7.02
2.00	3.44E-02	3.20
1.00	6.68E-02	6.21
2.00	6.77E-02	6.29

PARAMETERS	
LIQUID LIMIT:	48
PLASTIC LIMIT:	23
PLASTICITY INDEX:	25
SPECIFIC GRAVITY:	2.84
USCS CLASSIFICATION:	CL
DESCRIPTION:	Reddish gray SILTY CLAY, moist, plastic
SAMPLE DEPTH:	17-19'
DATE RECEIVED:	10/27/92
CONSOLIDOMETER No.:	#1
PROJECT NAME:	FLUOR DANIEL/LAB GEOTECH/NY
PROJECT NO.:	23594000-9-05K
SAMPLE NO.:	SW27 UD2

GOLDER ASSOCIATES INC.
MT. LAUREL, NJ

ONE-DIMENSIONAL CONSOLIDATION
ASTM D 2435

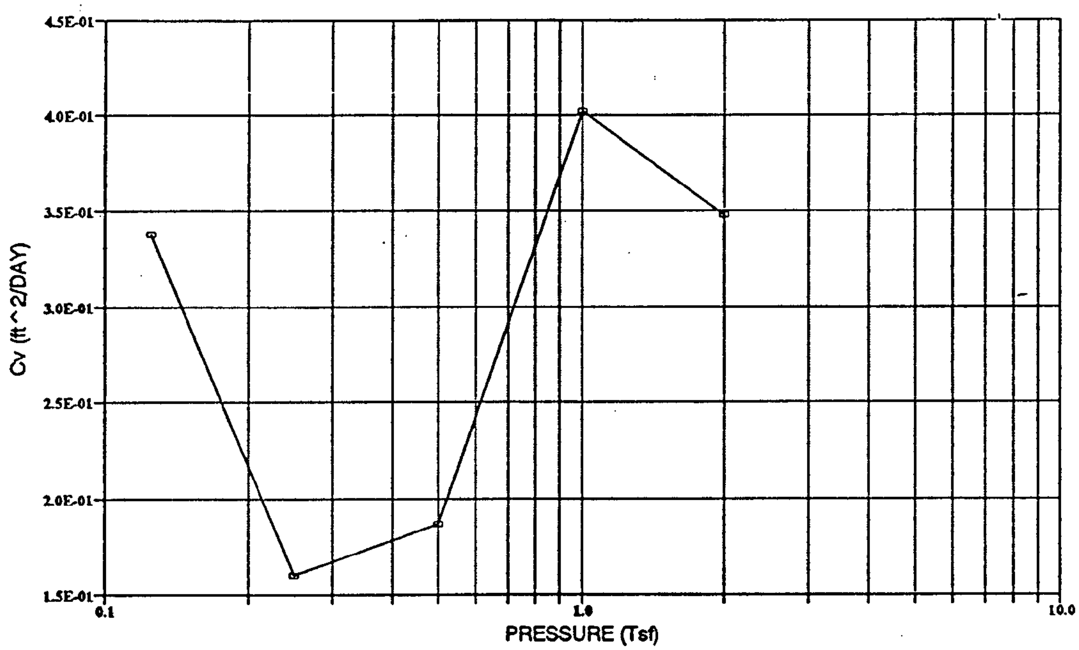


			INITIAL	FINAL	PARAMETERS	
Diameter (in)	2.50					
Height of specimen (in)	0.81	Total Heights(in)	0.81	0.55	LIQUID LIMIT:	48
Area of specimen (in^2)	4.92	Height of solids(in)	0.300	0.300	PLASTIC LIMIT:	23
Volume of specimen (in^3)	3.99	Height of voids(in)	0.510	0.253	PLASTICITY INDEX:	25
Water Content (Avg) from Trimmings	45.49%	Height of water(in)	0.374	0.246	SPECIFIC GRAVITY:	2.74
Specimen Wt (wet,g)	96.45	Void ratio	1.700	0.843	USCS CLASSIFICATION:	CL
Specimen Wt (dry,g)	66.29	Degree of saturation	73.33%	97.35%	DESCRIPTION:	Reddish gray
Water Wt (g)	30.16	Dry unit weight (pcf)	63.33	92.76		SILTY CLAY,
Unit weight of water (pcf)	62.40	Wet unit weight (pcf)	92.14	120.55		moist, plastic

PROJECT NAME:	FLUOR DANIEL/LAB GEOTECH/NY	SAMPLE DEPTH:	18.5'-20.5'
PROJECT NO.:	23594000-9-05K	DATE RECEIVED:	10/27/92
SAMPLE NO.:	SW28A UD1	CONSOLIDOMETER No.:	#1

GOLDER ASSOCIATES INC.
MT. LAUREL, NJ

ONE-DIMENSIONAL CONSOLIDATION
ASTM D 2435



PRESSURE (tsf)	COEFFICIENT OF CONSOLIDATION	
	Cv(cm^2/sec)	(ft^2/day)
0.00	0.00E+00	0.00
0.05	1.49E-02	1.38
0.13	3.64E-03	0.34
0.25	1.72E-03	0.16
0.50	2.01E-03	0.19
1.00	4.33E-03	0.40
2.00	3.74E-03	0.35
4.00	4.16E-03	0.39
2.00	7.42E-03	0.69
1.00	2.80E-03	0.26
2.00	5.25E-03	0.49

PARAMETERS	
LIQUID LIMIT:	48
PLASTIC LIMIT:	23
PLASTICITY INDEX:	25
SPECIFIC GRAVITY:	2.74
USCS CLASSIFICATION:	CL
DESCRIPTION:	Reddish gray SILTY CLAY, moist, plastic
SAMPLE DEPTH:	18.5'-20.5'
DATE RECEIVED:	10/27/92
CONSOLIDOMETER No.:	#1
PROJECT NAME:	FLUOR DANIEL/LAB GEOTECH/NY
PROJECT NO.:	23594000-9-05K
SAMPLE NO.:	SW28A UD1

GOLDER ASSOCIATES INC.
MT. LAUREL, NJ

APPENDIX A.10

NAPL Storage Tank Emissions

NAPL STORAGE TANK - EMMISSION CALCULATION
NAPL as CHLOROBENZENE

Location	Niagara Falls, NY
Company	Oxychem/Olin

Tank Data

Type	Horizontal, Insulated
Shell length (ft)	6.4
Diameter (ft)	4
Working volume (gallons)	200
Turnovers per year	6
Net throughput (gal/yr)	1200
Vacuum setting (psig)	-0.03
Pressure setting (psig)	0.03

Meteorological Data

Average maximum ambient temperature (F)	55.8
Average minimum ambient temperature (F)	39.3
Average ambient temperature (F)	47.6
Average annual solar isolation factor (Btu/ft.ft.day)	1034
Average wind speed (mph)	12
Tank paint solar absorptance	0.17
Liquid bulk temperature (F)	47.6
Daily average liquid surface temperature (F)	48.9
Daily maximum liquid surface temperature (F)	53.2
Daily minimum liquid surface temperature (F)	44.7
Daily vapor temperature range (F)	16.80

Tank Contents

Component	Molecular	Vapor Pressure (psia)		
	Wieght	@Avg. Temp.	@Min. Temp.	@Max. Temp
Chlorobenzene	112.60	0.0912	0.0784	0.1057

Standing Storage Loss

Vapor space volume (ft^3)	51.23
Vapor density (lb/ft^3)	0.0019
Vapor space expansion factor	0.0308
Vented vapor saturation factor	0.9904
Total Standing Storage Loss (lb/yr)	1.07

Withdrawal Losses

Turnover factor (1 for N upto 36; (180+N)/(6N) for N >	1.0000
Working loss product factor	1.0000
Total withdrawal lossess (lb/yr)	0.29
Total losses (lb/yr)	**1.36**

NAPL STORAGE TANK - EMMISSION CALCULATION
NAPL as TETRACHLOROETHYLENE

Location	Niagara Falls, NY
Company	Oxychem/Olin

Tank Data

Type	Horizontal, Insulated
Shell length (ft)	6.4
Diameter (ft)	4
Working volume (gallons)	200
Turnovers per year	6
Net throughput (gal/yr)	1200
Vacuum setting (psig)	-0.03
Pressure setting (psig)	0.03

Meteorological Data

Average maximum ambient temperature (F)	55.8
Average minimum ambient temperature (F)	39.3
Average ambient temperature (F)	47.6
Average annual solar isolation factor (Btu/ft.ft.day)	1034
Average wind speed (mph)	12
Tank paint solar absorptance	0.17
Liquid bulk temperature (F)	47.6
Daily average liquid surface temperature (F)	48.9
Daily maximum liquid surface temperature (F)	53.2
Daily minimum liquid surface temperature (F)	44.7
Daily vapor temperature range (F)	16.80

Tank Contents

Component	Molecular Wieght	Vapor Pressure (psia)		
		@Avg. Temp.	@Min. Temp.	@Max. Temp
Tetrachloroehtylene	165.83	0.1432	0.1237	0.1652

Standing Storage Loss

Vapor space volume (ft^3)	51.23
Vapor density (lb/ft^3)	0.0043
Vapor space expansion factor	0.0317
Vented vapor saturation factor	0.9850
Total Standing Storage Loss (lb/yr)	2.56

Withdrawal Losses

Turnover factor (1 for N upto 36; (180+N)/(6N) for N >	1.0000
Working loss product factor	1.0000
Total withdrawal lossess (lb/yr)	0.68

Total losses (lb/yr)	**3.23**

APPENDIX A.11

Hydraulic Conductivity Data in Till

SUMMARY OF HYDRAULIC CONDUCTIVITY DATA IN TILL

Well/Borehole Number	Tested Interval (Ft. BGS)	Hydraulic Conductivity (cm/sec.)	Analysis Method	Data Source	Relationship to Bedrock Interface	Used in Calculations
OW2-79	23.7-30.5	2.7×10^{-7}	A (field)	CRA, 1983	screen bottom 1 foot above bedrock	Yes
OW3-79	25.0-26.5	6.7×10^{-9}	C (lab)	CRA, 1980	5 feet above bedrock	Yes
OW6-79	34.9-40.5	4.5×10^{-5}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW8-79	35.6-41.5	1.3×10^{-4}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW8-79	29.0-30.5	1.5×10^{-8}	C (lab)	CRA, 1980	11 feet above bedrock	Yes
OW9-79	24.1-30.5	5.8×10^{-7} / 2.3×10^{-5}	A / A (field)	CRA, 1983/87	screen bottom at bedrock interface	No
OW10-79	35.0-36.5	1.5×10^{-8}	C (lab)	CRA, 1980	12 feet above bedrock	Yes
OW11-79	34.1-40.0	4.9×10^{-7}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW13-79	31.8-38.0	6.1×10^{-5}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW14-79	39.2-46.2	1.2×10^{-6}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW15-79	30.9-36.5	1.4×10^{-6}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW16-79	32.8-39.5	4.2×10^{-5}	A (field)	CRA, 1983	top of screen in alluvium	No
OW17-79	42.3-48.0	1.9×10^{-4}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW18-79	28.9-35.3	8.1×10^{-6}	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW19-79	24.7-32.0	4.3×10^{-7}	A (field)	CRA, 1983	screen bottom 1 foot above bedrock	Yes
OW20-79	38.5-44.2	2.4×10^{-7}	A (field)	CRA, 1983	screen bottom at bedrock interface	No

Well/Borehole Number	Tested Interval (Ft. BGS)	Hydraulic Conductivity (cm/sec.)	Analysis Method	Data Source	Relationship to Bedrock Interface	Used in Calculations
OW21-79	35.3-42.0	$3.3x10^{-4}$	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW22-79	36.9-42.7	$3.2x10^{-6}$	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW24-79	38.3-44.5	$1.0x10^{-4}$	A (field)	CRA, 1983	screen bottom at bedrock interface	No
OW62-87	27.1-33.3	$1.0x10^{-5}$	A (field)	CRA, 1987	screen bottom at bedrock interface	No
B-22	21.0-38.5	$5.6x10^{-6}$	B (field)	Wehran, 1981	screen bottom 1.1 feet above bedrock (bentonite seal)	Yes
B-24	29.5-39.5	$3.0x10^{-7}$	B (field)	Wehran, 1981	screen bottom 1.1 feet above bedrock (bentonite seal)	Yes
B-29	30.5-37.5	$1.0x10^{-7}$	B (field)	Wehran, 1981	screen bottom 1.1 feet above bedrock (bentonite seal)	Yes
B-31	34.5-40.0	$7.8x10^{-9}$	B (field)	Wehran, 1981	screen bottom 1 foot above bedrock (bentonite seal)	Yes
B-34D	28.5-39.0	$6.8x10^{-9}$	B (field)	Wehran, 1981	screen bottom 1 foot above bedrock (bentonite seal)	Yes
A-252	9.0	$5.7x10^{-8}$	C (lab)	CRA, 1987	Unknown	Yes
E-286	19.9	$7.4x10^{-8}$	C (lab)	CRA, 1987	Unknown	Yes
I-136	13.5	$6.0x10^{-8}$	C (lab)	CRA, 1987	Unknown	Yes
Geometric mean value (at least one foot away from the bedrock)		$6.5x10^{-8}$ $1.4x10^{-7}$ $2.7x10^{-8}$	Field + Lab Field Only Lab Only			

NOTES

1. Data taken from Table 4.3, Summary of Hydraulic Conductivity Estimates, Remedial Investigation Report, 102nd Street Landfill Site, Niagara Falls, N.Y., July 1990.

2. Data from PFAR was not available when IER calculations were done. PFAR data will be included in the FER calculations as appropriate.

3.

Method	Equation
A	$K = \dfrac{r^2 \ln(L/R)}{2\,LT_o}$ $L/R > 8$ for isotropic conditions
B	In Situ - Equation Unknown
C	Laboratory Tests

Where:

L = length of saturated interval (cm)
r = radius of opening (pipe) where water levels were monitored (cm)
R = radius of borehole at interval "L" (cm)
T_o = elapsed time where $\dfrac{H-h_t}{H-h_o} = 0.37$ (sec)
H = water level at equilibrium
H_o = initial water level when slug was introduced/removed
h_t = water level at time t
K = hydraulic conductivity (cm/sec)

APPENDIX A.12

Test Results - Claymax Liner

(for reference only)

GEOSERVICES INC. CONSULTING ENGINEERS

5950 LIVE OAK PARKWAY, SUITE 330
NORCROSS, GEORGIA, USA 30093
TELEPHONE: (404) 448-5400
TELEFAX: (404) 368-0447
TELEX: 759285

11 November 1988

Mr. William J. Simpson
Vice President and Technical Director
James Clem Corporation
444 North Michigan
Suite 1610
Chicago, Illinois 60611

Subject: Interim Test Results - CLAYMAX Liner
Freeze-Thaw Hydraulic Conductivity Tests
GeoServices Project Number: P1061
GeoServices Document Number: NL00016

Dear Bill,

GeoServices Inc. Consulting Engineers (GeoServices) is pleased to present interim test results of the hydraulic conductivity tests performed to evaluate freeze-thaw effects on samples of CLAYMAX Liner. The tests were performed in general accordance with Task Number 9, as detailed in our proposal issued to the Clem Corporation (Clem) 13 September 1988.

The purpose of the tests was to indicate the impact of successive freeze-thaw cycles on single-layer samples of CLAYMAX liner. Four specimens were tested after a total of 0, 1, 5, and 10 freeze-thaw cycles. A 20-cycle freeze-thaw sample is currently in preparation for hydraulic conductivity testing. Following a discrete number of freeze-thaw cycles, specimens were mounted in the triaxial device, backpressure saturated and consolidated. The hydraulic conductivity was measured after completion of the primary consolidation phase. Test specimens were initially saturated at an effective stress at approximately 20 kPa then consolidated at an effective stress of 200 kPa. The hydraulic conductivity was measured at a gradient of approximately 600 over a period of about seven days.

In order to simulate the mechanics of freezing and thawing in the field, the CLAYMAX liner specimens had access to water at the lower boundary and were subjected to freezing temperatures at the upper boundary. This was done to simulate the one-dimensional propagation of the freezing front, as anticipated in field applications.

T8905

Report

Project

HYDRAULIC CONDUCTIVITY AND
COMPATIBILITY TESTING OF CLAYMAX
BALTIMORE COUNTY LANDFILL PROJECT
TOWNSON, MARYLAND

Client

CLEM ENVIRONMENTAL CORPORATION
444 NORTH MICHIGAN AVENUE, SUITE 1610
CHICAGO, IL 60611

Project # 25868-XH

Date MAY 11, 1989

STS Consultants Ltd.
Consulting Engineers

111 Pfingsten Road
Northbrook, Illinois 60062
(312) 272-6520

HYDRAULIC CONDUCTIVITY AND COMPATIBILITY TESTING OF CLAYMAX
BALTIMORE COUNTY LANDFILL PROJECT
TOWNSON, MARYLAND

SCOPE OF SERVICES

STS was to perform two hydraulic conductivity tests on sections of Claymax liner material in conjunction with a six inch sand layer utilizing leachates as the hydration medium and the permeants. The Claymax specimens were supplied to STS by Clem Environmental and the leachate specimens were obtained from L.A. Solamen, Inc. All testing materials were delivered to our Northbrook Testing Facility.

Test Equipment

The equipment used in the compatibility study was a triaxial compression permeameter. This equipment incorporates the use of a flexible membrane, preventing sidewall seepage, back pressure to facilitate specimen saturation small diameter burettes making measurement of small volumes of collected permeant possible and the system is closed preventing the permeant from being exposed to the surrounding air.

Specimen Construction

Each of the specimens, utilized throughout the testing program, consisted of an approximately six inch cylindrical column of silica sand on top of which a circular section of Claymax was placed. The orientation of the Claymax to the sand provided for permeant flow initiated through the sand followed by the Claymax section. The directional flow of the permeant, is similar to those conditions found in the field applications.

STS Consultants Ltd.

STS PROJECT NO. 25868-XH

PROJECT Baltimore County Landfill Project

DATE 4-24-89

SUMMARY OF HYDRAULIC CONDUCTIVITY TESTS

Permeant	Parkton Landfill	Eastern Sanitary Landfill
Sample No.	1	2
Classification	Claymax with 6" Silica Sand	Claymax with 6" Silica Sand
Dry Unit Weight (pcf)	51.6	62.5
Water Content (%)	Dry	Dry
Diameter (cm)	7.028	7.026
Length (cm)	0.568	0.616
Saturation B Value	0.97	0.99
Hydraulic Conductivity k (cm/sec)	1 ft. 2×10^{-10} 35 ft. 4×10^{-10}	1 ft. 3×10^{-10} 35 ft. 4×10^{-10}

Tests performed as falling head tests in a triaxial permeameter.

'89 04/11 15:46 ☎ 494 2931 BALTO. CO. 03

Rev: 12/87

BALTIMORE COUNTY
WASTEWATER MONITORING AND ANALYSIS DIVISION
INDUSTRIAL DISCHARGE CONTROL PROGRAM

Dennis F. Rasmussen
County Executive

SAMPLING/ANALYSIS FORM Sample No.: 9 02104

Industry Name: PARKTON Facility No.:

Address:

Telephone: Requested by: R. Much

Sampling Site Location: Cell #3

Special Instructions: pH, BOD, COD, TSS, Alkalinity, Chloride, Metals

FIELD

Date and Time of Sampling: Start 2/9/89 Finish

Sampled by: R. Much, B. Kramer

Type of Sample: Grab

Sampler Settings:

Sample Characteristics:

Preservatives Added:

Comments and Observations:

Delivered to Lab by: BK, RM Date: 2/9/89 Time: 2:20 P.M.

LABORATORY

Sample received by: WP Date: 2/9/89 Time: 2:20 P.M.

Characteristics of Note:

(Origin of Seed: Polyseed)

ANALYTICAL RESULTS

Code	BDL	Parameter	Conc. (mg/L)	Code	BDL	Parameter	Conc. (mg/L)
		pH	6.1	3011	0.05	Ni (Nickel)	1.44 mg/L
		BOD	38,888 mg/L	3015	0.01	Zn (Zinc)	5.45 mg/
		COD	60,831 mg/L	3130		Phenols	
		TSS	691 mg/L	3013	0.01	Silver	0.03 mg/L
[illegible]012		FOG - A&V					
[illegible]13		FOG - Petr		*		GRAB pH	
[illegible]26		P(Phosphorus)	interference				
[illegible]006	0.01	Cd(Cadmium)	0.10 mg/L			Total Fe	736.00 mg/L
[illegible]007	0.05	Cr(Chromium)	0.22 mg/L			Total alkalinity	15,000 mg/L
[illegible]008	0.02	Cu(Copper)	0.17 mg/L			Chloride	1,500 mg/L
[illegible]		Cn(Cyanide)					
[illegible]	0.10	Pb (Lead)	0.60 mg/L				

Rev: 12/87

BALTIMORE COUNTY
WASTEWATER MONITORING AND ANALYSIS DIVISION
INDUSTRIAL DISCHARGE CONTROL PROGRAM
SAMPLING/ANALYSIS FORM

Dennis F. Rasmussen
County Executive

Sample No.: 9 01110

Industry Name: EASTERN SANITARY LANDFILL Facility No.:

Address: Days Cove Road

Telephone: Requested by: P. Phillips

Sampling Site Location: Leachate pit

Special Instructions: STD 5, metals, Total alkalinity & Chlorides

FIELD

Date and Time of Sampling: Start 1/18/89 10:20 a.m. Finish

Sampled by: P. Phillips, T.E. Ryan

Type of Sample: Grab

Sampler Settings: N/A

Sample Characteristics: 1 quart: dark gray; 1 quart: dark brown

Preservatives Added: Cooled with ice

Comments and Observations:

Delivered to Lab by: PP, TER Date: 1/18/89 Time: 11:50 a.m.

LABORATORY

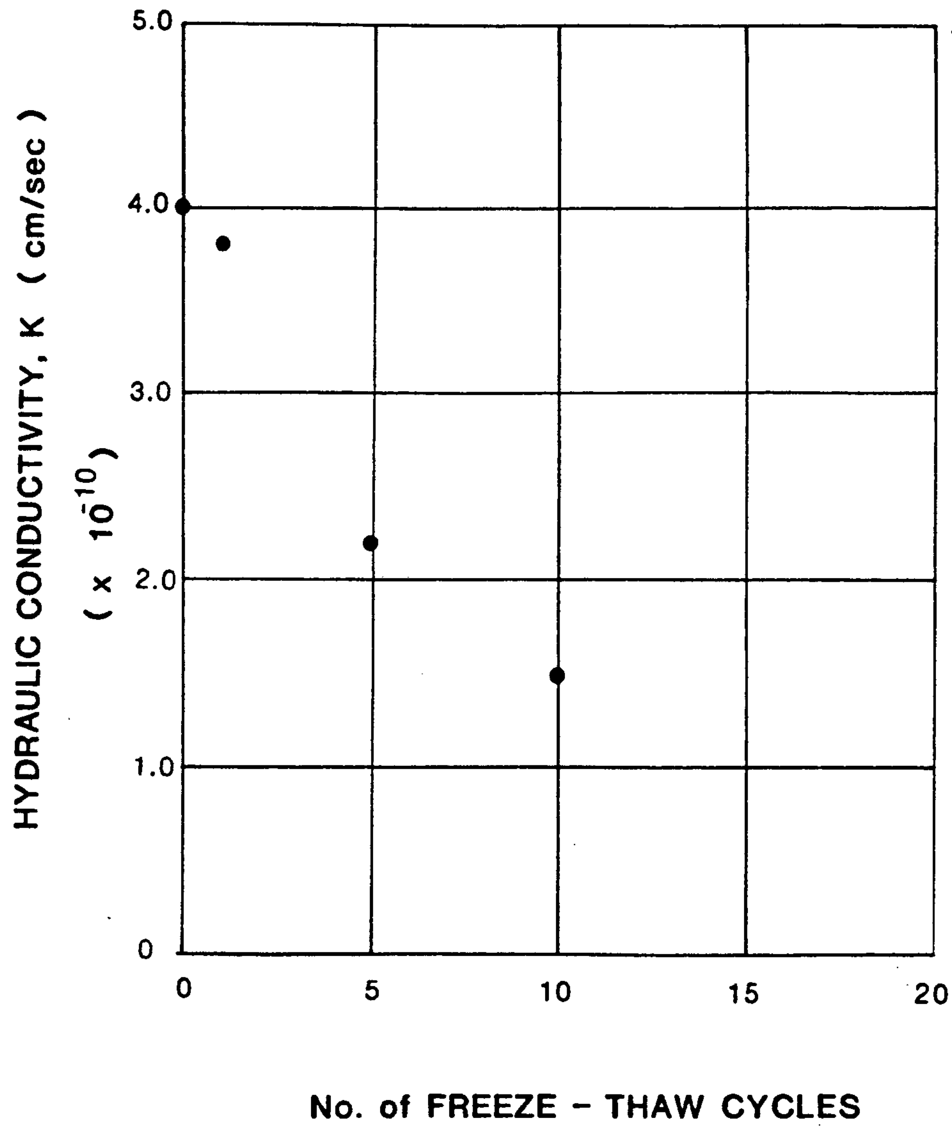
Sample received by: WP Date: 1/18/89 Time: 11:50 a.m.

Characteristics of Note:

(Origin of Seed: Polyseed)

ANALYTICAL RESULTS

Code	BDL	Parameter	Conc. (mg/L)	Code	BDL	Parameter	Conc. (mg/L)
		pH	6.3	3011	0.05	Ni (Nickel)	BDL
		BOD	122 mg/L	3015	0.01	Zn (Zinc)	0.05 mg/L
		COD	148 mg/L	3130		Phenols	
		TSS	125 mg/L	3013	0.01	Silver	BDL
5012		FOG - A&V					
5013		FOG - Petr		*		GRAB pH	
2026		P(Phosphorus)	2.52 mg/L				
3006	0.01	Cd(Cadmium)	BDL			Total Fe	3.88 mg/L
3007	0.05	Cr(Chromium)	BDL			Total alkalinity	350 mg/L
3008	0.02	Cu(Copper)	0.04 mg/L			Chloride	80 mg/L
3002		Cn(Cyanide)					
3009	0.10	Pb (Lead)	0.36 mg/L				



GEOSERVICES INC. CONSULTING ENGINEERS	CLAYMAX STUDY
	PROJECT NO. P1061
	DOCUMENT NO. NL00016
	PAGE NO.

The test results obtained thus far indicate a very slight reduction in hydraulic conductivity with increasing number of freeze-thaw cycles. This is likely due to the weathering and subsequent breakdown of the bentonite particles, thereby exposing the pore water to more bentonite particle surface area. Subsequent consolidation causes the bentonite to achieve a slightly denser configuration, giving a very slight reduction in hydraulic conductivity. The results of the hydraulic conductivity tests are shown as hydraulic conductivity versus number of freeze thaw cycles.

GeoServices appreciates the opportunity to provide our testing services to Clem. As we discussed, the 20-cycle sample will become available for testing within a couple of days; the results of the hydraulic conductivity test will be available about one week later. Should you have any questions regarding the contents of this interim report, please do not hesitate to contact us.

Very truly yours,

Scott M. Luettich, E.I.T.
Laboratory Manager

Neil D. Williams, Ph.D., P.E.
Vice President

GEOSYNTEC CONSULTANTS (formerly GeoServices Inc. Consulting Engineers)

Geomechanics and Environmental Laboratory
1600 Oakbrook Drive, Suite 565
Norcross, Georgia 30093 USA
Telephone: (404) 242-7624
Telefax: (404) 242-8615

17 July 1991

Mr. Martin J. Simpson
James Clem Corporation
444 North Michigan, Suite 1610
Chicago, Illinois 60611

Subject: Preliminary Results
Hydraulic Transmissivity Testing of
Composite Lining Systems

Dear Mr. Simpson:

GeoSyntec Consultants is pleased to provide the attached preliminary test results to James Clem Corporation. The testing procedures and conditions were outlined in GeoSyntec Consultants' Work Plan and Agreement for Testing Services, dated 13 May 1991. The preliminary test results of the first two test series are summarized in the attached Tables and Figures.

GeoSyntec Consultants appreciates the opportunity to provide laboratory testing services for James Clem Corporation. Should you have any questions regarding the attached information, please do not hesitate to contact either of the undersigned.

Sincerely,

Scott M Luettich

Scott M. Luettich, P.E.
Laboratory Manager

Robert C. Bachus

Robert C. Bachus, Ph.D.
Senior Project Manager

Attachments

GL3024/GEL91179

TABLE 1A

HYDRAULIC TRANSMISSIVITY TEST #1A RESULTS
TYPAR 3601 GEOTEXTILE WITHOUT CLAYMAX

Confining Pressure (psf)	Duration of Pressure (minutes)	Hydraulic Gradient ( )	Hydraulic Transmissivity (m^2/s)	Unit Flowrate (gpm/ft)
500	15	0.10 0.25 0.50	2.8×10^{-3} 1.7×10^{-3} 1.3×10^{-3}	1.36 2.00 3.21
1,000	15	0.10 0.25 0.50	2.6×10^{-3} 1.5×10^{-3} 1.3×10^{-3}	1.26 1.78 3.16
2,000	15	0.10 0.25 0.50	2.5×10^{-3} 1.4×10^{-3} 1.3×10^{-3}	1.20 1.65 3.14
4,000	15	0.10 0.25 0.50	2.4×10^{-3} 1.3×10^{-3} 1.3×10^{-3}	1.18 1.60 3.09
8,000	15	0.10 0.25 0.50	2.1×10^{-3} 1.3×10^{-3} 1.2×10^{-3}	1.03 1.55 2.91
16,000	15	0.10 0.25 0.50	1.7×10^{-3} 1.1×10^{-3} 9.5×10^{-4}	0.83 1.34 2.29

Notes:

Test Date: 15 May 1991
Test Configuration (from top to bottom): load platen/60-mil geomembrane/ Typar 3601 geotextile/geonet/60-mil geomembrane/base platen.
Confining Pressures and Pressure Durations as shown.

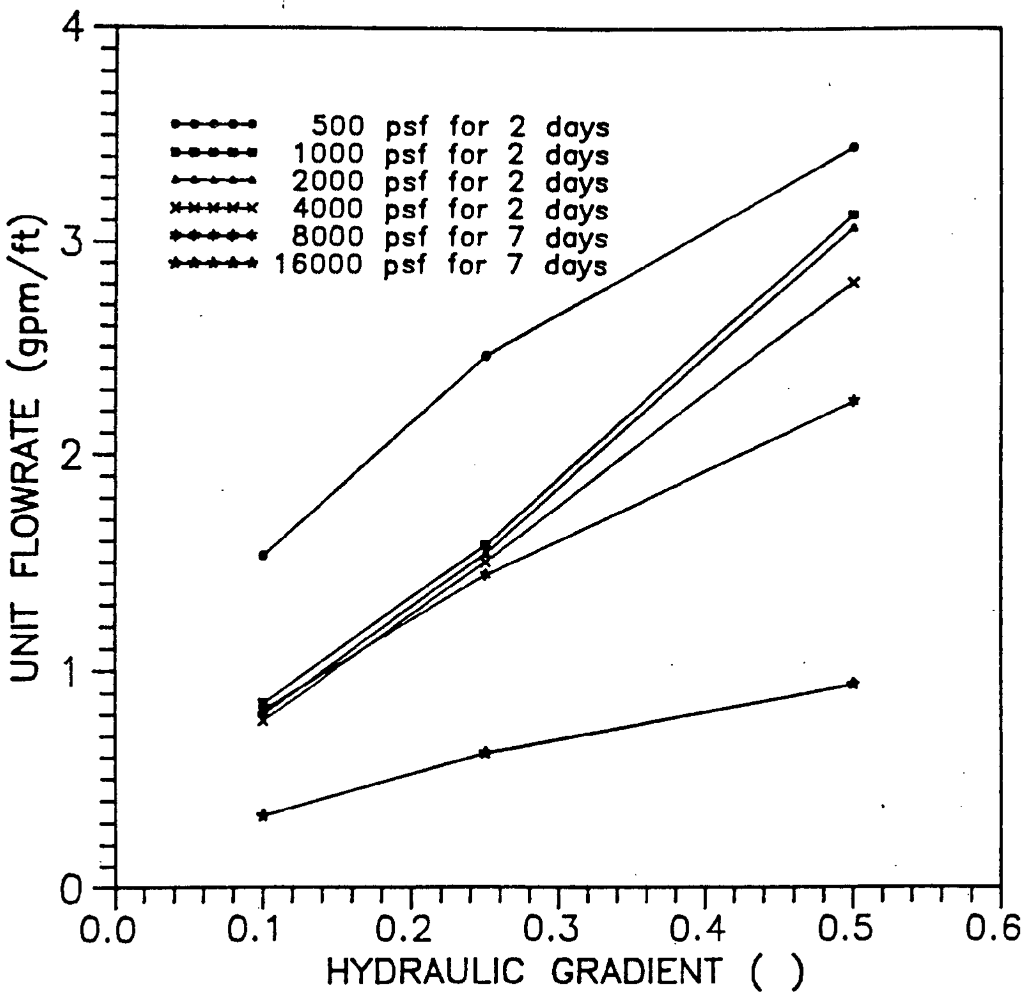
Specimen saturation was considered complete when a Skempton's Pore Pressure B-parameter of 0.95 or greater was obtained. The "B" parameter is simply a ratio of an increase in pore water pressure to a simultaneous increase in confining pressure. When full specimen saturation was determined, permeant flow was initiated through the bottom of the test specimen, allowed to flow through the top of the test specimen and collect in a calibrated burette. The test was performed utilizing two separate gradients. The initial gradient consisted of an application of a hydraulic head of one foot. The second gradient was applied as a hydraulic head equivalent to 35 feet.

During the entire test, permeant volume versus time measurements were recorded and the hydraulic conductivity of the test specimen at the two gradients was determined. The test was allowed to continue until it had been determined that a minimum of three pore volumes of pore fluid had passed through the test specimen. Once this had occurred and steady state flow had been established, the test was terminated.

Laboratory Test Results

As a result of the testing as outlined above, the Claymax section utilizing the Parkton Landfill Leachate, as the permeant, obtained hydraulic conductivity values of 2×10^{-10} centimeters per second (cm/sec) for a hydraulic head of one foot and 4×10^{-10} cm/sec for a hydraulic head of 35 feet. The Claymax section exposed to the Eastern Sanitary Landfill leachate obtain hydraulic conductivity values of 3 x 10 -10 cm/sec utilizing a hydraulic head of 1 foot and 4×10^{-10} cm/sec utilizing a hydraulic head of 35 feet. A summary of specific specimen characteristics and final hydraulic conductivity values is attached to this report.

HYDRAULIC TRANSMISSIVITY TEST #1B RESULTS
TYPAR 3601 GEOTEXTILE WITH CLAYMAX



Notes:
Test Starting Date: 15 May 1991
Test Configuration (from top to bottom): load platen/ 60-mil geomembrane/Claymax/Typar 3601 geotextile/ geonet/60-mil geomembrane/base platen.
Confining Pressures and Pressure Durations are as Shown.

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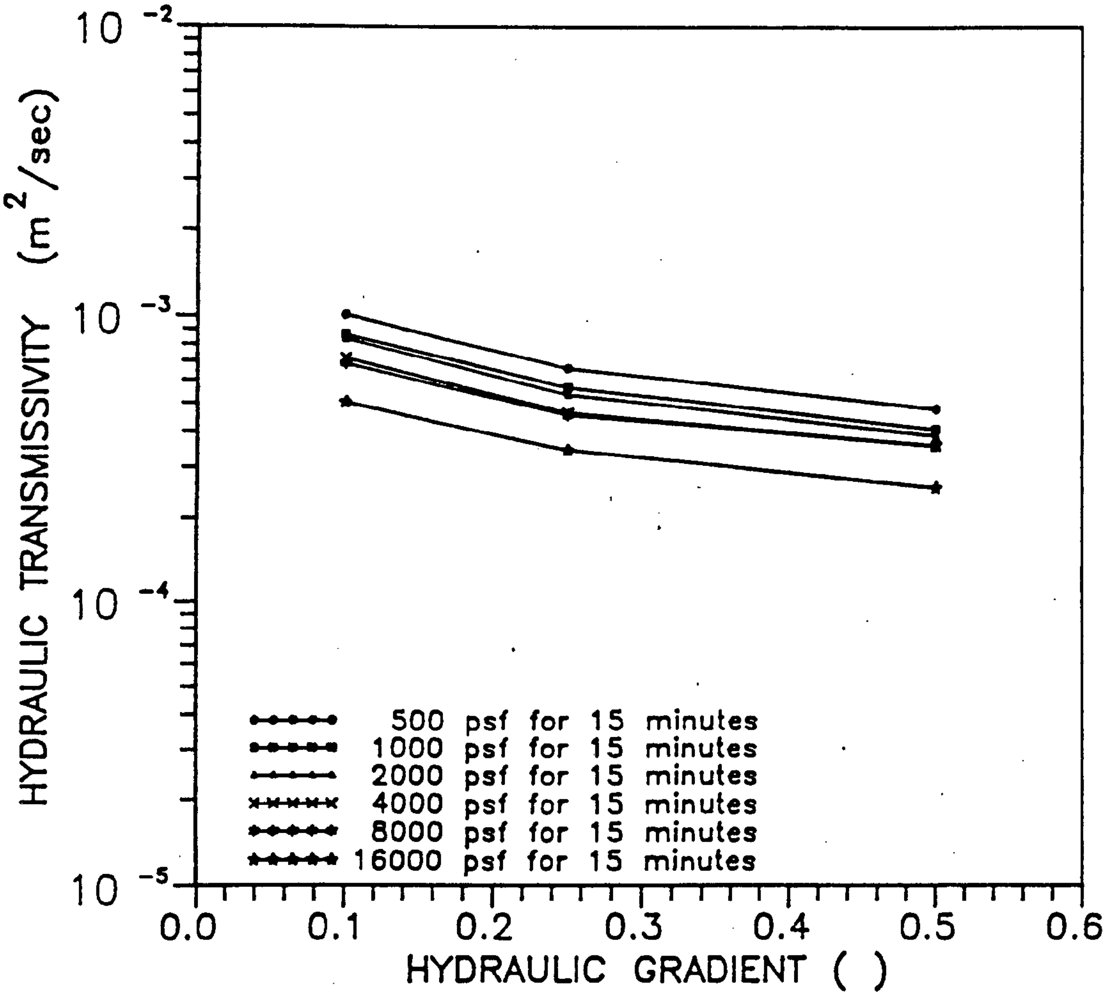
HYDRAULIC TRANSMISSIVITY TEST #2A RESULTS
AMOCO 4557 GEOTEXTILE WITHOUT CLAYMAX



Notes:
Test Date: 11 June 1991
Test Configuration (from top to bottom): load platen/ 60-mil geomembrane/Amoco 4557 geotextile/ geonet/60-mil geomembrane/base platen.
Confining Pressures and Pressure Durations are as shown.

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FIGURE NO.	2A-1
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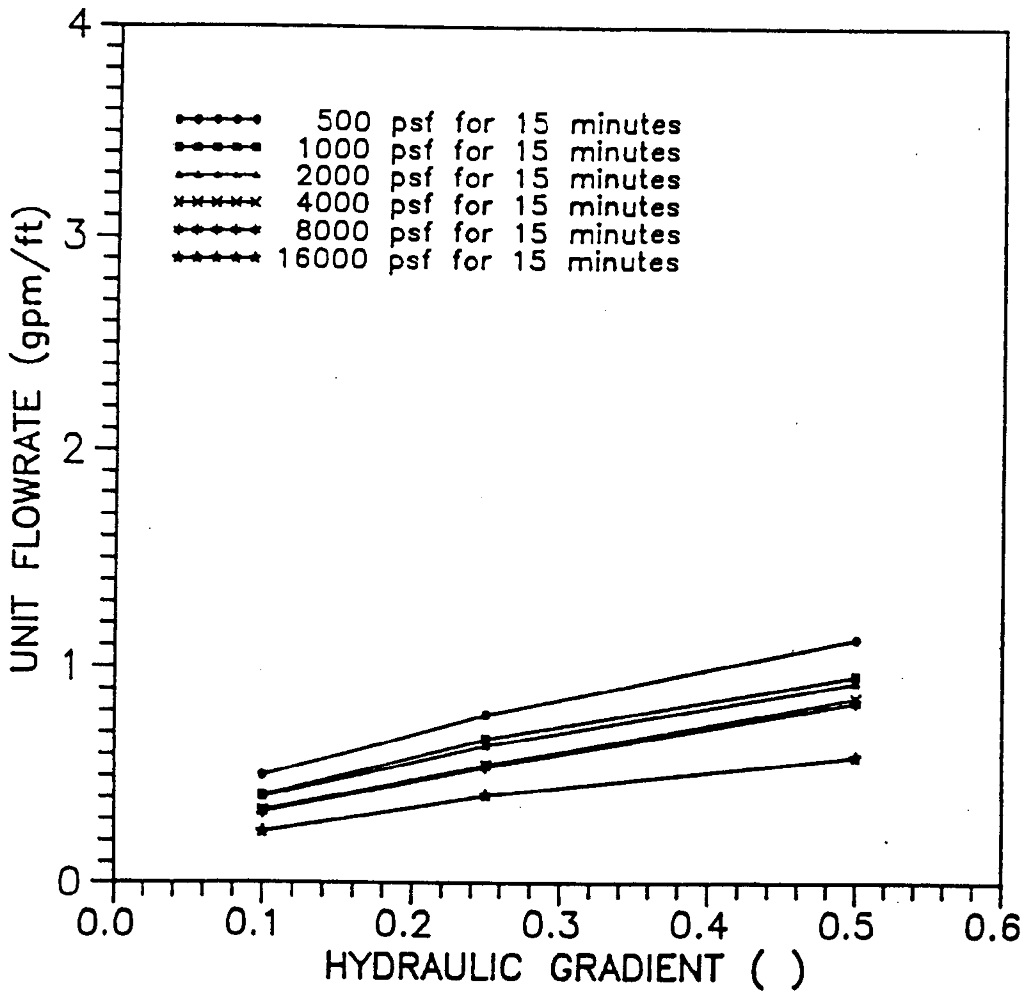
HYDRAULIC TRANSMISSIVITY TEST #2A RESULTS
AMOCO 4557 GEOTEXTILE WITHOUT CLAYMAX



Notes:
Test Date: 11 June 1991
Test Configuration (from top to bottom): load platen/ 60-mil geomembrane/Amoco 4557 geotextile/ geonet/60-mil geomembrane/base platen.
Confining Pressures and Pressure Durations are as shown.

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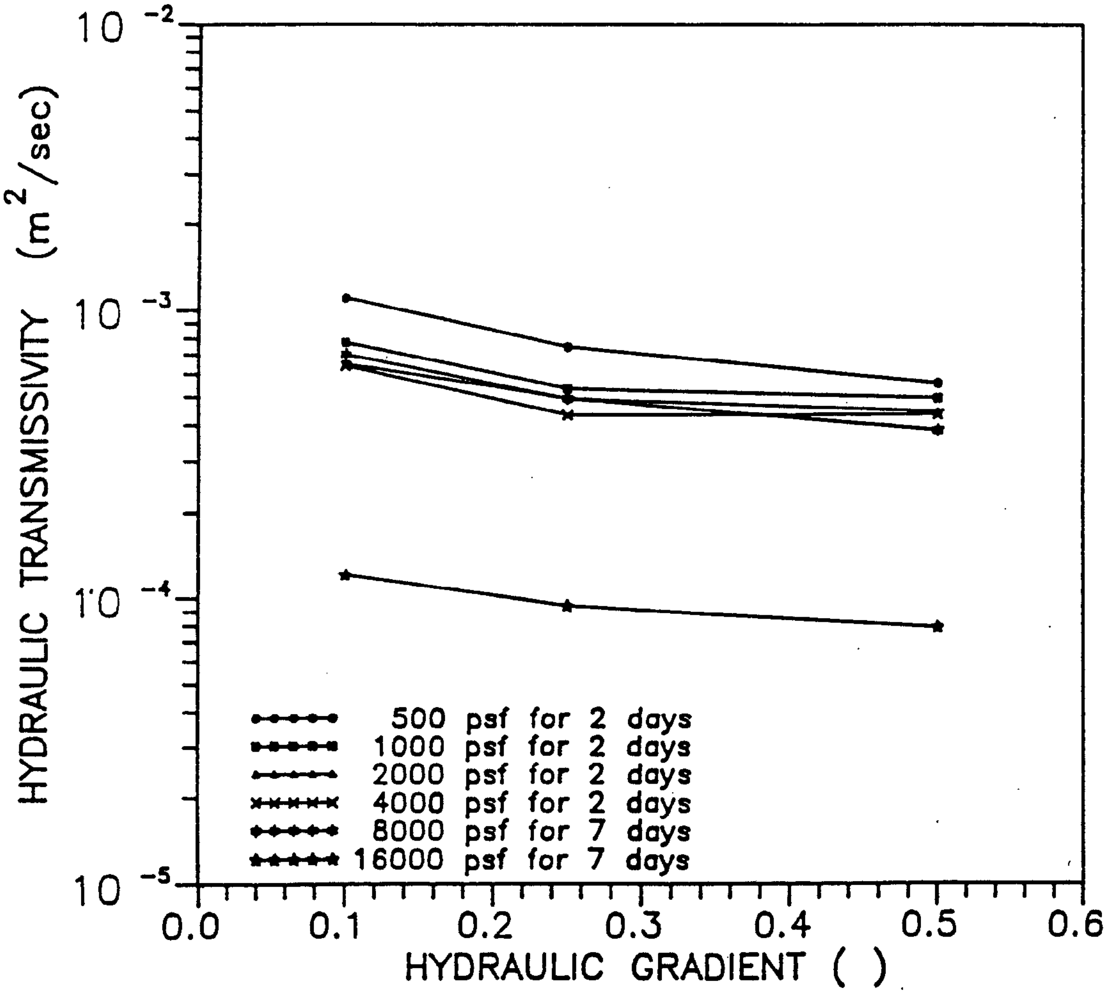
FIGURE NO.	2A-2
PROJECT NO.	GL3024
DOCUMENT NO.	GEL91179
PAGE NO.	

HYDRAULIC TRANSMISSIVITY TEST #2B RESULTS
AMOCO 4557 GEOTEXTILE WITH CLAYMAX



Notes:
Test Starting Date: 12 June 1991
Test Configuration (from top to bottom): load platen/ 60–mil geomembrane/Claymax/Amoco 4557 geotextile/ geonet/60–mil geomembrane/base platen.
Confining Pressures and Pressure Durations are as shown.

FIGURE NO.	2B-1
PROJECT NO.	GL3024
DOCUMENT NO.	GEL91179
PAGE NO.	

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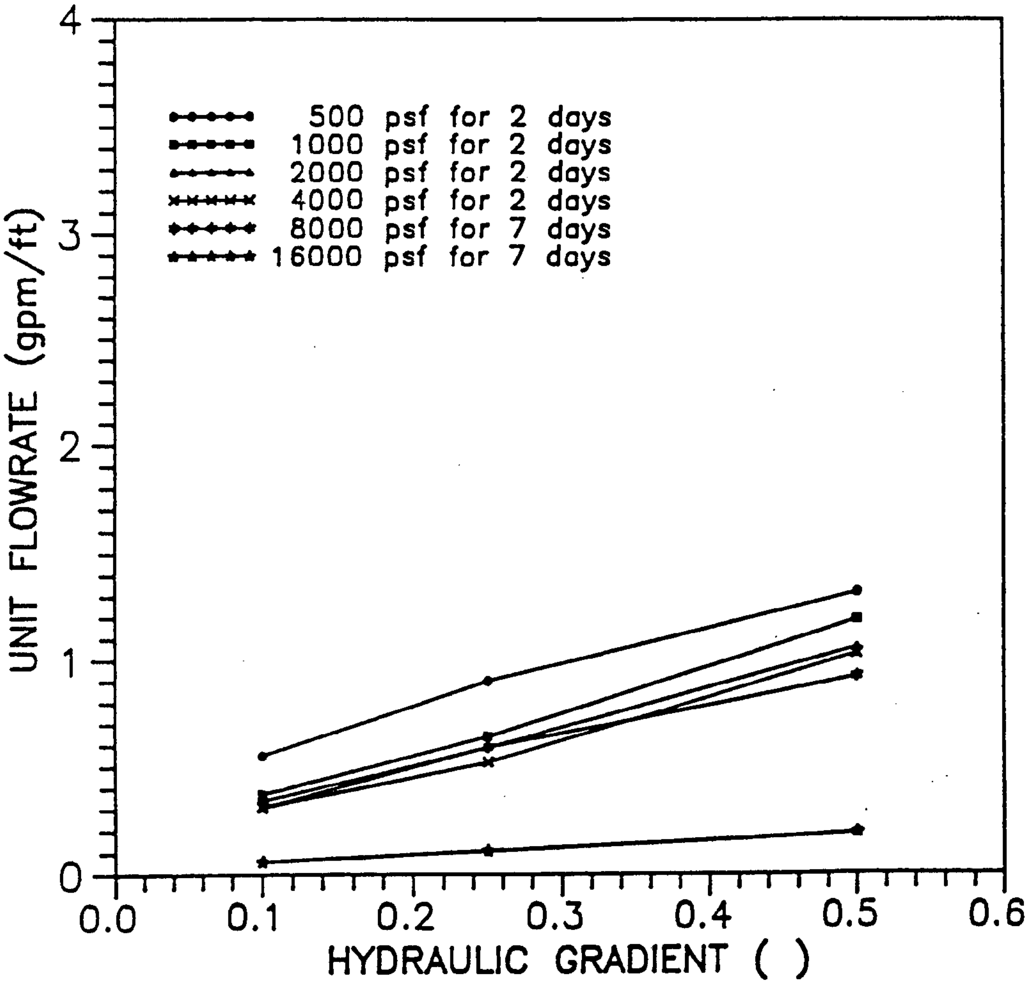
HYDRAULIC TRANSMISSIVITY TEST #2B RESULTS
AMOCO 4557 GEOTEXTILE WITH CLAYMAX



Notes:
Test Starting Date: 12 June 1991
Test Configuration (from top to bottom): load platen/ 60-mil geomembrane/Claymax/Amoco 4557 geotextile/ geonet/60-mil geomembrane/base platen.
Confining Pressures and Pressure Durations are as shown.

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	2B-2
PROJECT NO.	GL3024
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PAGE NO.	

GeoSyntec® Consultants

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No. T9209
24 November 1992

Mr. Thomas N. Dobras, P.E.
Technical Manager
James Clem Corporation
444 N. Michigan Avenue, Suite 1610
Chicago, Illinois 60611

Subject: Final Report
Interface Direct Shear Testing
Select Claymax Shear-Pro Bentonite Composite Interfaces

Dear Mr. Dobras:

GeoSyntec Consultants is pleased to present the enclosed final report on the interface direct shear testing performed for James Clem Corporation (Clem). The tests were conducted in accordance with the procedures defined during the 19 March 1992 telephone conversation between Mr. Thomas N. Dobras, P.E. of Clem and Mr. Robert H. Swan, Jr. of GeoSyntec Consultants. All of the interface direct shear testing was conducted at GeoSyntec Consultants' Geomechanics and Environmental Laboratory located in Atlanta, Georgia.

GeoSyntec Consultants appreciates the opportunity to provide laboratory testing services for Clem. Should you have any questions regarding the enclosed report, please do not hesitate to contact either of the undersigned.

Sincerely,

Zehong Yuan, Ph.D.
Assistant Division Manager
Soil-Geosynthetic Interaction Testing

Robert H. Swan, Jr., E.I.T.
Division Manager
Soil-Geosynthetic Interaction Testing

Robert C. Bachus, Ph.D.
Associate

Enclosure

GL3160/GEL92211

FINAL REPORT
INTERFACE DIRECT SHEAR TESTING

SELECT CLAYMAX SHEAR-PRO BENTONITE COMPOSITE INTERFACES

Prepared for

James Clem Corporation
444 N. Michigan Avenue, Suite 1610
Chicago, Illinois 60611

Prepared by

GeoSyntec Consultants
Geomechanics and Environmental Laboratory
5775 Peachtree Dunwoody Road, Suite 10D
Atlanta, Georgia 30342

Project Number: GL3160

24 November 1992

TABLE OF CONTENTS

1. INTRODUCTION

1.1 Terms of Reference

This report was prepared by Mr. Robert H. Swan, Jr. and Dr. Zehong Yuan and was reviewed by Dr. Robert C. Bachus, all of GeoSyntec Consultants, Atlanta, Georgia. The laboratory testing program described in this report was performed at the request of Mr. Thomas N. Dobras, P.E. of James Clem Corporation (Clem), Chicago, Illinois.

Clem authorized GeoSyntec Consultants to undertake a laboratory testing program to evaluate the shearing resistance at the interface of selected Claymax Shear-Pro bentonite composites and various types of geomembranes and soils. All samples of the geosynthetic materials used at the test interfaces were provided to GeoSyntec Consultants by Clem.

The tests were conducted in accordance with the test procedures defined during the 19 March 1992 telephone conversation between Mr. Dobras and Mr. Swan. The sample preparation procedures and testing conditions used for the program were provided by Clem to model anticipated field conditions. All of the interface direct shear testing was conducted at GeoSyntec Consultants' Geomechanics and Environmental Laboratory located in Atlanta, Georgia.

1.2 Organization

The remainder of the report is organized as follows:

- background information regarding the laboratory testing program is presented in Section 2;

- details of the laboratory testing program are described in Section 3; and

- results of the laboratory testing program are presented in Section 4.

2. BACKGROUND

2.1 Overview

The laboratory testing program consisted of interface direct shear tests between select Claymax Shear-Pro bentonite composites, textured and smooth geomembranes and two types of soils. A total of seven, five-point interface direct shear test series were performed. Each test series included testing a specific Claymax Shear-Pro bentonite composite-geomembrane or -soil interface at five normal stress levels ranging from 50 to 1000 psf (2 to 49 kPa). A general description of the test method used during this program is given in the following section.

2.2 Interface Direct Shear Tests

The interface direct shear tests were performed in general accordance with the American Society for Testing and Materials (ASTM) Draft Standard Test Method D 35.01.81.07, "*Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method*". The interface direct shear tests were conducted in GeoSyntec Consultants' large direct shear device. The device has a shear box which consists of an upper and lower component. The upper component measures 12 in. by 12 in. (300 mm by 300 mm) in plan and 3 in. (75 mm) in depth. The lower component measures 12 in. by 14 in. (300 mm by 355 mm) in plan and 3 in. (75 mm) in depth.

3. TESTING PROGRAM

3.1 Overview

The testing program consisted of seven interface direct shear test series. The materials used in the testing program are summarized in Section 3.2. Details of each of the test series and specific testing procedures are presented in Section 3.3.

3.2 Geosynthetic and Soil Materials

3.2.1 Geosynthetic Materials

All geosynthetic materials used in the testing program were provided to GeoSyntec Consultants by Clem. These materials are referenced by name in this report, and include:

- Geomembranes:
  - 60-mil (1.5-mm) thick National Seal Company (NSC) textured high density polyethylene (HDPE) geomembrane, referred to as 60-mil NSC textured HDPE geomembrane;
  - 60-mil (1.5-mm) thick NSC textured very low density polyethylene (VLDPE) geomembrane, referred to as 60-mil NSC textured VLDPE geomembrane; and
  - 60-mil (1.5-mm) thick NSC smooth HDPE geomembrane, referred to as 60-mil NSC smooth HDPE geomembrane.
- Bentonite Composites:
  - Claymax Shear-Pro bentonite composite consisting of Amoco 4034 geotextile on one side of the bentonite component and Clem HS geotextile on the other side, referred to as Claymax Shear-Pro Prototype No. 3;

- Claymax Shear-Pro bentonite composite consisting of Amoco 4034 geotextile on one side of the bentonite component and Amoco EP356 geotextile on the other side, referred to as Claymax Shear-Pro Prototype No. 4; and
- Production run of Claymax Shear-Pro bentonite composite consisting of Amoco 4034 geotextile on one side of the bentonite component and Amoco EP356 geotextile on the other side, referred to as Claymax Shear-Pro 500SP bentonite composite.

3.2.2 Soil Materials

At the request of Clem, GeoSyntec Consultants provided a clay soil and an American Association of State Highway and Transportation Officials (AASHTO) No. 57 stone for use in the study. The clay soil was a sandy clay material having a plasticity index of 13, a liquid limit of 25, and a soil classification as determined by the Unified Soil Classification System of CL (lean clay with sand). The AASHTO No. 57 stone had a maximum particle size of about 1.5 in. (38 mm) and only about five percent of the material was finer than 0.2 in. (5 mm) diameter. The particle shape of the AASHTO No. 57 stone was angular to sub-angular. A concrete sand was used as a bedding layer above and below each test interface. This concrete sand was also provided by GeoSyntec Consultants.

3.3 <u>Interface Direct Shear Tests</u>

3.3.1 Configuration of the Test Specimens

The configurations of the specimens used in the seven test series are described below. Table 1 summarizes the general testing conditions that were used for each of the interface direct shear test series.

- *Test Series Number 1:* interface between dry Claymax Shear-Pro Prototype No. 3 bentonite composite and 60-mil NSC textured HDPE geomembrane. From top to bottom, each test specimen consisted of:
  - concrete sand;
  - Claymax Shear-Pro Prototype No. 3 bentonite composite (Clem HS geotextile component against geomembrane);
  - 60-mil NSC textured HDPE geomembrane; and
  - concrete sand.

- *Test Series Number 2:* interface between soaked Claymax Shear-Pro Prototype No. 3 bentonite composite and 60-mil NSC textured HDPE geomembrane. From top to bottom, each test specimen consisted of:
  - concrete sand;
  - Claymax Shear-Pro Prototype No. 3 bentonite composite (Clem HS geotextile component against geomembrane);
  - 60-mil NSC textured HDPE geomembrane; and
  - concrete sand.

- *Test Series Number 3:* interface between hydrated Claymax Shear-Pro Prototype No. 4 bentonite composite and 60-mil NSC textured HDPE geomembrane. From top to bottom, each test specimen consisted of:
  - concrete sand;
  - Claymax Shear-Pro Prototype No. 4 bentonite composite (Amoco EP356 geotextile component against geomembrane);
  - 60-mil NSC textured HDPE geomembrane; and
  - concrete sand.

- *Test Series Number 4:* interface between soaked Claymax Shear-Pro Prototype No. 3 bentonite composite and 60-mil NSC textured VLDPE geomembrane. From top to bottom, each test specimen consisted of:

    - concrete sand;
    - Claymax Shear-Pro Prototype No. 3 bentonite composite (Clem HS geotextile component against geomembrane);
    - 60-mil NSC textured VLDPE geomembrane; and
    - concrete sand.

- *Test Series Number 5:* interface between soaked Claymax Shear-Pro Prototype No. 3 bentonite composite and 60-mil NSC smooth HDPE geomembrane. From top to bottom, each test specimen consisted of:
    - concrete sand;
    - Claymax Shear-Pro Prototype No. 3 bentonite composite (Clem HS geotextile component against geomembrane);
    - 60-mil NSC smooth HDPE geomembrane; and
    - concrete sand.

- *Test Series Number 6:* interface between soaked AASHTO No. 57 stone and Claymax Shear-Pro Prototype No. 4 bentonite composite. From top to bottom, each test specimen consisted of:

    - AASHTO No. 57 stone;
    - Claymax Shear-Pro Prototype No. 4 bentonite composite (Amoco 4034 geotextile against AASHTO No. 57 stone); and
    - concrete sand.

- *Test Series Number 7:* interface between soaked sandy clay soil and Claymax Shear-Pro 500SP bentonite composite. From top to bottom, each test specimen consisted of:

    - sandy clay soil;
    - Claymax Shear-Pro 500SP bentonite composite (Amoco 4034 geotextile against sandy clay soil); and
    - concrete sand.

3.3.2 Test Procedure

A summary of the test equipment and conditions used to conduct the interface direct shear tests is presented in Table 2. This table includes the size of the shear box, the initial moisture content of the bentonite composite, soaking/hydration stress, time for soaking/ hydration, the moisture content of the bentonite composite at the completion of testing, the normal stress at the interface during testing, and the horizontal displacement rate for each test.

In each test in Test Series 1 through 5, the bentonite composite and the geomembrane specimens were attached to the upper and lower shear box, respectively, with mechanical compression clamps to confine failure to the bentonite composite-geomembrane interface. For each test, fresh geosynthetic specimens were prepared for each normal stress condition.

In each test in Test Series 6 and 7, a fresh bentonite composite specimen was attached to the lower shear box with mechanical compression clamps to confine failure to the soil-bentonite composite interface. Fresh specimens of the sandy clay soil and AASHTO No. 57 stone were placed and compacted directly about the bentonite composite by hand tamping. The sandy clay soil was moisture-conditioned to approximately 13.5 percent and compacted to approximatley 112 to 113 lb/ft^3 (17.6 to 17.7 kN/m^3) which corresponded to 95 percent of the maximum dry unit weight and 2 percentage points wet of the optimum moisture content, based on the standard Proctor compaction test (ASTM D 698). The AASHTO No. 57 stone was placed under dry condition and compacted to approximately 90 percent relative density. The reported values of dry unit weight were determined by measuring the as-placed volume of soil and dividing this volume into the calculated total dry weight of the moisture-conditioned soil specimen (Test Series 7) or dry stone specimen (Test Series 6).

For Test Series 1, the bentonite composite-geomembrane interface was tested under dry conditions. For Test Series 2, 4, 5, 6, and 7, the bentonite composite-geomembrane or the soil-bentonite composite interface was soaked in water for 24 hours under an applied normal stress of 100 psf (5 Kpa). The normal stress was applied prior to the soaking. After the 24-hour soaking period, each interface was then placed and secured in the shear box. For Test Series 3, the bentonite composite was hydrated in water for 24 hours under an applied normal stress of 100 psf (5 kPa). The normal stress was applied prior to the hydration. After the 24-hour hydration period, the bentonite composite was then placed on the geomembrane and secured in the shear box. In all of the test series, the test normal stress was applied to the interface within approximately five minutes of the removal of the soaking/hydration normal stress. Each of the test specimens were then sheared immediately upon application of the test normal stress.

Other features of the testing procedure included the following:

- a freshly remolded 3-in. (75-mm) thick layer of concrete sand was used as the bedding layer beneath the lower geosynthetic specimen in each test; the concrete sand was compacted by hand tamping to approximately 90 percent relative density under dry conditions;
- each specimen was sheared at a constant displacement rate immediately after application of the normal stress;
- the direction of shear for each test was in the direction of manufacture (machine direction) of the geosynthetic samples;
- all of the tests were performed using a constant effective sample area, where the lower geosynthetic component was larger than the upper shear box; therefore, no area correction was required when computing shear stresses; and
- all of the tests were sheared until a constant, residual load was recorded.

4. TEST RESULTS

4.1 Overview

The data reduction procedures and test results are summarized in the following section. The interface direct shear test results and comprehensive summary plots are presented in the following appendices to this report.

- Appendix A: (Test Series Numbers 1 through 7)
  - shear force versus displacement results; and
  - shear stress versus normal stress results for tests conducted at 50, 200, 400, 700, and 1000 psf (2, 10, 20, 34, and 49 kPa).

- Appendix B: (Test Series Numbers 1 through 7)
  - shear stress versus normal stress results for tests conducted at 50, 200, and 400 psf (2, 10, and 20 kPa); and
  - shear stress versus normal stress results for tests conducted at 400, 700, and 1000 psf (20, 34, and 49 kPa).

4.2 Interface Direct Shear Tests

The total stress interface shearing resistance was evaluated for each applied normal stress. The test data were plotted on a graph of shear force versus horizontal displacement and are presented in Appendix A. The peak value of shear force was used to calculate the peak shear stress. For this report the residual shear stress was calculated from the stabilized post-peak shear force which occurred at the end of each test.

Once the specimens were assembled, a flexible rubber membrane was used to encase the specimens while sealed in the triaxial permeameter chamber.

Test Procedures

After its initial construction and placement in a triaxial compression permeameter each of the specimens is backpressure saturated. To aide in specimen saturation, carbon dioxide gas was allowed to flow freely through the test specimen, inundating the voids in the sand and dry Claymax. The use of this carbon dioxide gas has been accepted as a procedure to aide in specimen saturation. The carbon dioxide gas will go into solution more readily than normal atmospheric air. Once it was determined that the carbon dioxide gas had completely inundated the voids of the test specimen, the permeants were allowed to free flow through the test specimen first saturating the silica sand and then the Claymax section. For this study, the leachates were utilized both as a set hydrating medium and as the actual permeant for the hydraulic conductivity determination.

Two leachates were used during the study. The first was labeled Parkton Landfill and the second labeled as Eastern Sanitary Landfill. It is the understanding of STS Consultants that the two leachates were a municipal landfill leachate and contained such things as heavy metals, phenals, cyanide, copper, phosphorus and other substances.

Once the leachate had fully hydrated the test specimen, the specimen was allowed to stand for a 24 hour hydration period. Following the hydration period, the backpressure saturation techniques were implemented to complete the saturation procedures. This was accomplished by simultaneously increasing the cell and back pressures in increments while maintaining a pressure differential of 0.125 kilograms per square centimeter (KSC). Pressures were incrimentally increased until obtaining testing pressures of 4.125 KSC cell pressure and 4.00 KSC back pressure.

TABLE 1B

HYDRAULIC TRANSMISSIVITY TEST #1B RESULTS
TYPAR 3601 GEOTEXTILE WITH CLAYMAX

Confining Pressure (psf)	Duration of Pressure (days)	Hydraulic Gradient ( )	Hydraulic Transmissivity (m^2/s)	Unit Flowrate (gpm/ft)
500	2	0.10 0.25 0.50	3.2×10^{-3} 2.0×10^{-3} 1.4×10^{-3}	1.53 2.46 3.45
1,000	2	0.10 0.25 0.50	1.8×10^{-3} 1.3×10^{-3} 1.3×10^{-3}	0.85 1.58 3.13
2,000	2	0.10 0.25 0.50	1.7×10^{-3} 1.3×10^{-3} 1.3×10^{-3}	0.80 1.54 3.07
4,000	2	0.10 0.25 0.50	1.6×10^{-3} 1.2×10^{-3} 1.2×10^{-3}	0.77 1.50 2.81
8,000	7	0.10 0.25 0.50	1.7×10^{-3} 1.2×10^{-3} 9.3×10^{-4}	0.82 1.44 2.25
16,000	7	0.10 0.25 0.50	6.9×10^{-4} 5.1×10^{-4} 3.9×10^{-4}	0.33 0.62 0.94

Notes:

Test Starting Date: 15 May 1991
Test Configuration (from top to bottom): load platen/60-mil geomembrane/ Claymax/Typar 3601 geotextile/geonet/60-mil geomembrane/base platen.
Hydration of the Claymax specimen occurred under the 500 psf confining pressure for 2 days.

TABLE 2A

HYDRAULIC TRANSMISSIVITY TEST #2A RESULTS
AMOCO 4557 GEOTEXTILE WITHOUT CLAYMAX

Confining Pressure (psf)	Duration of Pressure (minutes)	Hydraulic Gradient ( )	Hydraulic Transmissivity (m^2/s)	Unit Flowrate (gpm/ft)
500	15	0.10 0.25 0.50	1.0×10^{-3} 6.5×10^{-4} 4.7×10^{-4}	0.50 0.78 1.13
1,000	15	0.10 0.25 0.50	8.6×10^{-4} 5.6×10^{-4} 4.0×10^{-4}	0.41 0.67 0.96
2,000	15	0.10 0.25 0.50	8.3×10^{-4} 5.3×10^{-4} 3.8×10^{-4}	0.40 0.64 0.93
4,000	15	0.10 0.25 0.50	7.1×10^{-4} 4.6×10^{-4} 3.5×10^{-4}	0.34 0.55 0.86
8,000	15	0.10 0.25 0.50	6.8×10^{-4} 4.5×10^{-4} 3.5×10^{-4}	0.33 0.54 0.84
16,000	15	0.10 0.25 0.50	5.0×10^{-4} 3.4×10^{-4} 2.5×10^{-4}	0.24 0.41 0.59

Notes:

Test Date: 11 June 1991
Test Configuration (from top to bottom): load platen/60-mil geomembrane/ Amoco 4557 geotextile/geonet/60-mil geomembrane/base platen.
Confining Pressures and Pressure Durations as shown.

TABLE 2B

HYDRAULIC TRANSMISSIVITY TEST #2B RESULTS
AMOCO 4557 GEOTEXTILE WITH CLAYMAX

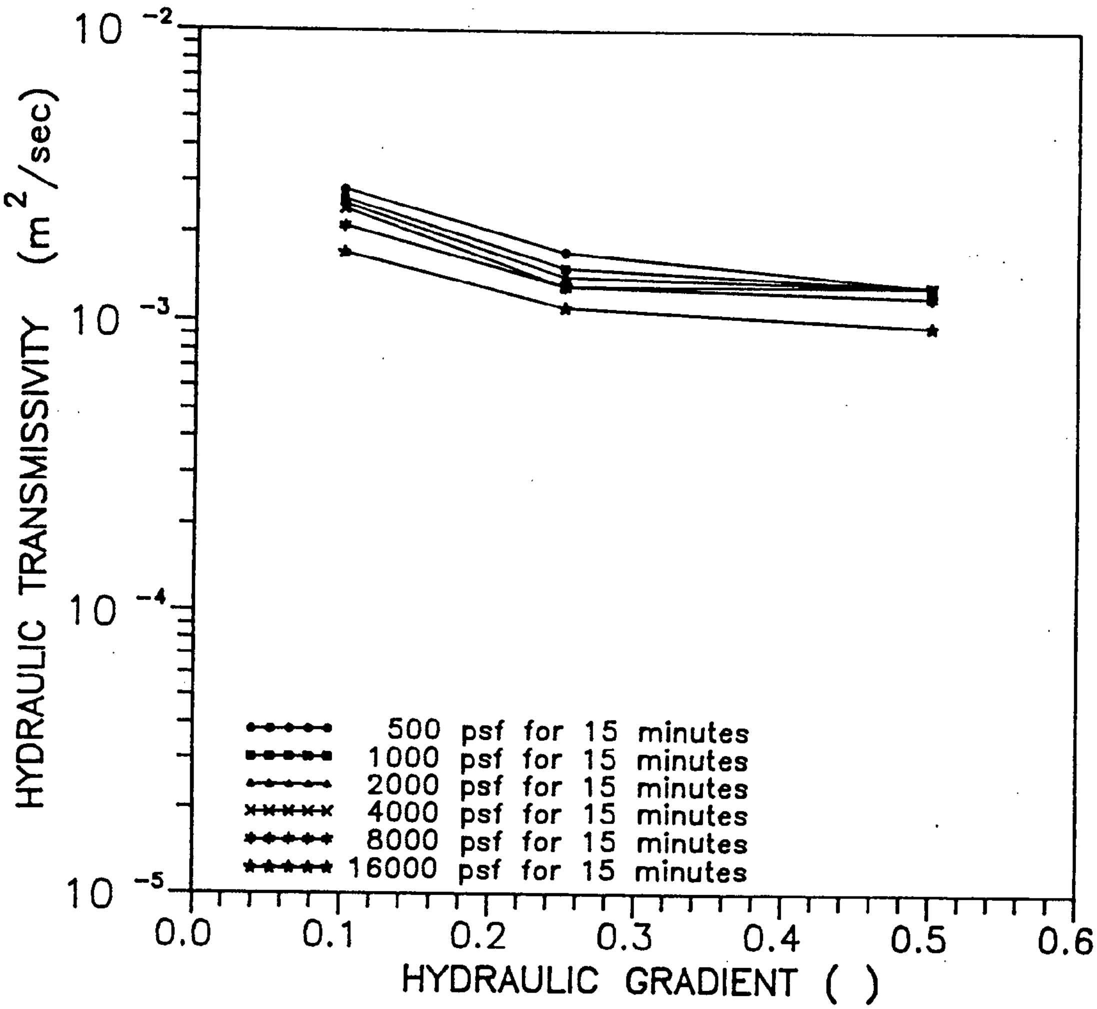
Confining Pressure (psf)	Duration of Pressure (days)	Hydraulic Gradient ( )	Hydraulic Transmissivity (m^2/s)	Unit Flowrate (gpm/ft)
500	2	0.10 0.25 0.50	1.1×10^{-3} 7.4×10^{-4} 5.5×10^{-4}	0.55 0.90 1.32
1,000	2	0.10 0.25 0.50	7.7×10^{-4} 5.3×10^{-4} 4.9×10^{-4}	0.37 0.64 1.19
2,000	2	0.10 0.25 0.50	6.5×10^{-4} 4.9×10^{-4} 4.4×10^{-4}	0.31 0.59 1.06
4,000	2	0.10 0.25 0.50	6.4×10^{-4} 4.3×10^{-4} 4.3×10^{-4}	0.31 0.52 1.03
8,000	7	0.10 0.25 0.50	7.0×10^{-4} 4.9×10^{-4} 3.8×10^{-4}	0.34 0.59 0.92
16,000	7	0.10 0.25 0.50	1.2×10^{-4} 9.3×10^{-5} 7.8×10^{-5}	0.06 0.11 0.19

Notes:

Test Starting Date: 12 June 1991
Test Configuration (from top to bottom): load platen/60-mil geomembrane/ Claymax/Amoco 4557 geotextile/geonet/60-mil geomembrane/base platen.
Confining Pressures and Pressure Durations as shown.
Hydration of the Claymax specimen occurred under the 500 psf confining pressure for 2 days.

308913

HYDRAULIC TRANSMISSIVITY TEST #1A RESULTS
TYPAR 3601 GEOTEXTILE WITHOUT CLAYMAX

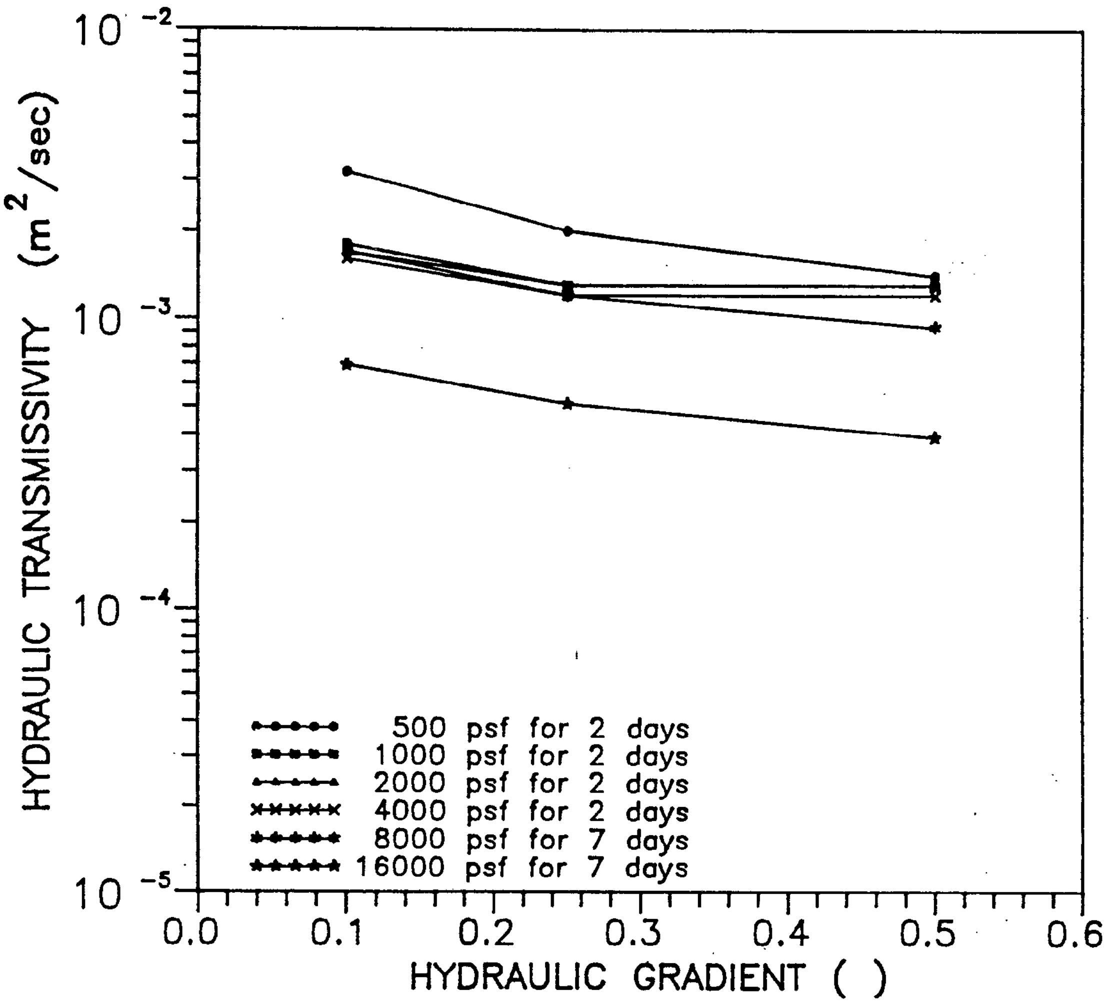


Notes:
Test Date: 15 May 1991
Test Configuration (from top to bottom): load platen/ 60-mil geomembrane/Typar 3601 geotextile/ geonet/60-mil geomembrane/base platen.
Confining Pressures and Pressure Durations are as shown.

GeoSyntec Consultants
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	IA-1
PROJECT NO.	GL3024
DOCUMENT NO.	GEL91179
PAGE NO.	

HYDRAULIC TRANSMISSIVITY TEST #1B RESULTS
TYPAR 3601 GEOTEXTILE WITH CLAYMAX



Notes:
Test Starting Date: 15 May 1991
Test Configuration (from top to bottom): load platen/ 60-mil geomembrane/Claymax/Typar 3601 geotextile/ geonet/60-mil geomembrane/base platen.
Confining Pressures and Pressure Durations are as shown.

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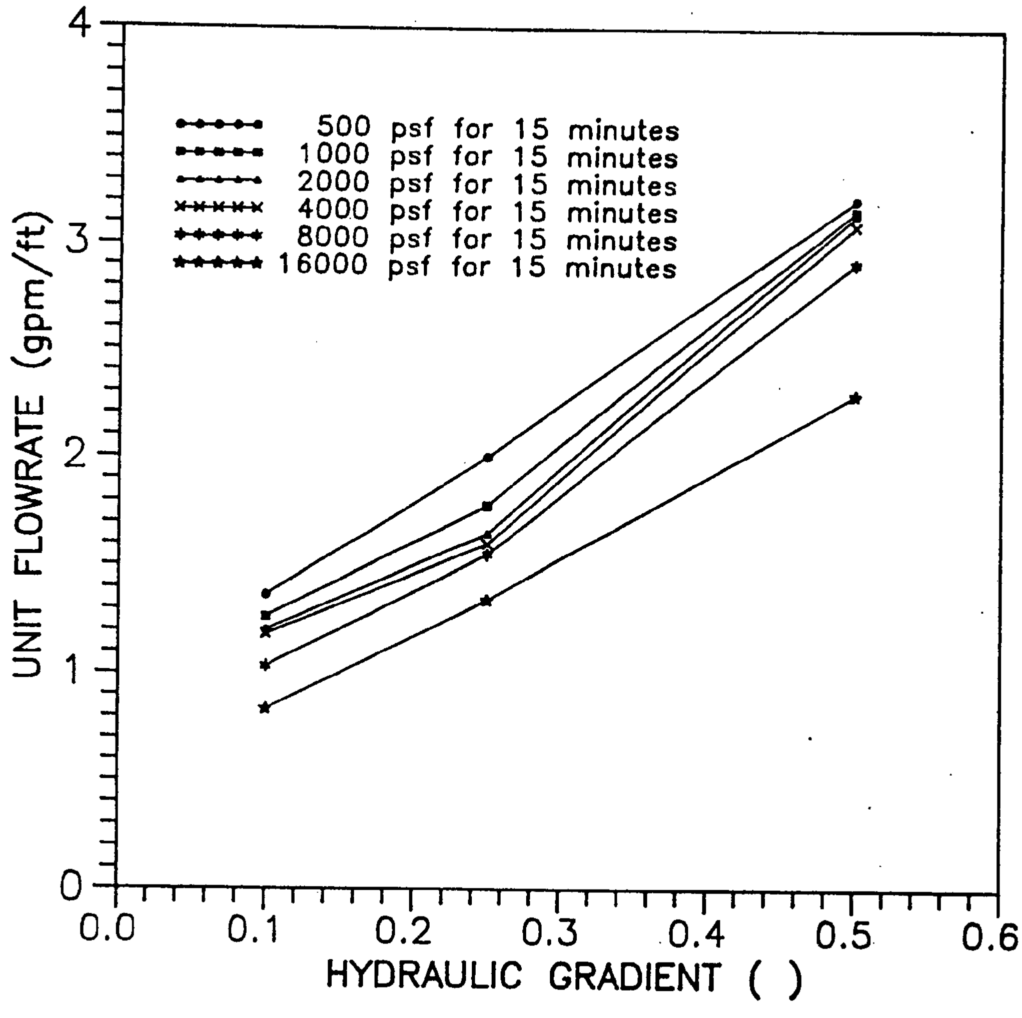
FIGURE NO.	1B-1
PROJECT NO.	GL3024
DOCUMENT NO.	GEL91179
PAGE NO.	

HYDRAULIC TRANSMISSIVITY TEST #1A RESULTS
TYPAR 3601 GEOTEXTILE WITHOUT CLAYMAX



Notes:
Test Date: 15 May 1991
Test Configuration (from top to bottom): load platen/ 60-mil geomembrane/Typar 3601 geotextile/ geonet/60-mil geomembrane/base platen.
Confining Pressures and Pressure Durations are as shown.

GEOSYNTEC CONSULTANTS
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FIGURE NO.	1A-2
PROJECT NO.	GL3024
DOCUMENT NO.	GEL91179
PAGE NO.	

The total stress peak and residual shear strengths derived from the plotted test results are summarized in Table 3. These strengths were plotted on a graph of shear stress versus the corresponding normal stress to evaluate a total stress peak or residual strength envelope. A best fit straight line was drawn through the five data points from each test series to obtain a total peak stress or residual stress interface friction angle and adhesion. The interface friction angles and adhesions derived from the plotted test results are summarized in Table 4. The summary plots of shear stress versus the corresponding normal stress are also presented for each test series in Appendix A.

Shear stress versus normal stresses plots reported in Appendix A included a best fit straight line trough all of the data points. Clem requested that additional plots be prepared for each test series. For each test series, a best fit straight line was drawn through: (i) data points corresponding to 50, 200, and 400 psf (2, 10, 20 kPa) normal stresses and herein refered to as low normal stress conditions; and (ii) data points corresponding to 400, 700, and 1000 psf (20, 34, and 49 kPa) normal stresses and herein refered to as high normal stress conditions. The summary plots are presented in Appendix B. The peak and residual total stress interface friction angles and adhesions derived from each set of the plotted test results are summarized in Table 5 (low normal stress conditions) and Table 6 (high normal stress conditions).

For all tests it is noted that the reported adhesion is the shear stress axis intercept of the best fit straight line drawn through the plotted test data points on the shear stress versus normal stress plot. This value may not be the "true adhesion" of the interface and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

4.3 Closure

The reported results were obtained from tests conducted on the samples provided by Clem. The testing was performed in accordance with general engineering testing standards and requirements. This testing report is submitted for the exclusive use of Clem.

TABLE 1

SUMMARY OF GENERAL TESTING CONDITIONS
INTERFACE DIRECT SHEAR TESTING
JAMES CLEM CORPORATION
SELECT CLAYMAX SHEAR-PRO BENTONITE COMPOSITE INTERFACES

Test Series Number	Interface Tested[1]	Soaking/Hydration Stress (psf)	Time for Soaking/Hydration (hours)	Normal Stresses (psf)[2]	Rate of Shear (in./min)
1	Dry Claymax Shear-Pro Prototype No. 3 Bentonite Composite/60-mil NSC Textured HDPE Geomembrane	0	0	50, 200, 400, 700, and 1000	0.04
2	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/60-mil NSC Textured HDPE Geomembrane	100	24	50, 200, 400, 700, and 1000	0.04
3	Hydrated Claymax Shear-Pro Prototype No. 4 Bentonite Composite/60-mil NSC Textured HDPE Geomembrane	100	24	50, 200, 400, 700, and 1000	0.04
4	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/60-mil NSC Textured VLDPE Geomembrane	100	24	50, 200, 400, 700, and 1000	0.04
5	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/60-mil NSC Smooth HDPE Geomembrane	100	24	50, 200, 400, 700, and 1000	0.04
6	Soaked AASHTO No. 57 Stone/Claymax Shear-Pro Prototype No. 4 Bentonite Composite	100	24	50, 200, 400, 700, and 1000	0.04
7	Soaked Sandy Clay Soil/Claymax Shear-Pro 500SP Bentonite Composite	100	24	50, 200, 400, 700, and 1000	0.04

NOTES: (1) For Test Series 1, the bentonite composite-geomembrane interface was tested under dry conditions. For Test Series 2, 4, 5, 6, and 7, the bentonite composite-geomembrane or soil-bentonite composite interface was soaked in water for 24 hours under a normal stress of 100 psf prior to shearing. For Test Series 3, the bentonite composite was hydrated in water for 24 hours under a normal stress of 100 psf and then placed in the shear box.

(2) Test specimens was sheared immediately after application of the normal stress.

TABLE 2

SUMMARY OF ACTUAL INTERFACE DIRECT SHEAR
TEST EQUIPMENT AND CONDITIONS
JAMES CLEM CORPORATION
SELECT CLAYMAX SHEAR-PRO BENTONITE COMPOSITE INTERFACES

Test[1] Series Number	Shear Box Size	ω_{ci} (%)	Soaking/Hydration Stress (psf)	Time of Soaking/Hydration (Hours)	ω_{cf} (%)	Normal Stress (psf)	Displacement Rate (in./min)
1	12" x 12"	18.1	0	0	18.1	50	0.04
		18.0	0	0	18.0	200	0.04
		18.2	0	0	18.2	400	0.04
		18.3	0	0	18.3	700	0.04
		18.2	0	0	18.2	1000	0.04
2	12" x 12"	18.5	100	24	141.2	50	0.04
		18.4	100	24	140.8	200	0.04
		18.3	100	24	140.5	400	0.04
		18.3	100	24	141.5	700	0.04
		18.1	100	24	141.7	1000	0.04
3	12" x 12"	18.5	100	24	150.0	50	0.04
		18.7	100	24	149.5	200	0.04
		18.4	100	24	149.2	400	0.04
		18.6	100	24	149.7	700	0.04
		19.1	100	24	149.5	1000	0.04
4	12" x 12"	18.3	100	24	141.1	50	0.04
		18.8	100	24	141.7	200	0.04
		18.5	100	24	141.5	400	0.04
		18.2	100	24	140.6	700	0.04
		18.6	100	24	141.2	1000	0.04

TABLE 2 (continued)

5	12" x 12"	18.2 18.4 18.1 18.6 18.7	100 100 100 100 100	24 24 24 24 24	140.1 140.7 140.3 141.1 140.5	50 200 400 700 1000	0.04 0.04 0.04 0.04 0.04
6	12" x 12"	18.2 18.3 18.3 18.5 18.7	100 100 100 100 100	24 24 24 24 24	144.6 145.8 145.3 146.0 146.2	50 200 400 700 1000	0.04 0.04 0.04 0.04 0.04
7	12" x 12"	18.5 18.2 18.4 18.3 18.1	100 100 100 100 100	24 24 24 24 24	141.5 140.4 141.7 140.9 141.1	50 200 400 700 1000	0.04 0.04 0.04 0.04 0.04

Notes: [1] For Test Series 1, the bentonite composite-geomembrane interface was tested under dry conditions. For Test Series 2, 4, 5, 6, and 7, the bentonite composite-geomembrane or soil-bentonite composite interface was soaked in water for 24 hours under a normal stress of 100 psf prior to shearing. For Test Series 3, the bentonite composite was hydrated in water for 24 hours under a normal stress of 100 psf and then placed in the shear box.

[2] ω_{ci} refers to initial moisture content of Claymax Shear-Pro bentonite composite specimen.
ω_{cf} refers to final moisture content of Claymax Shear-Pro bentonite composite specimen.

TABLE 3

INTERFACE DIRECT SHEAR TEST RESULTS
MEASURED PEAK AND RESIDUAL TOTAL SHEAR STRENGTHS
JAMES CLEM CORPORATION
SELECT CLAYMAX SHEAR-PRO BENTONITE COMPOSITE INTERFACES

Test[1] Series Number	Normal[2] Stress (psf)	Measured Peak Shear Strength (psf)	Measured Residual Shear Strength (psf)	Reference Appendix Figure Number
1	50 200 400 700 1000	46 142 286 469 615	39 115 232 332 435	A-1 and A-2
2	50 200 400 700 1000	42 149 283 391 597	34 127 205 293 391	A-3 and A-4
3	50 200 400 700 1000	12 60 171 273 342	12 60 160 244 273	A-5 and A-6
4	50 200 400 700 1000	49 186 313 459 625	39 122 225 361 449	A-7 and A-8
5	50 200 400 700 1000	20 51 86 155 203	20 51 86 155 203	A-9 and A-10
6	50 200 400 700 1000	80 125 350 640 935	50 115 320 590 800	A-11 and A-12
7	50 200 400 700 1000	60 170 225 435 505	60 170 225 435 505	A-13 and A-14

Notes: (1) For Test Series 1, the bentonite composite-geomembrane interface was tested under dry conditions. For Test Series 2, 4, 5, 6, and 7, the bentonite composite-geomembrane or soil-bentonite composite interface was soaked in water for 24 hours under a normal stress of 100 psf prior to shearing. For Test Series 3, the bentonite composite was hydrated in water for 24 hours under a normal stress of 100 psf and then placed in the shear box.

(2) Test specimens were sheared immediately after application of the normal stress.

TABLE !

INTERFACE DIRECT SHEAR TEST RESULTS
LOW NORMAL STRESS SHEAR STRENGTH PARAMETERS
(for Normal Stresses of 50 to 400 psf)
JAMES CLEM CORPORATION
SELECT CLAYMAX SHEAR-PRO BENTONITE COMPOSITE INTERFACES

Test Series Number	Interface Tested[1,2]	Peak Strength		Residual Strength	
		Friction Angle	Adhesion[3] (psf)	Friction Angle	Adhesion[3] (psf)
1	Dry Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Textured HDPE Geomembrane	35°	10	29°	10
2	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Textured HDPE Geomembrane	35°	10	26°	20
3	Hydrated Claymax Shear-Pro Prototype No. 4 Bentonite Composite/60-mil NSC Textured HDPE Geomembrane	23°	0	22°	0
4	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Textured VLDPE Geomembrane	37°	20	28°	10
5	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Smooth HDPE Geomembrane	11°	10	11°	10
6	Soaked AASHTO No. 57 Stone/Claymax Shear-Pro Prototype No. 4 Bentonite Composite	39°	10	38°	0
7	Soaked Sandy Clay Soil/Claymax Shear-Pro 500SP Bentonite Composite	25°	50	25°	50

Notes:

[1] For Test Series 1, the bentonite composite-geomembrane interface was tested under dry conditions. For Test Series 2, 4, 5, 6, and 7, the bentonite composite-geomembrane or soil-bentonite composite interface was soaked in water for 24 hours under a normal stress of 100 psf prior to shearing. For Test Series 3, the bentonite composite was hydrated in water for 24 hours under a normal stress of 100 psf and then placed in the shear box.

[2] Test specimens were sheared immediately after application of the normal stress.

[3] The reported value of adhesion may not be the "true adhesion" of the interface and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

TABLE 4

INTERFACE DIRECT SHEAR TEST RESULTS
MEASURED TOTAL STRESS SHEAR STRENGTH PARAMETERS
(for Normal Stresses of 50 to 1000 psf)
JAMES CLEM CORPORATION
SELECT CLAYMAX SHEAR-PRO BENTONITE COMPOSITE INTERFACES

Test Series Number	Interface Tested[1,2]	Peak Strength		Residual Strength	
		Friction Angle	Adhesion[3] (psf)	Friction Angle	Adhesion[3] (psf)
1	Dry Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Textured HDPE Geomembrane	31°	25	22°	35
2	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Textured HDPE Geomembrane	27°	40	20°	40
3	Hydrated Claymax Shear-Pro Prototype No. 4 Bentonite Composite/60-mil NSC Textured HDPE Geomembrane	20°	5	16°	15
4	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Textured VLDPE Geomembrane	30°	50	24°	35
5	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Smooth HDPE Geomembrane	11°	10	11°	10
6	Soaked AASHTO No. 57 Stone/Claymax Shear-Pro Prototype No. 4 Bentonite Composite	43°	0	39°	0
7	Soaked Sandy Clay Soil/Claymax Shear-Pro 500SP Bentonite Composite	26°	55	26°	55

Notes: [1] For Test Series 1, the bentonite composite-geomembrane interface was tested under dry conditions. For Test Series 2, 4, 5, 6, and 7, the bentonite composite-geomembrane or soil-bentonite composite interface was soaked in water for 24 hours under a normal stress of 100 psf prior to shearing. For Test Series 3, the bentonite composite was hydrated in water for 24 hours under a normal stress of 100 psf and then placed in the shear box.

[2] Test specimens were sheared immediately after application of the normal stress.

[3] The reported value of adhesion may not be the "true adhesion" of the interface and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

..LE 6

INTERFACE DIRECT SHEAR TEST RESULTS
HIGH NORMAL STRESS SHEAR STRENGTH PARAMETERS
(for Normal Stresses of 400 to 1000 psf)
JAMES CLEM CORPORATION
SELECT CLAYMAX SHEAR-PRO BENTONITE COMPOSITE INTERFACES

2

Test Series Number	Interface Tested[1,2]	Peak Strength		Residual Strength	
		Friction Angle	Adhesion[3] (psf)	Friction Angle	Adhesion[3] (psf)
1	Dry Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Textured HDPE Geomembrane	28°	70	19°	100
2	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Textured HDPE Geomembrane	24°	100	17°	80
3	Hydrated Claymax Shear-Pro Prototype No. 4 Bentonite Composite/60-mil NSC Textured HDPE Geomembrane	16°	60	11°	90
4	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Textured VLDPE Geomembrane	27°	100	20°	80
5	Soaked Claymax Shear-Pro Prototype No. 3 Bentonite Composite/ 60-mil NSC Smooth HDPE Geomembrane	11°	10	11°	10
6	Soaked AASHTO No. 57 Stone/Claymax Shear-Pro Prototype No. 4 Bentonite Composite	43°	0	39°	0
7	Soaked Sandy Clay Soil/Claymax Shear-Pro 500SP Bentonite Composite	25°	60	25°	60

Notes: [1] For Test Series 1, the bentonite composite-geomembrane interface was tested under dry conditions. For Test Series 2, 4, 5, 6, and 7, the bentonite composite-geomembrane or soil-bentonite composite interface was soaked in water for 24 hours under a normal stress of 100 psf prior to shearing. For Test Series 3, the bentonite composite was hydrated in water for 24 hours under a normal stress of 100 psf and then placed in the shear box.

[2] Test specimens were sheared immediately after application of the normal stress.

[3] The reported value of adhesion may not be the "true adhesion" of the interface and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

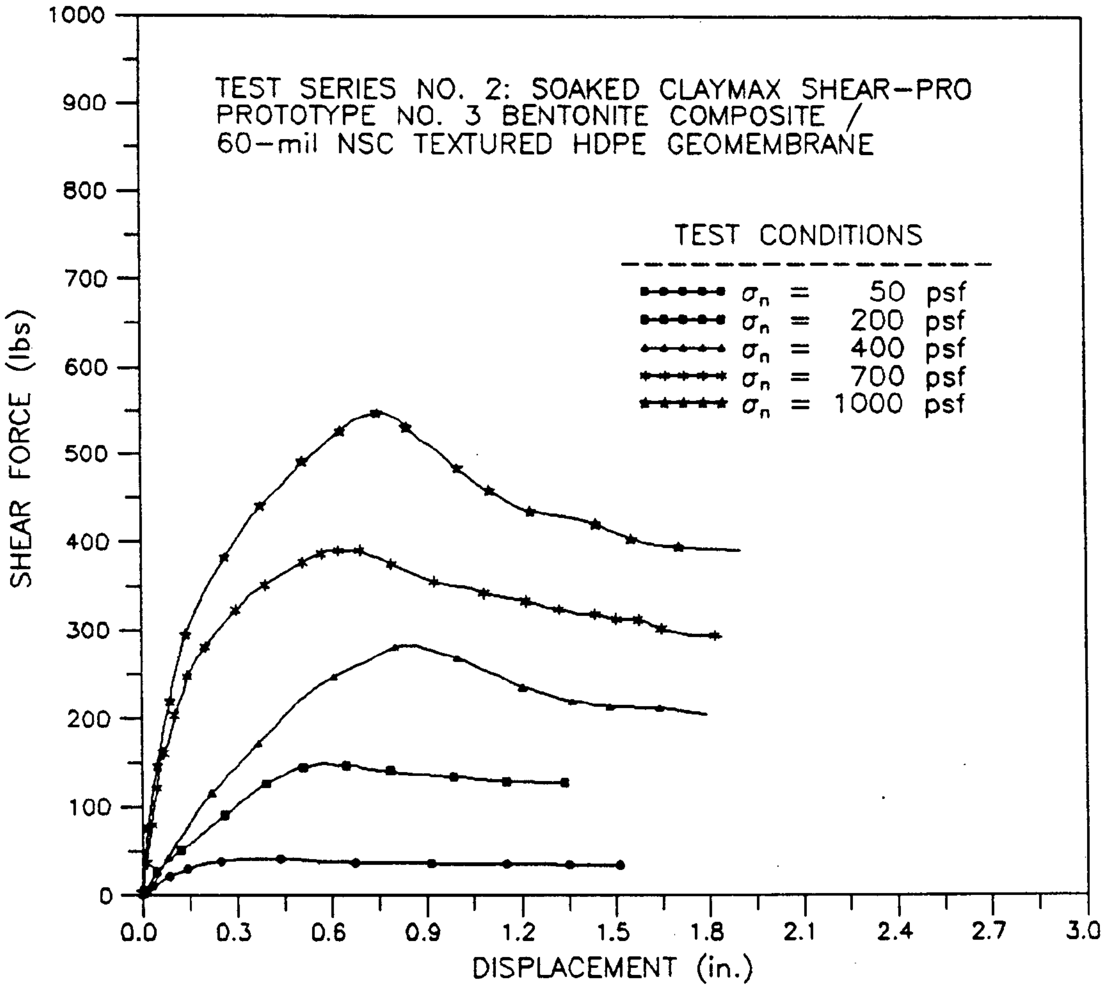
GL3160/GEL92211

APPENDIX A

INTERFACE DIRECT SHEAR TEST DATA
(TEST SERIES NUMBERS 1 THROUGH 7)

- SHEAR FORCE VERSUS DISPLACEMENT DATA
- SHEAR STRESS VERSUS NORMAL STRESS DATA FOR TESTS CONDUCTED AT 50 TO 1000 PSF NORMAL STRESS

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING

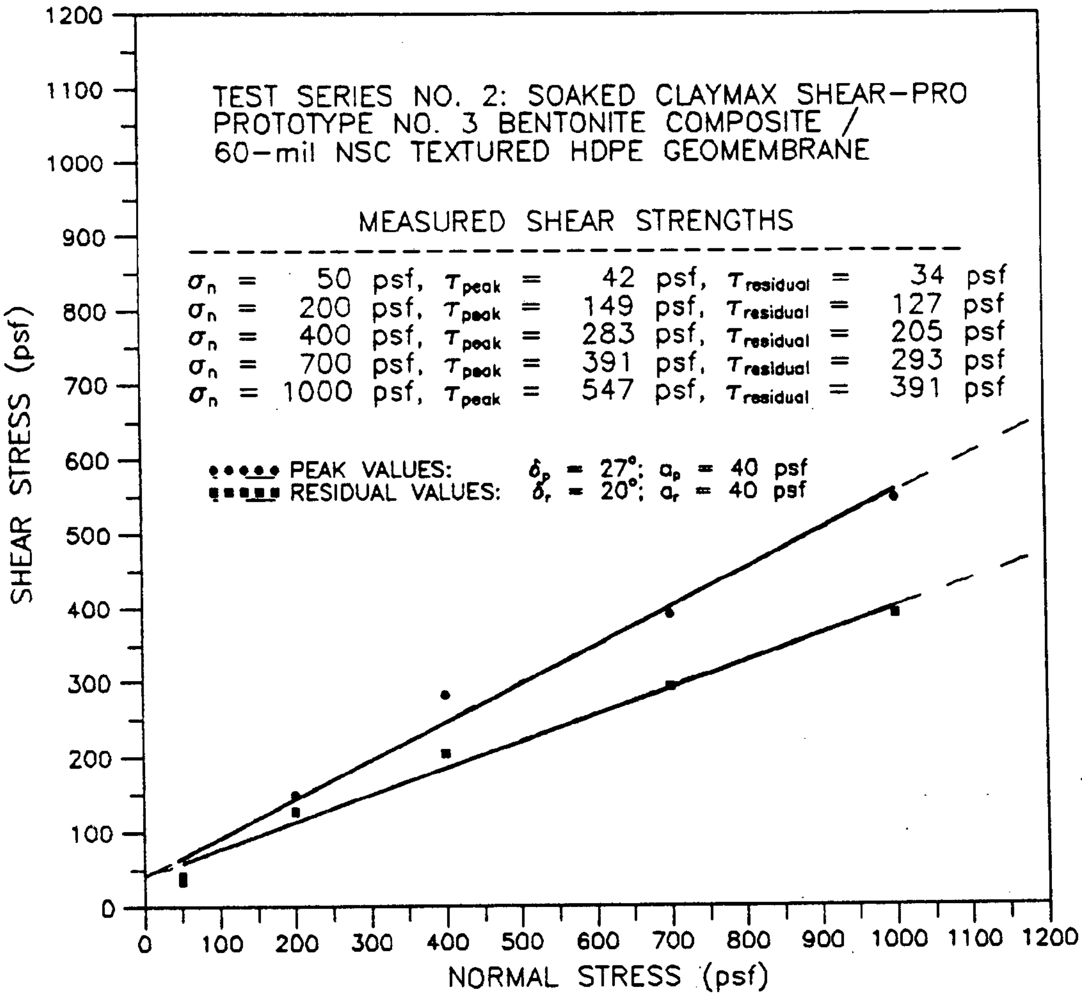


NOTE: The shear box size was 12 in. by 12 in. (300 mm by 300 mm), and the contact area remained constant throughout the entire test.

DATE TESTED: MARCH 1992

GeoSyntec Consultants GEOMECHANICS AND ENVIRONMENTAL LABORATORY	FIGURE NO.	A-3
	PROJECT NO.	GL3160
	DOCUMENT NO.	GEL92211
	PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING

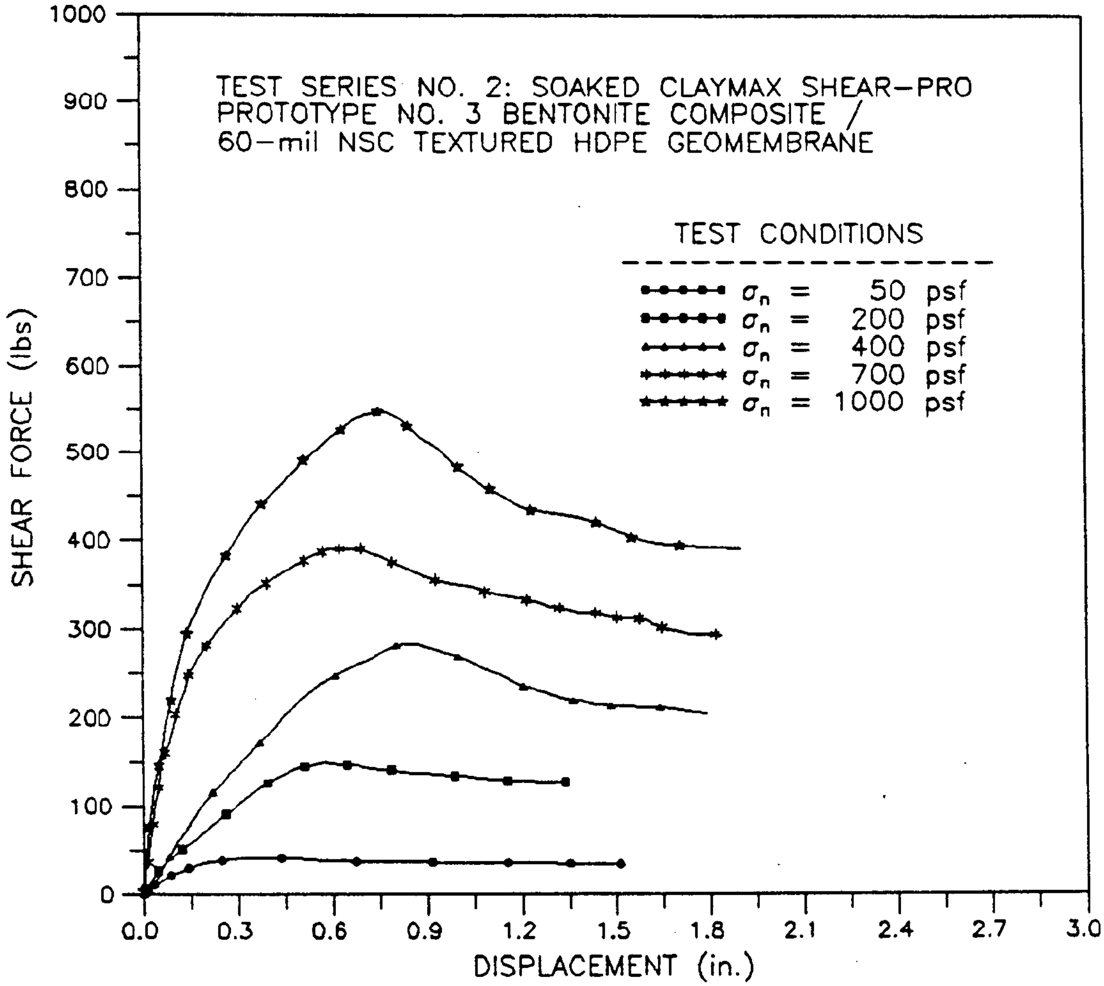


NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MARCH 1992

GEOSYNTEC CONSULTANTS GEOMECHANICS AND ENVIRONMENTAL LABORATORY	
FIGURE NO.	A-4
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING

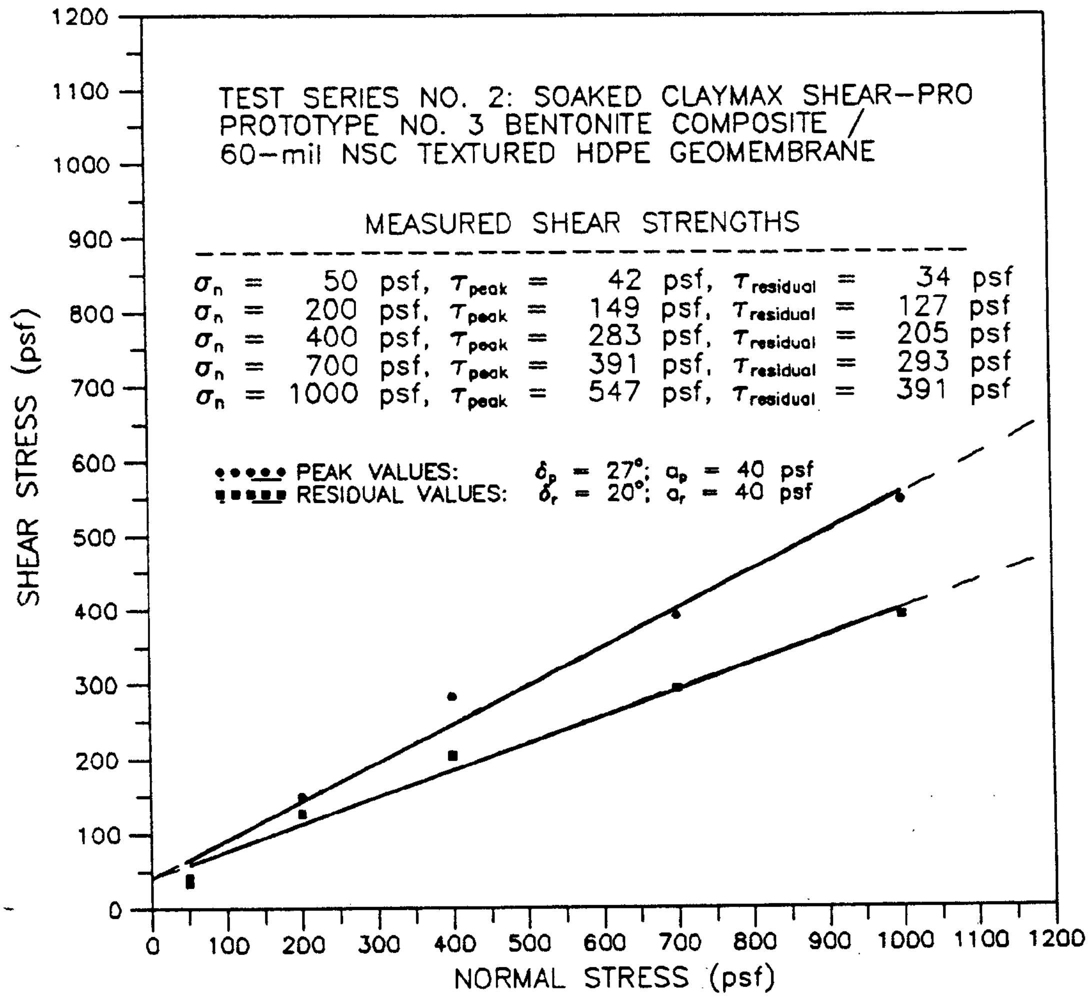


NOTE: The shear box size was 12 in. by 12 in. (300 mm by 300 mm), and the contact area remained constant throughout the entire test.

DATE TESTED: MARCH 1992

GEOSYNTEC CONSULTANTS GEOMECHANICS AND ENVIRONMENTAL LABORATORY	FIGURE NO.	A-3
	PROJECT NO.	GL3160
	DOCUMENT NO.	GEL92211
	PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



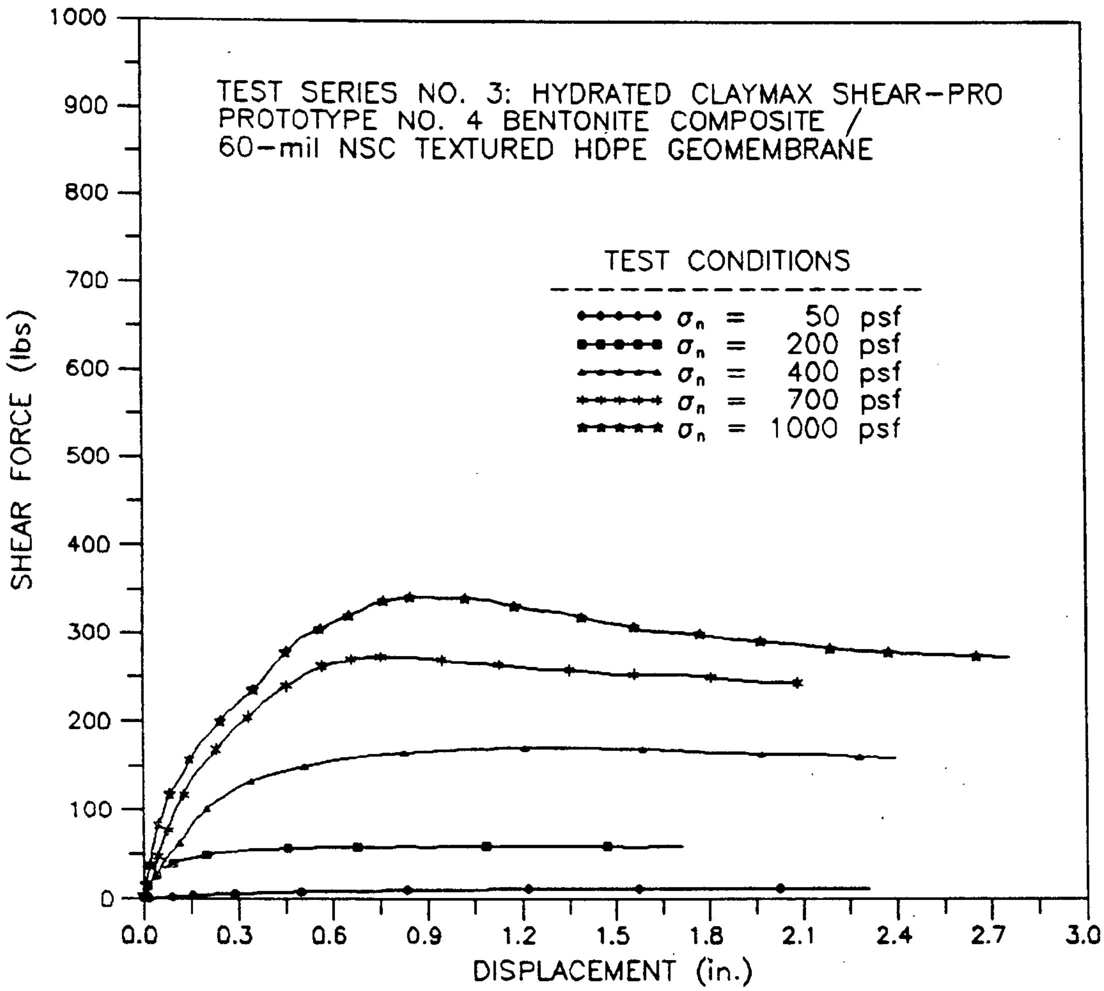
NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MARCH 1992

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	A-4
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The shear box size was 12 in. by 12 in. (300 mm by 300 mm), and the contact area remained constant throughout the entire test.

DATE TESTED: APRIL 1992

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	A-5
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

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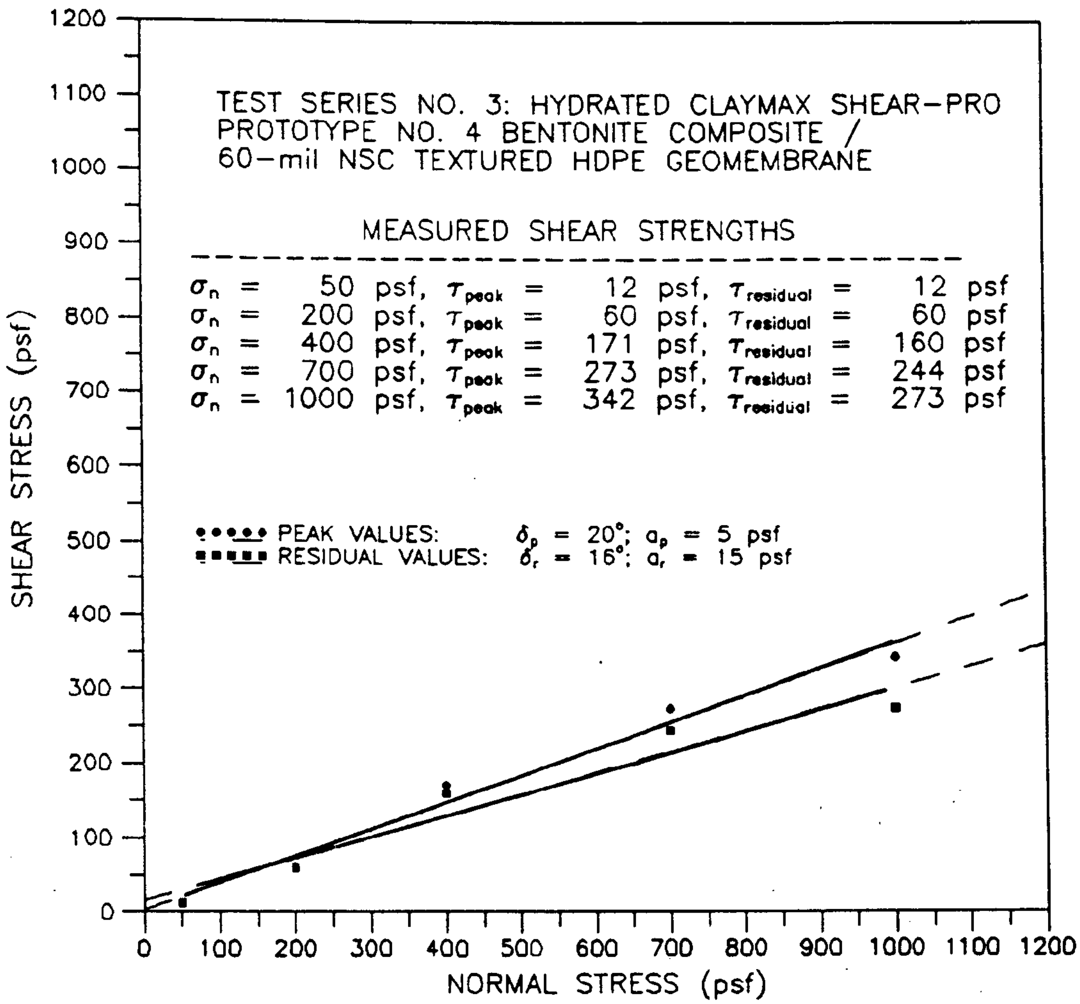
JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



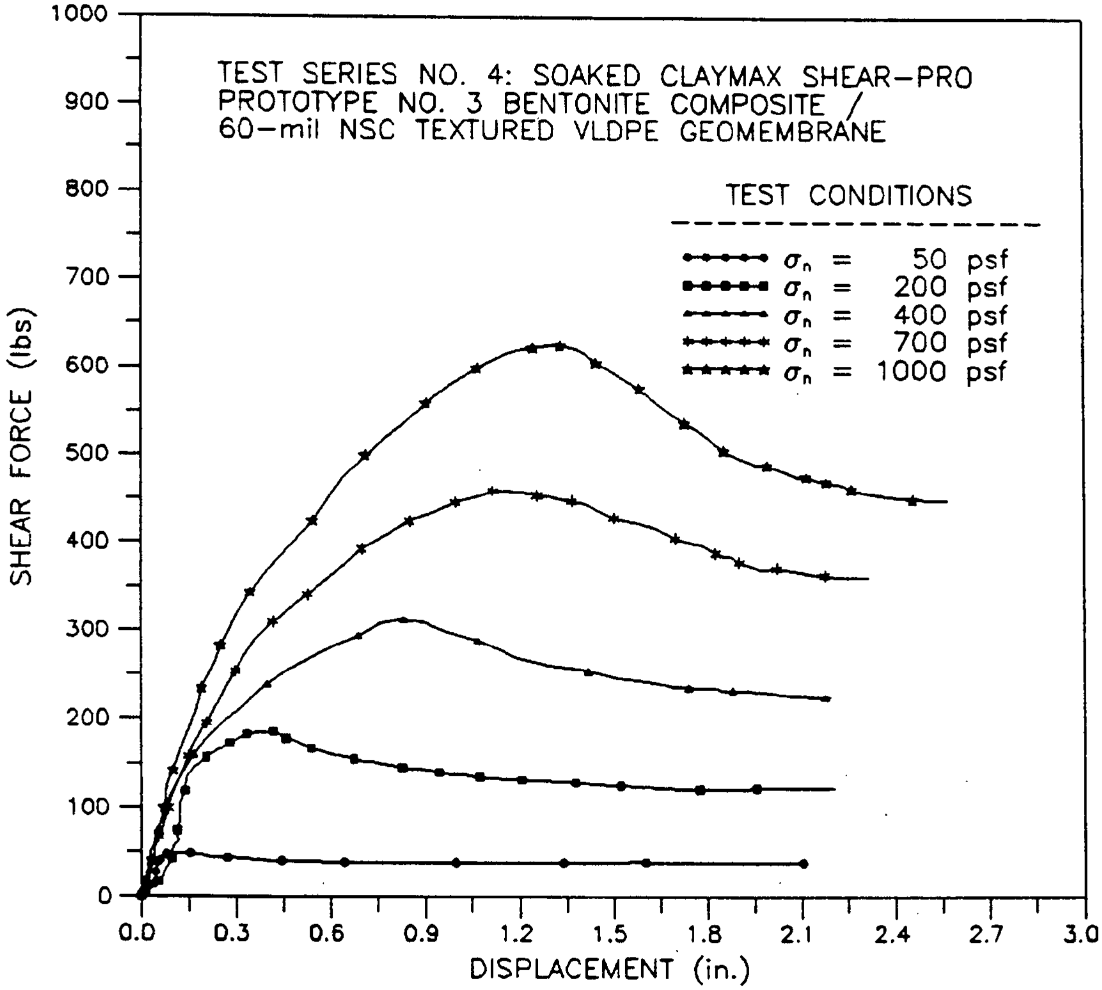
NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: APRIL 1992

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	A-6
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The shear box size was 12 in. by 12 in. (300 mm by 300 mm), and the contact area remained constant throughout the entire test.

DATE TESTED: MARCH 1992

GeoSyntec Consultants
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	A-7
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

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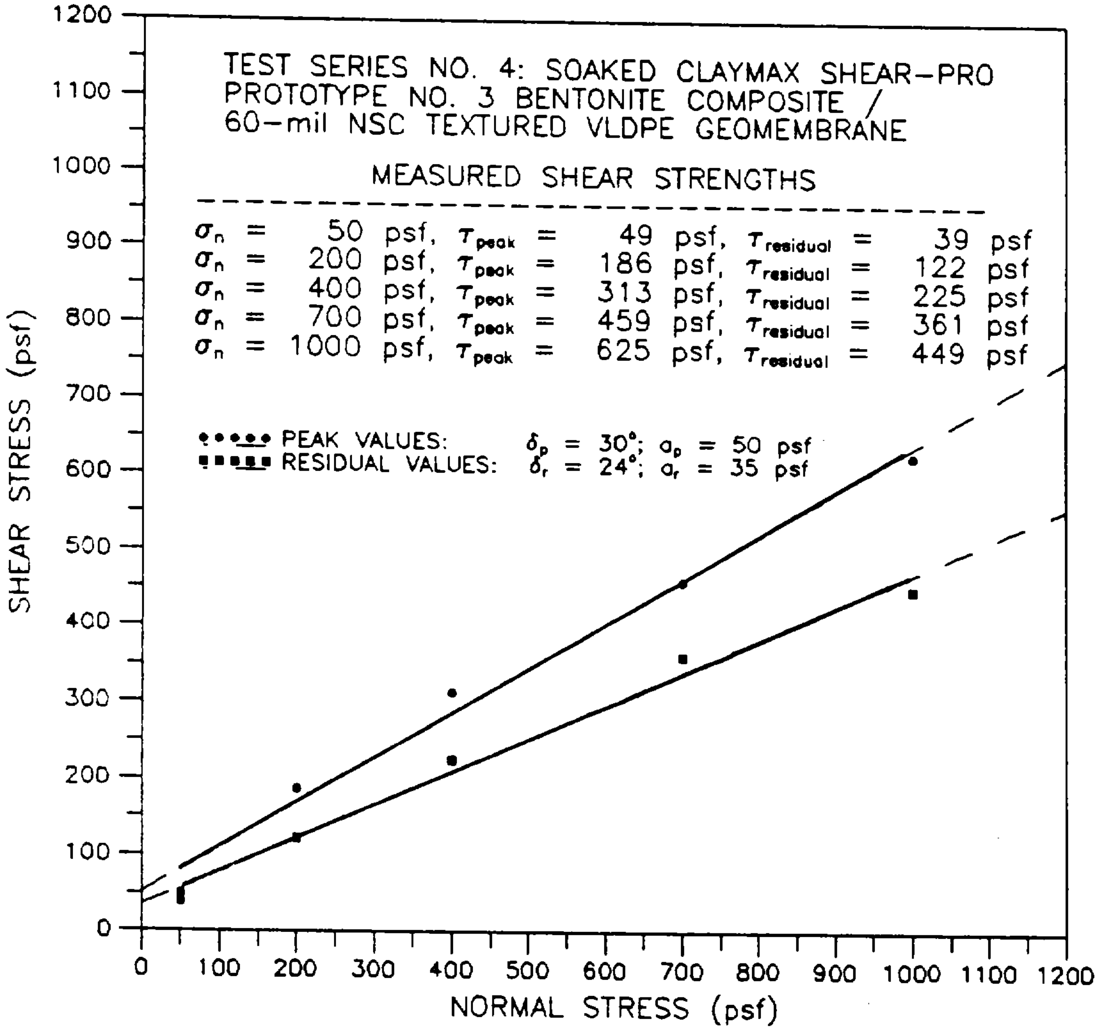
JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MARCH 1992

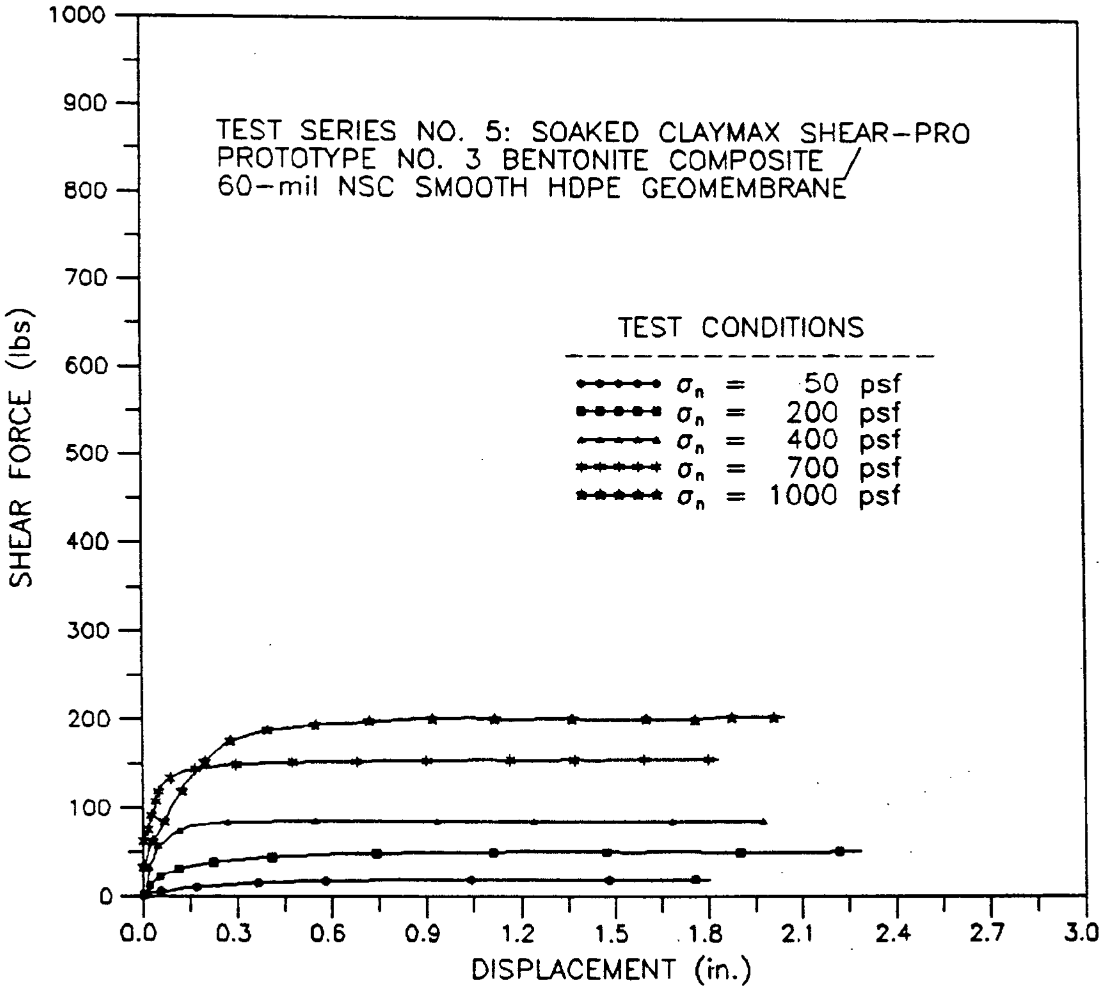
GEOSYNTEC CONSULTANTS

GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	A-8
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

308913

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The shear box size was 12 in. by 12 in. (300 mm by 300 mm), and the contact area remained constant throughout the entire test.

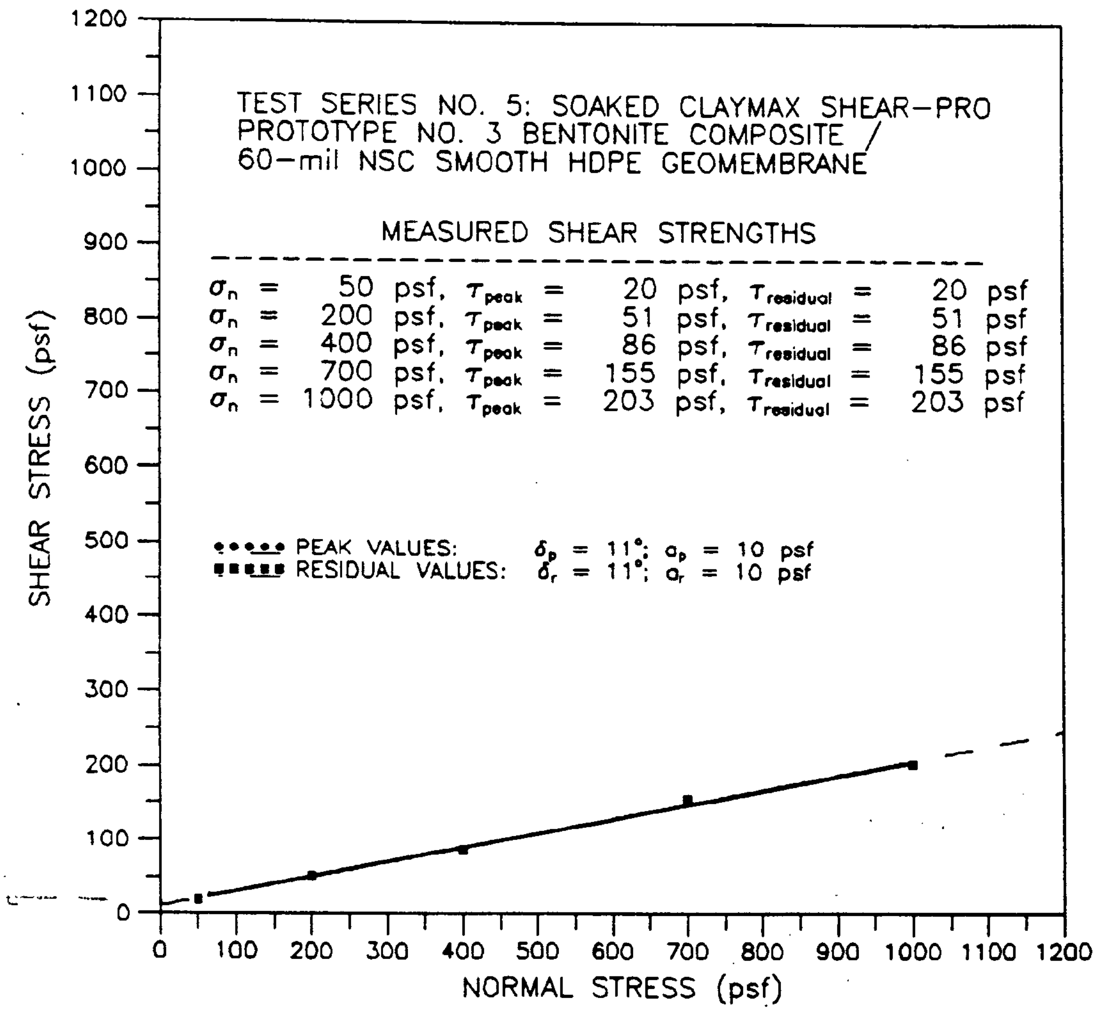
DATE TESTED: APRIL 1992

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	A-9
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

308913

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



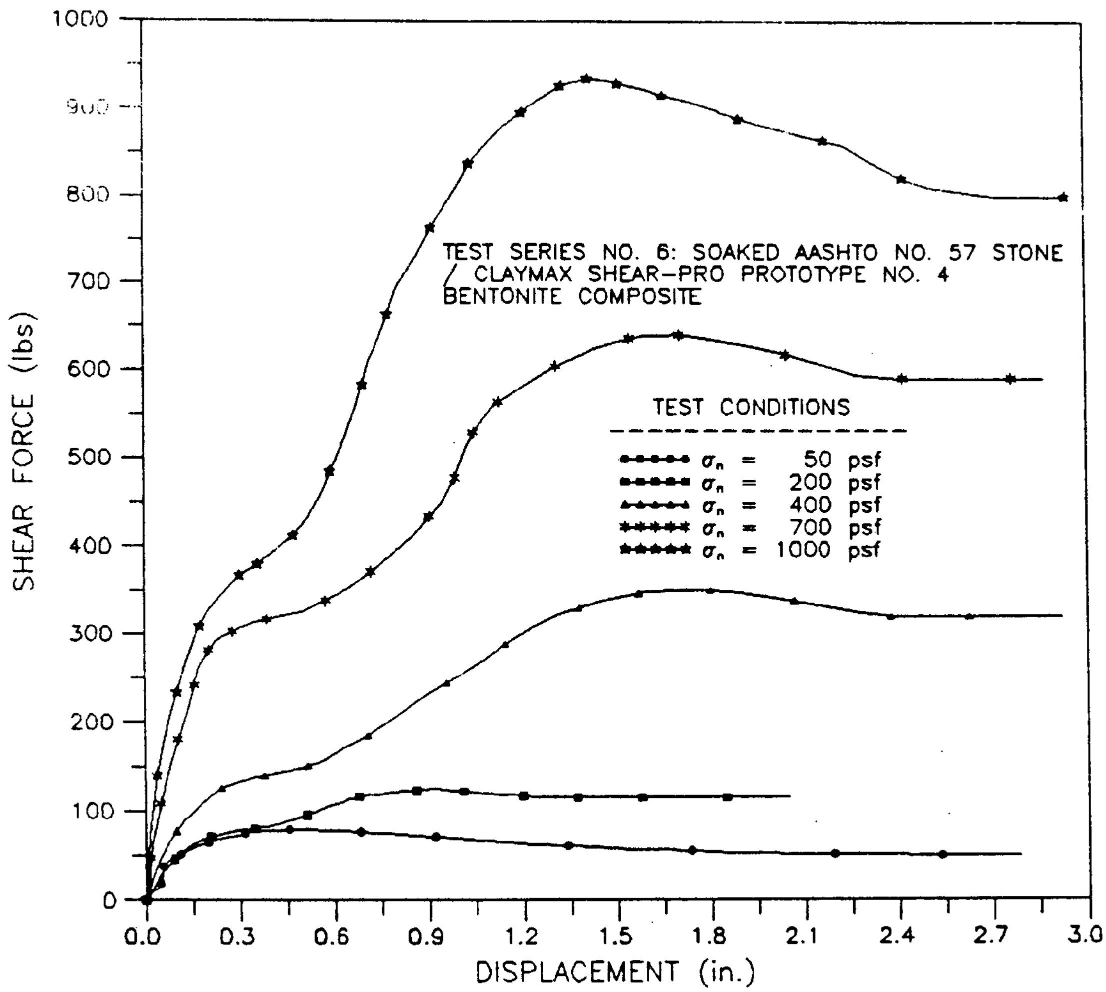
NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: APRIL 1992

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	A-10
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



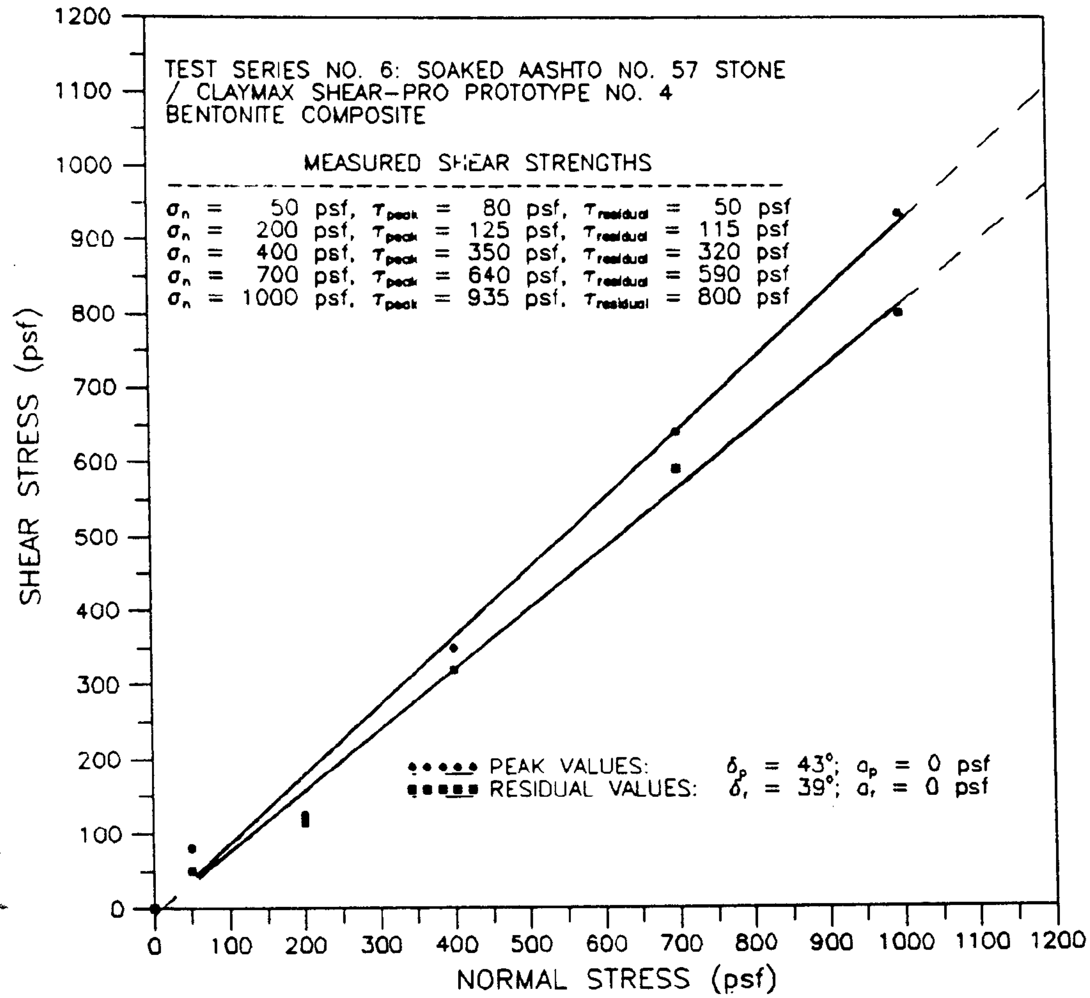
NOTE: The shear box size was 12 in. by 12 in. (300 mm by 300 mm), and the contact area remained constant throughout the entire test.

DATE TESTED: MAY 1992

GEOSYNTEC CONSULTANTS GEOMECHANICS AND ENVIRONMENTAL LABORATORY	FIGURE NO.	A-11
	PROJECT NO.	GL3160
	DOCUMENT NO.	GEL92211
	PAGE NO.	

J08913

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



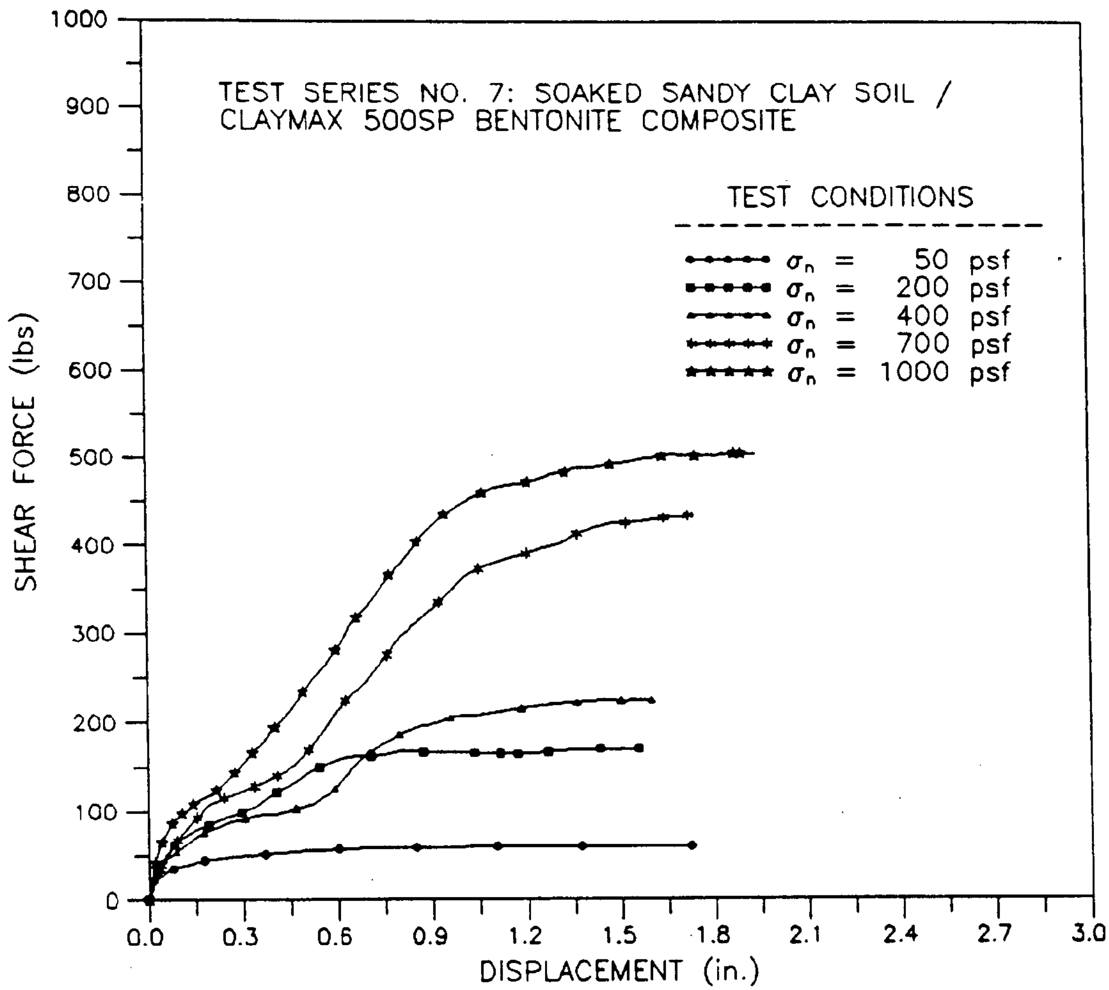
NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MAY 1992

FIGURE NO.	A-12
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



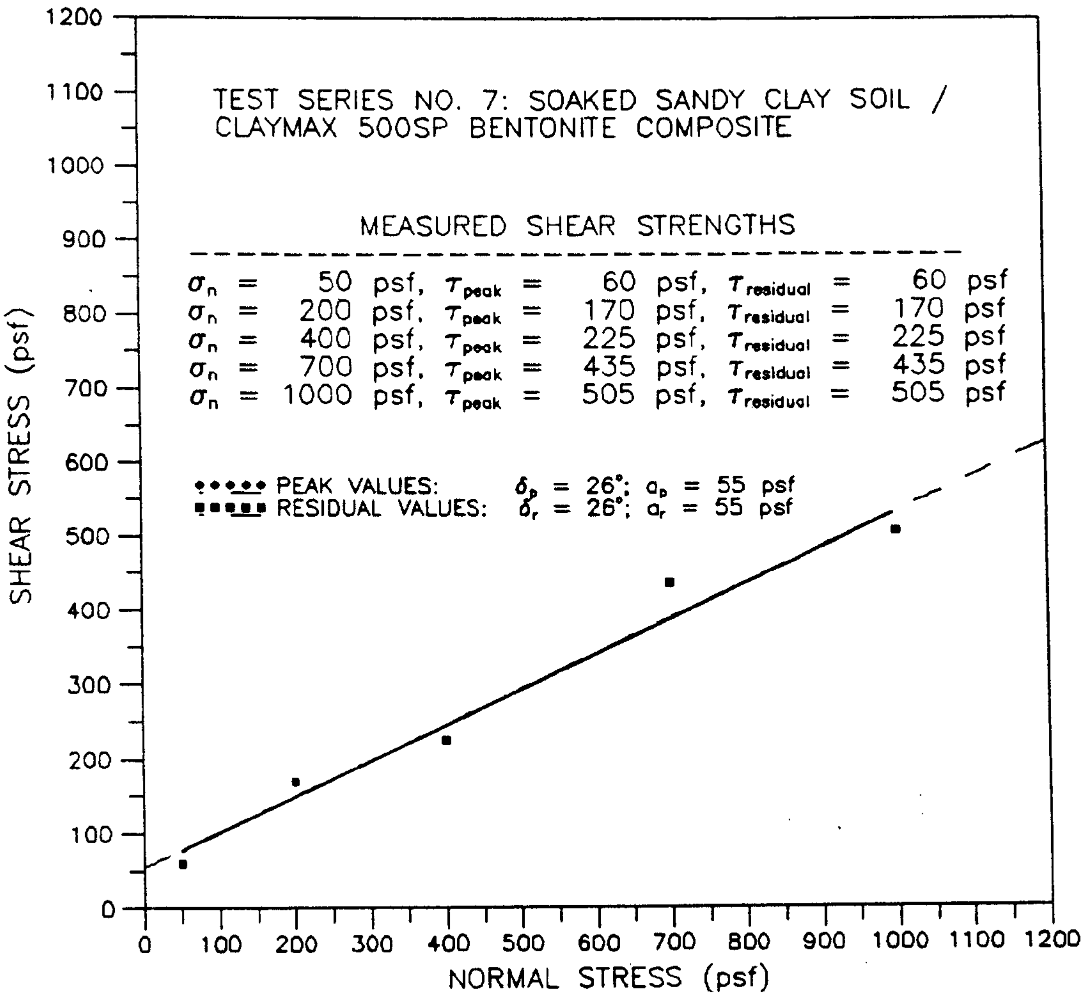
NOTE: The shear box size was 12 in. by 12 in. (300 mm by 300 mm), and the contact area remained constant throughout the entire test.

DATE TESTED: NOVEMBER 1992

FIGURE NO.	A-13
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: NOVEMBER 1992

FIGURE NO.	A-14
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

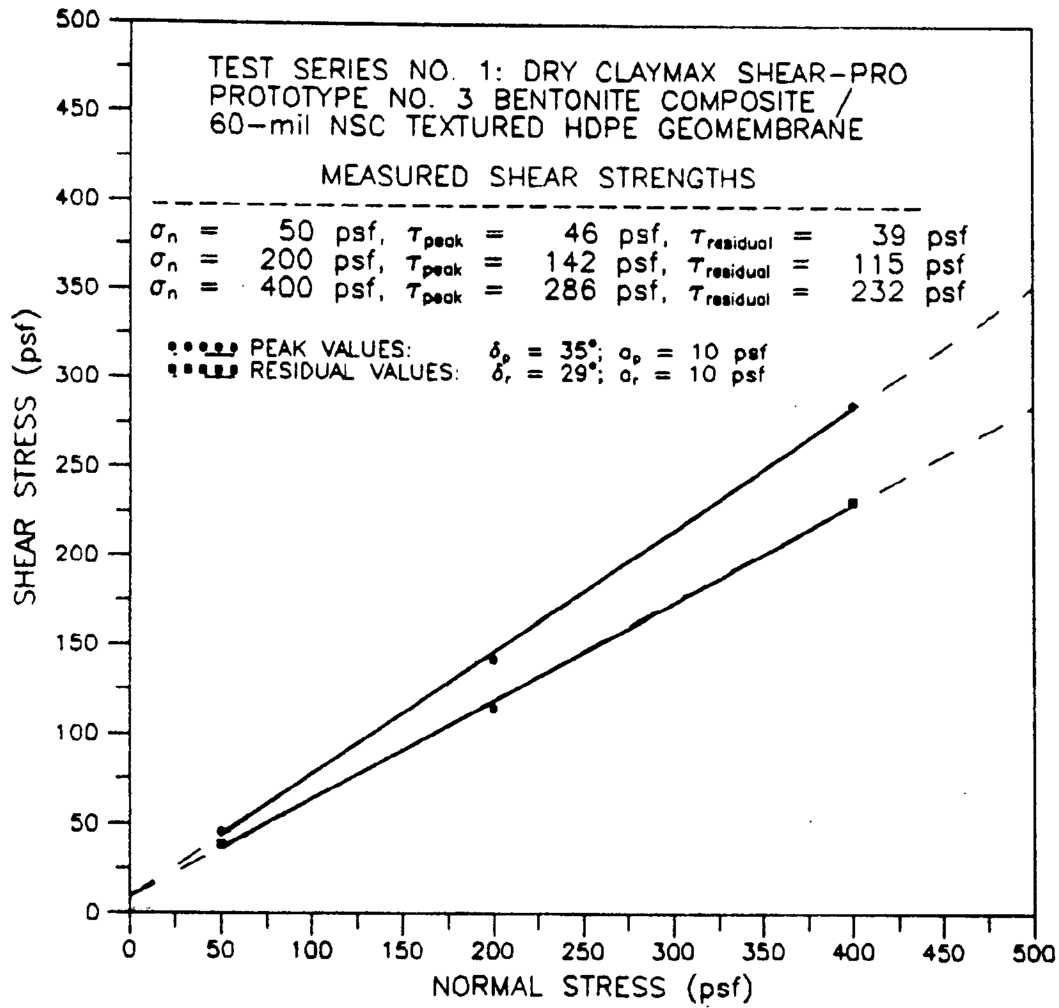
APPENDIX B

INTERFACE DIRECT SHEAR TEST RESULTS
(TEST SERIES NUMBERS 1 THROUGH 7)

- SHEAR FORCE VERSUS DISPLACEMENT RESULTS
  - 50 TO 400 PSF NORMAL STRESS
  - 400 TO 1000 PSF NORMAL STRESS

J08913

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MARCH 1992

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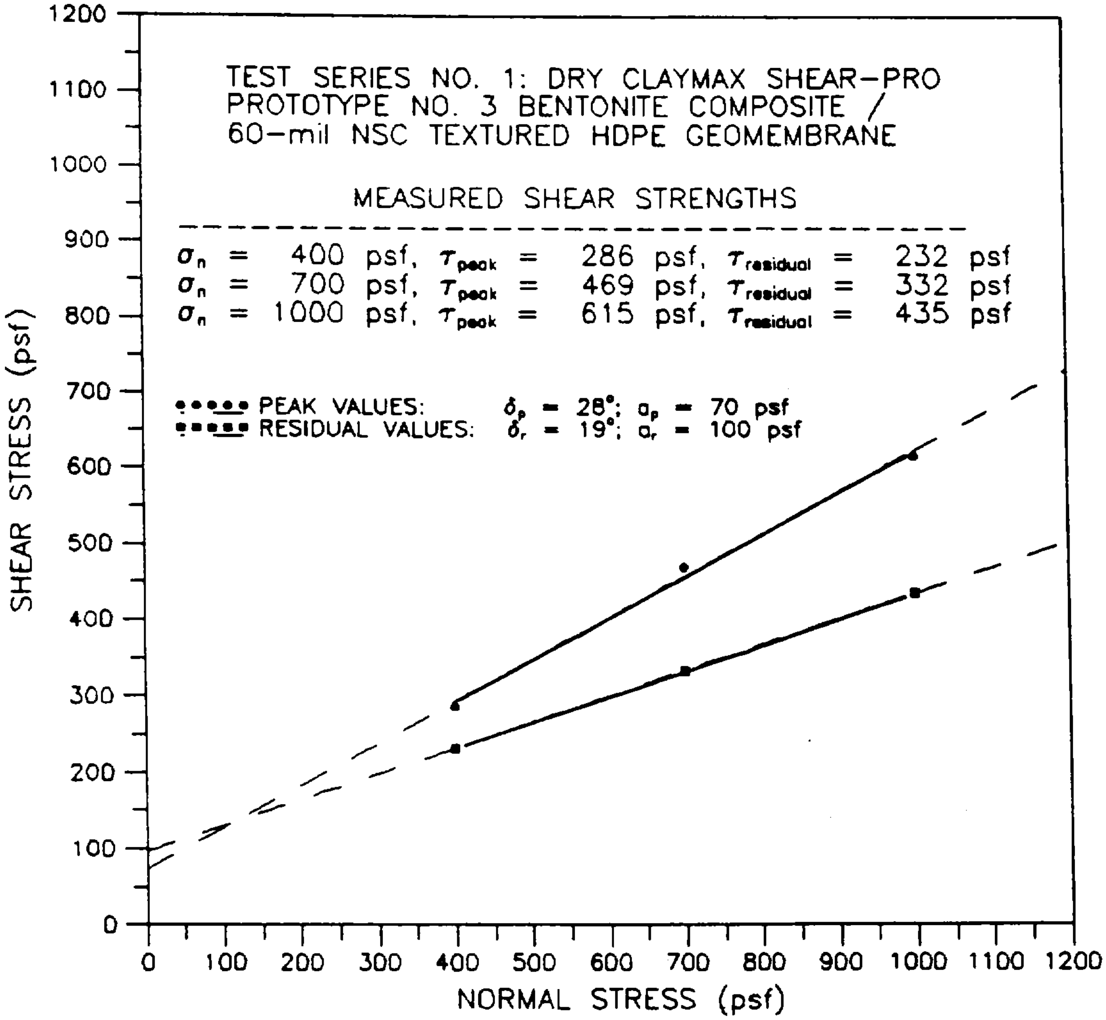
FIGURE NO.	B-1
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

30891J

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MARCH 1992

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FIGURE NO.	B-2
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

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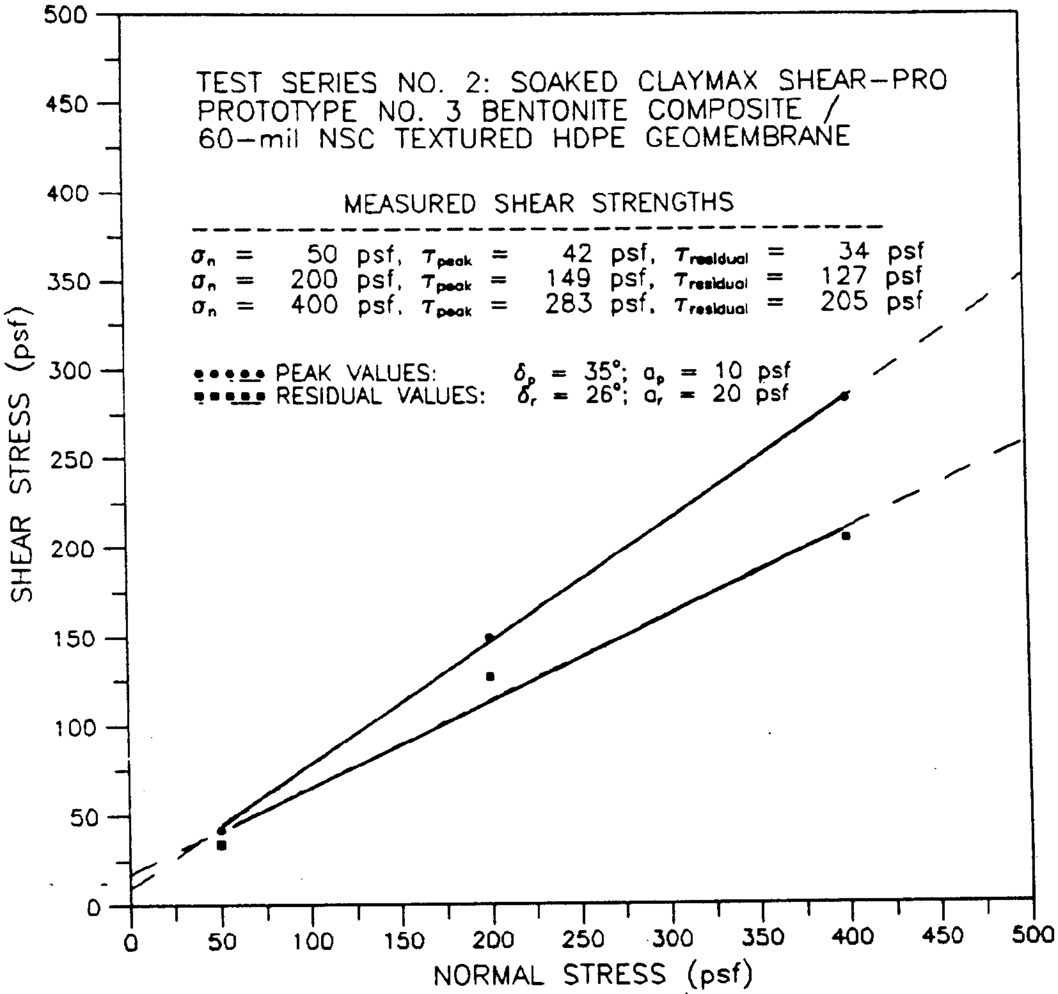
JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MARCH 1992

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	B-3
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

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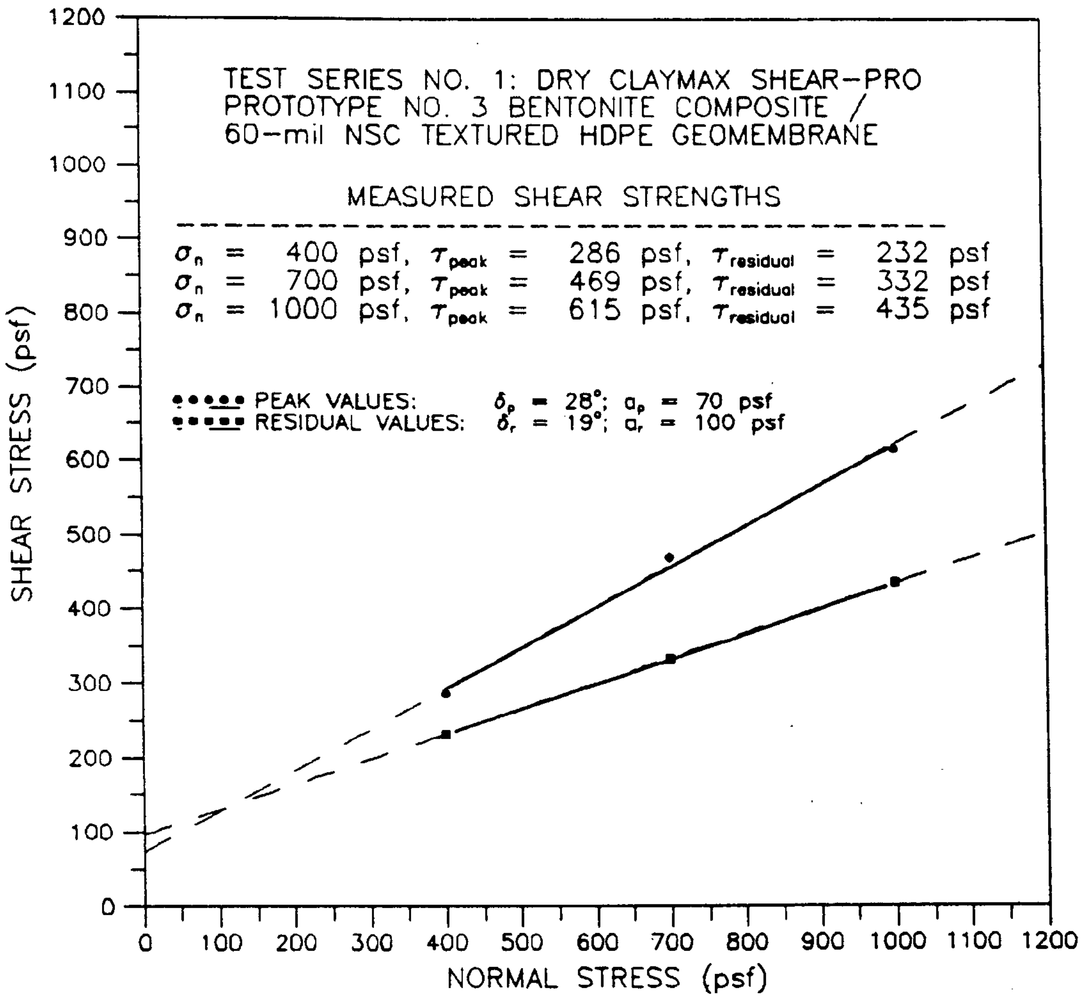
JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



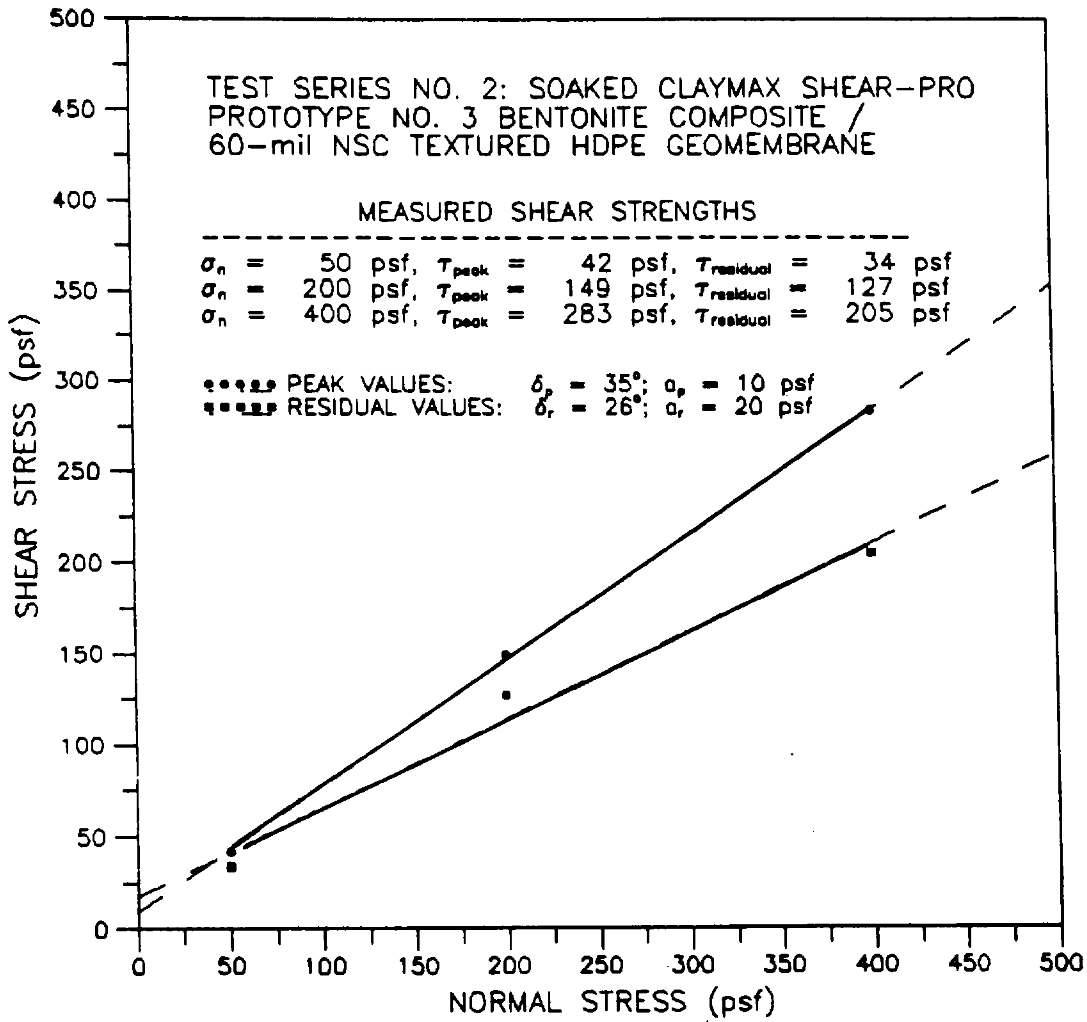
NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MARCH 1992

GEOSYNTEC CONSULTANTS GEOMECHANICS AND ENVIRONMENTAL LABORATORY	FIGURE NO.	B-2
	PROJECT NO.	GL3160
	DOCUMENT NO.	GEL92211
	PAGE NO.	

30891J

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



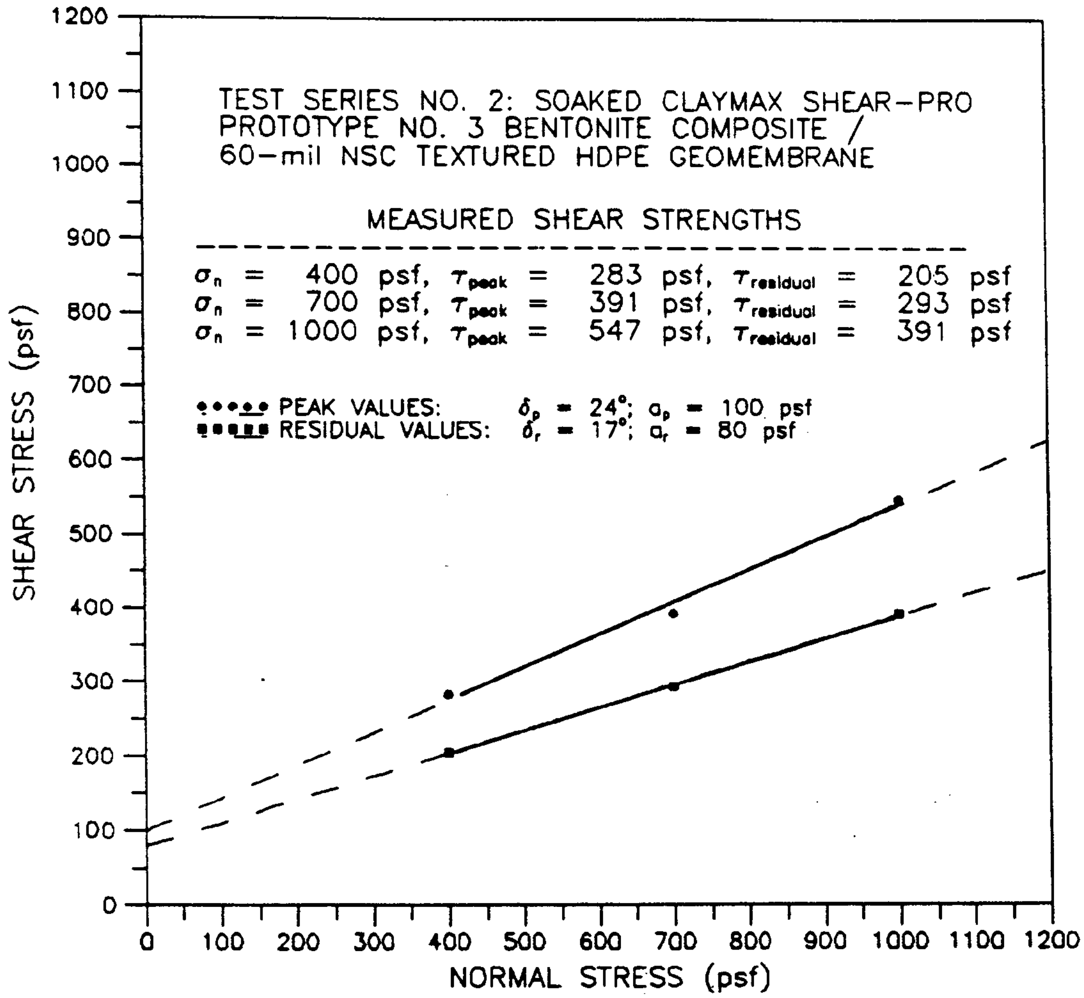
NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MARCH 1992

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	~~B-3~~
PROJECT NO.	~~GL3160~~
DOCUMENT NO.	~~GEL92211~~
PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING

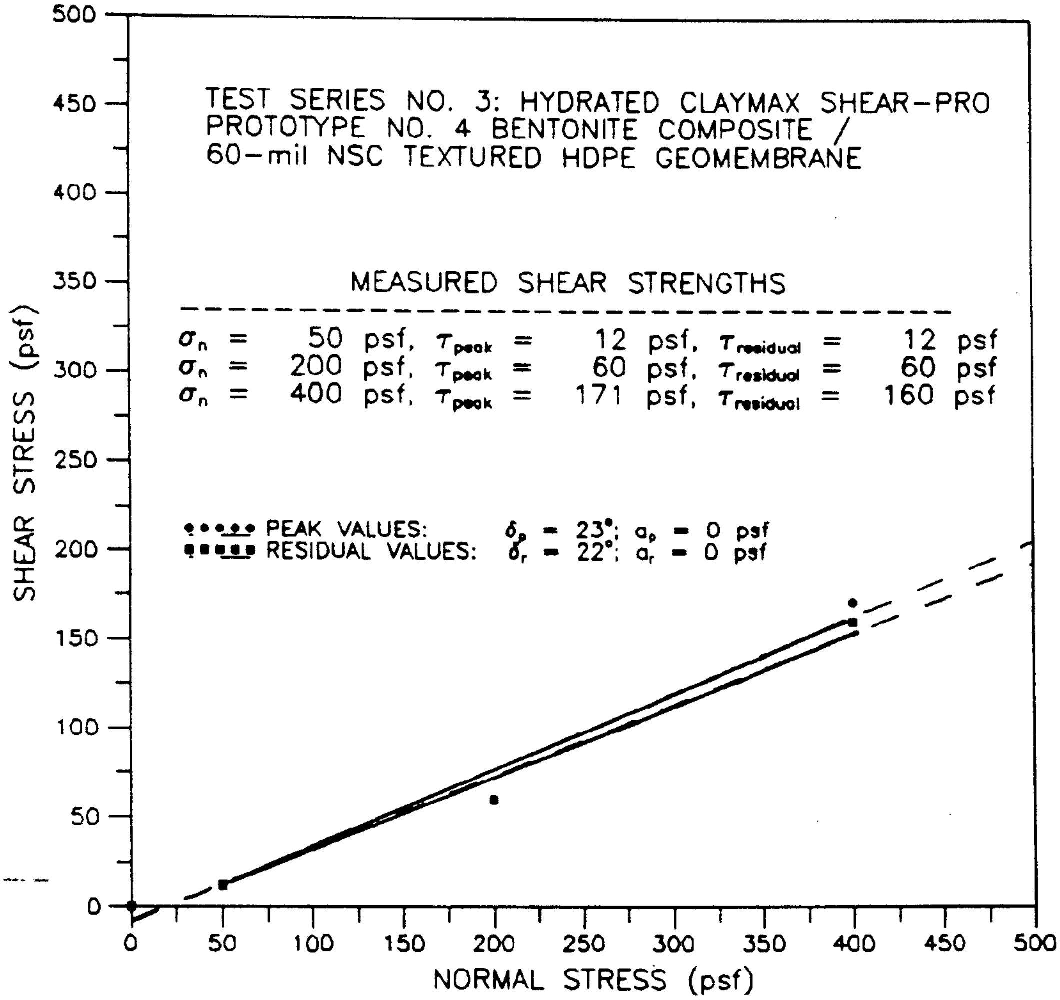


NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MARCH 1992

GEOSYNTEC CONSULTANTS — GEOMECHANICS AND ENVIRONMENTAL LABORATORY		
	FIGURE NO.	B-4
	PROJECT NO.	GL3160
	DOCUMENT NO.	GEL92211
	PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING

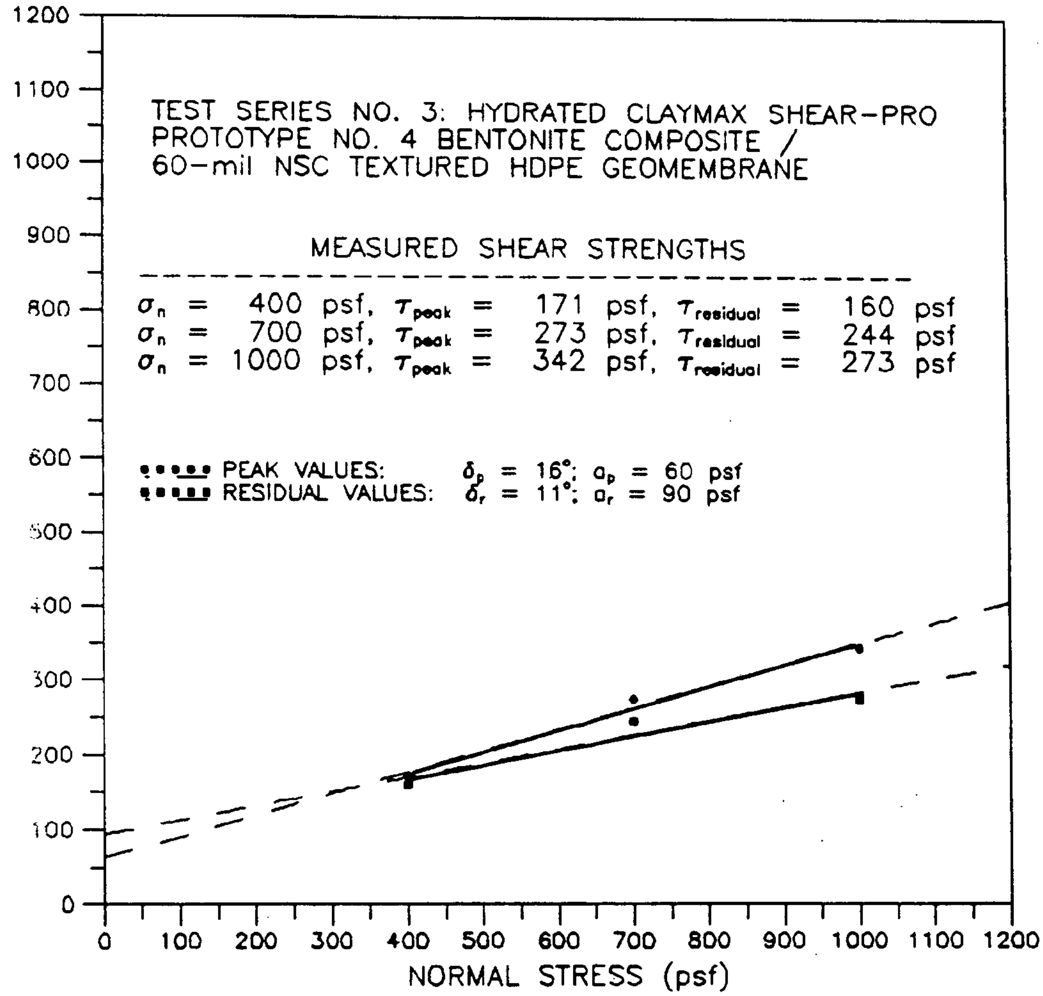


NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: APRIL 1992

GEOSYNTEC CONSULTANTS GEOMECHANICS AND ENVIRONMENTAL LABORATORY	FIGURE NO.	B-5
	PROJECT NO.	GL3160
	DOCUMENT NO.	GEL92211
	PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



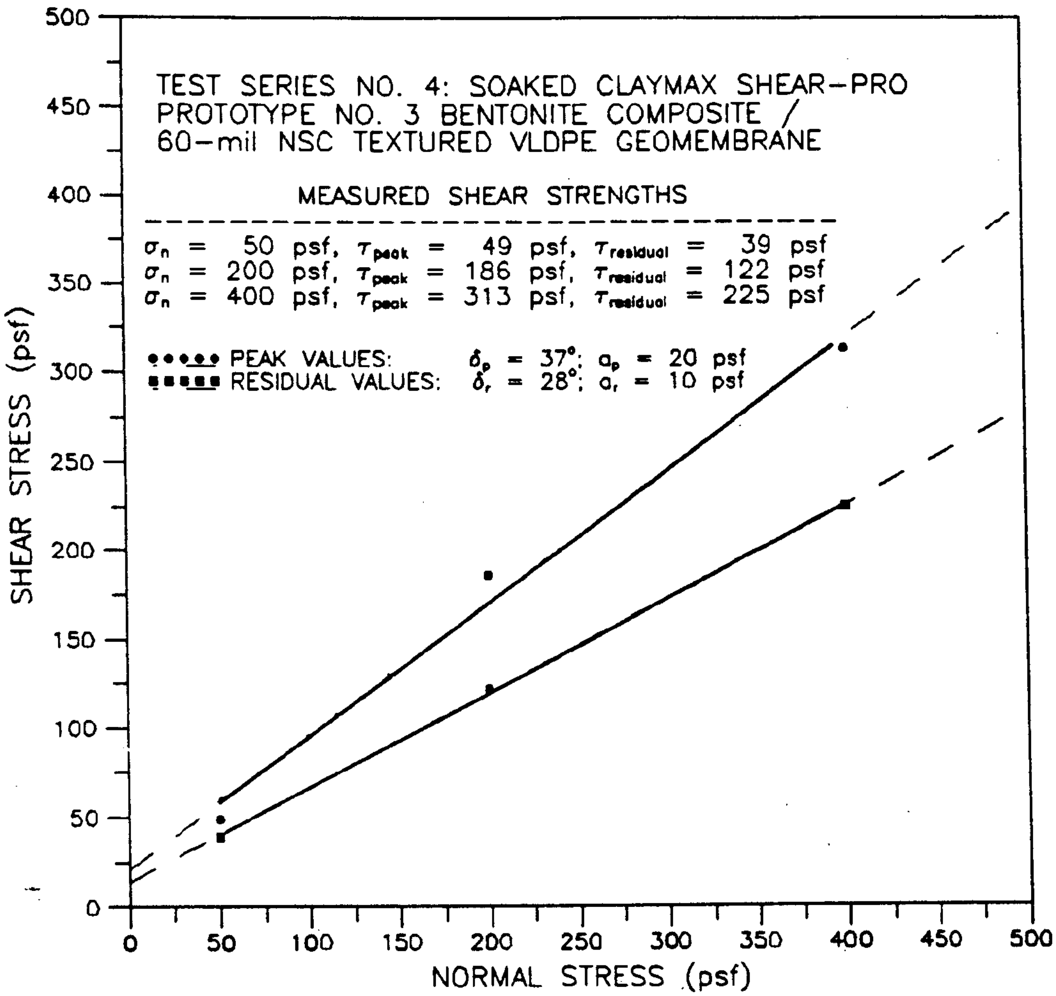
NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: APRIL 1992

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	B-6
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MARCH 1992

FIGURE NO.	B-7
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

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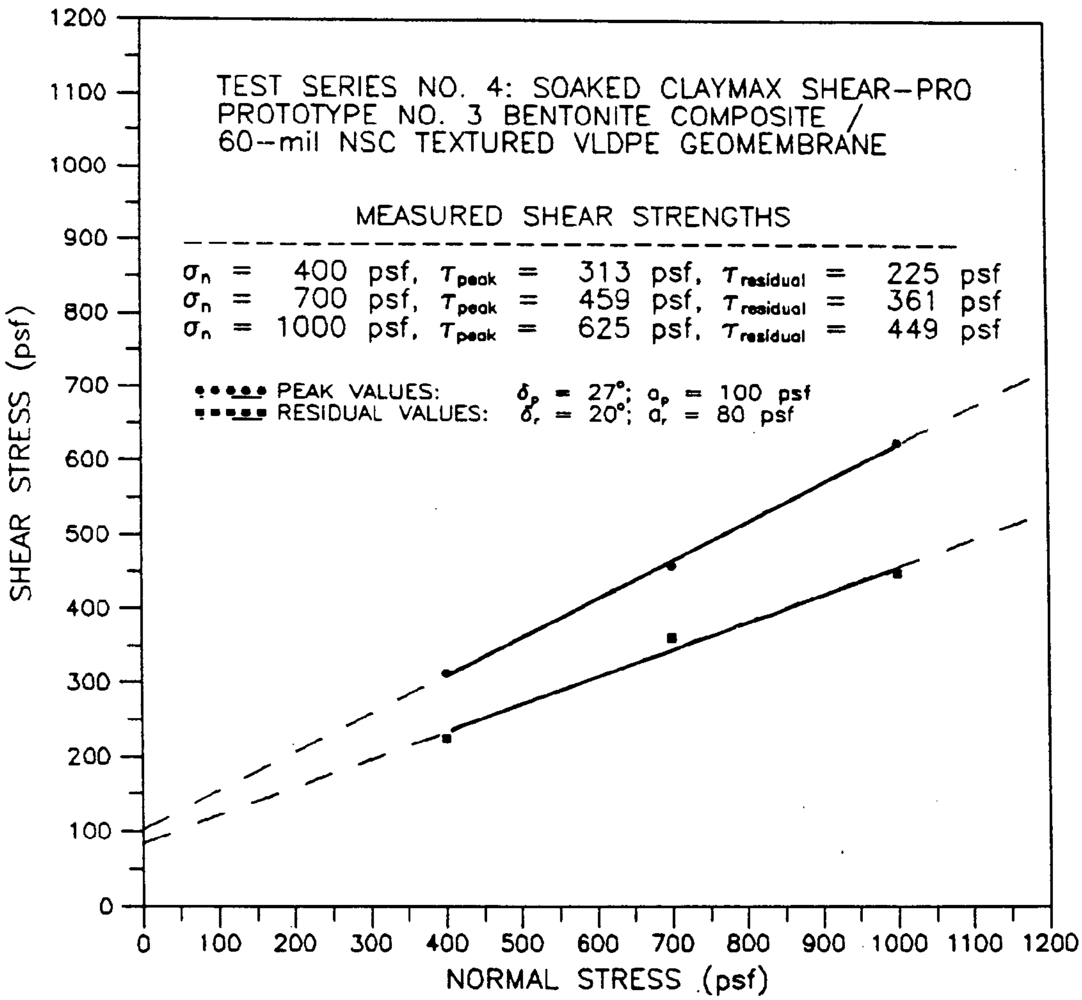
JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MARCH 1992

FIGURE NO.	B-8
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

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GEOMECHANICS AND ENVIRONMENTAL LABORATORY

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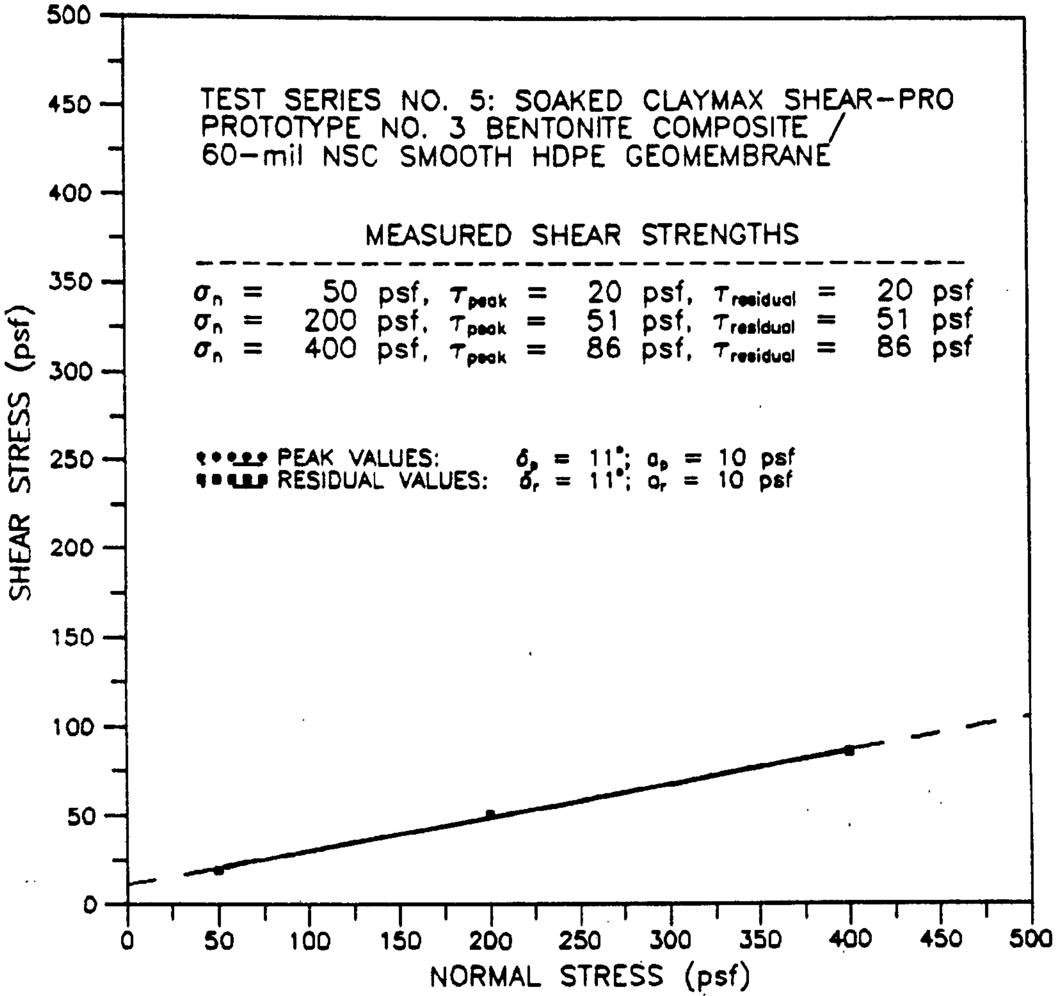
JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



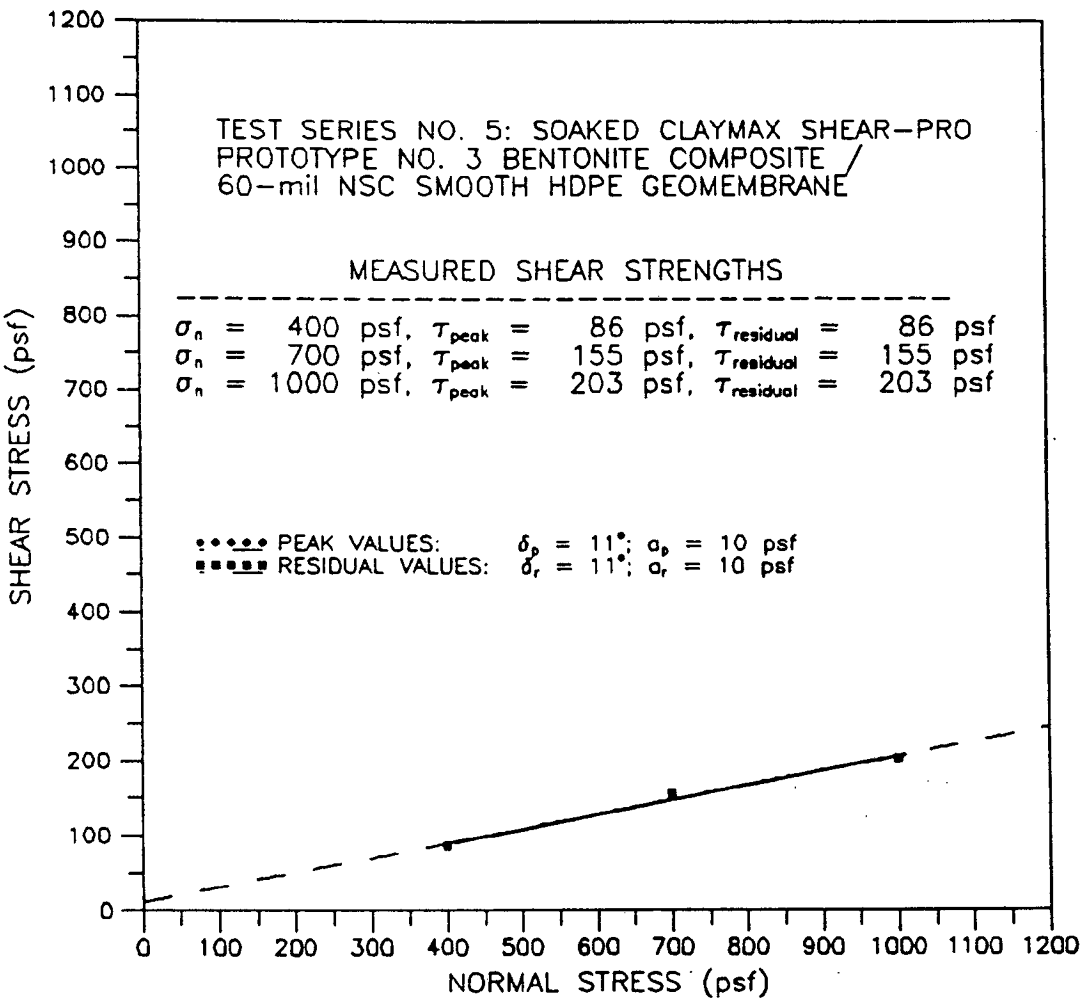
NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: APRIL 1992

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	B-9
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING

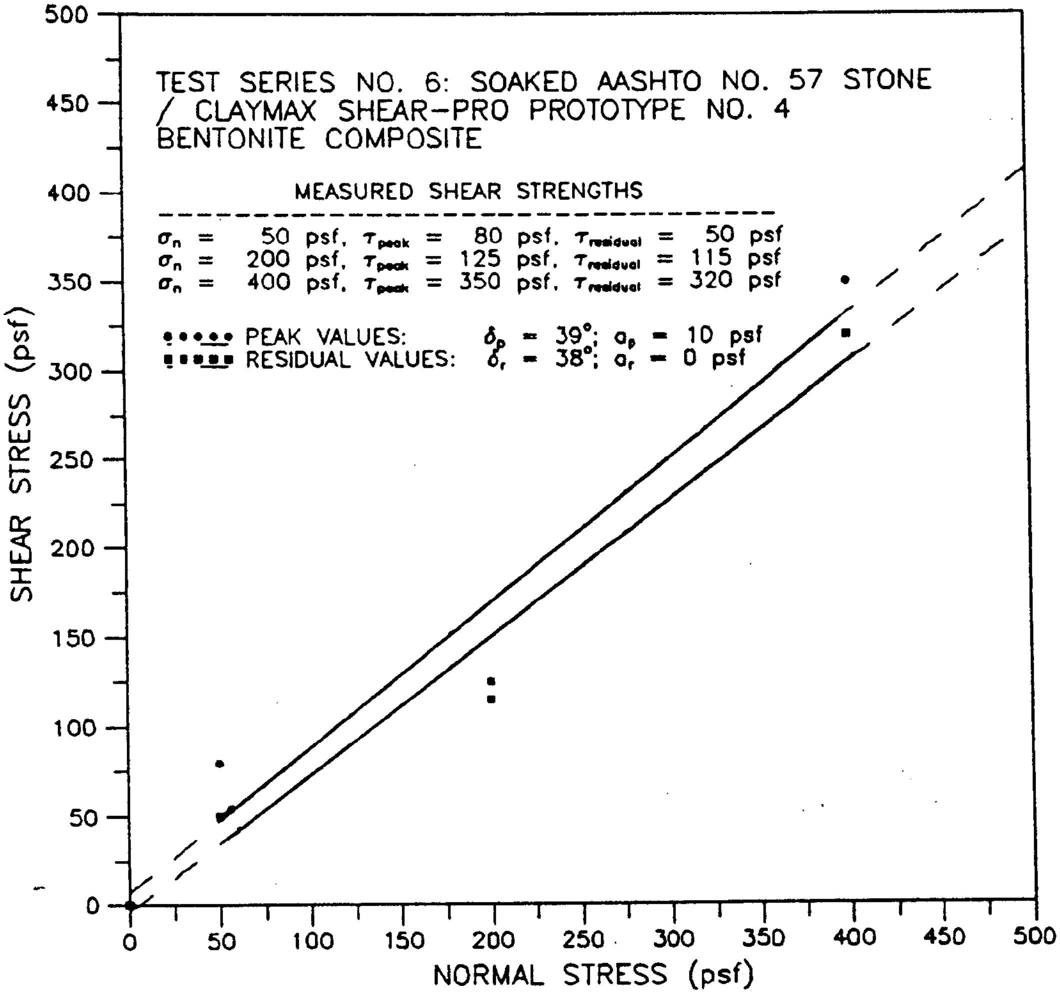


NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: APRIL 1992

GeoSyntec Consultants — GEOMECHANICS AND ENVIRONMENTAL LABORATORY		
	FIGURE NO.	B–10
	PROJECT NO.	GL3160
	DOCUMENT NO.	GEL92211
	PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



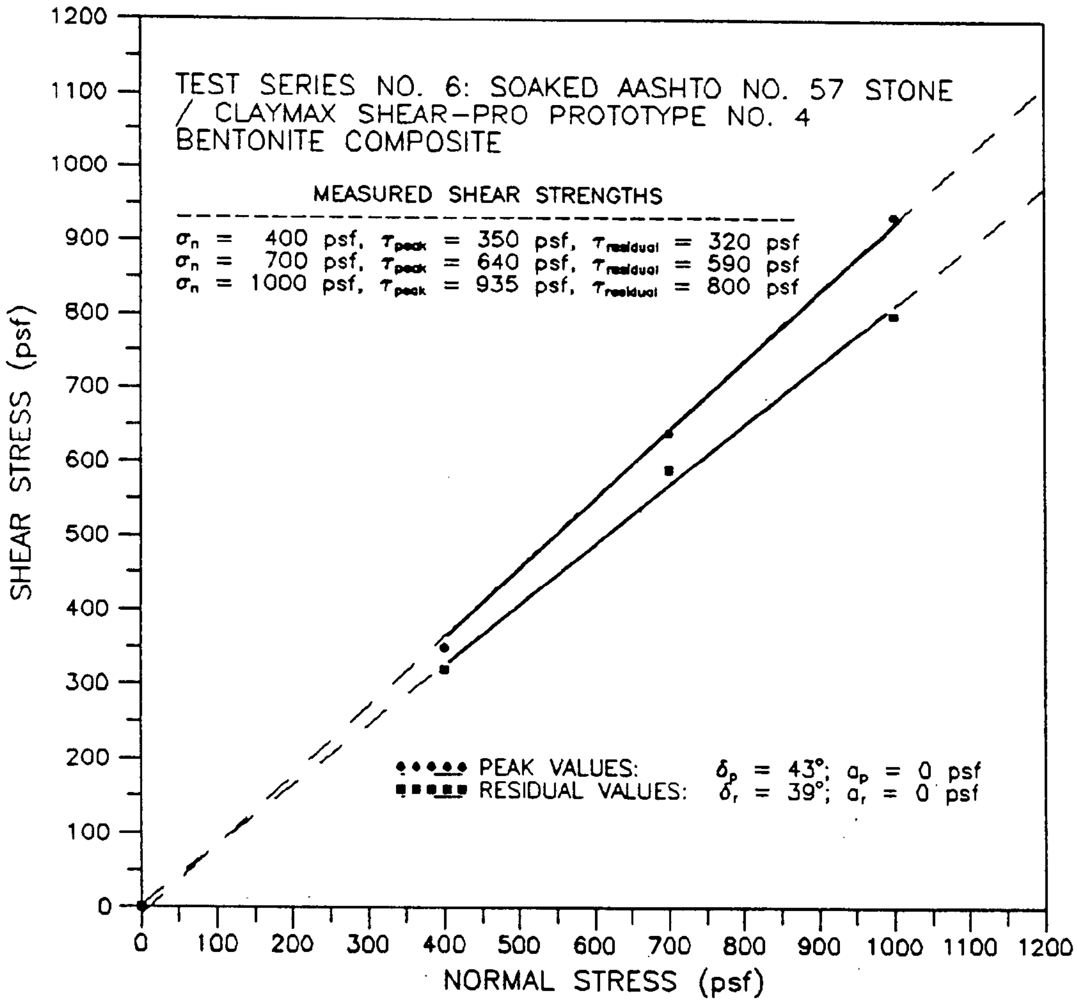
NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MAY 1992

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	B-11
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: MAY 1992

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	B-12
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

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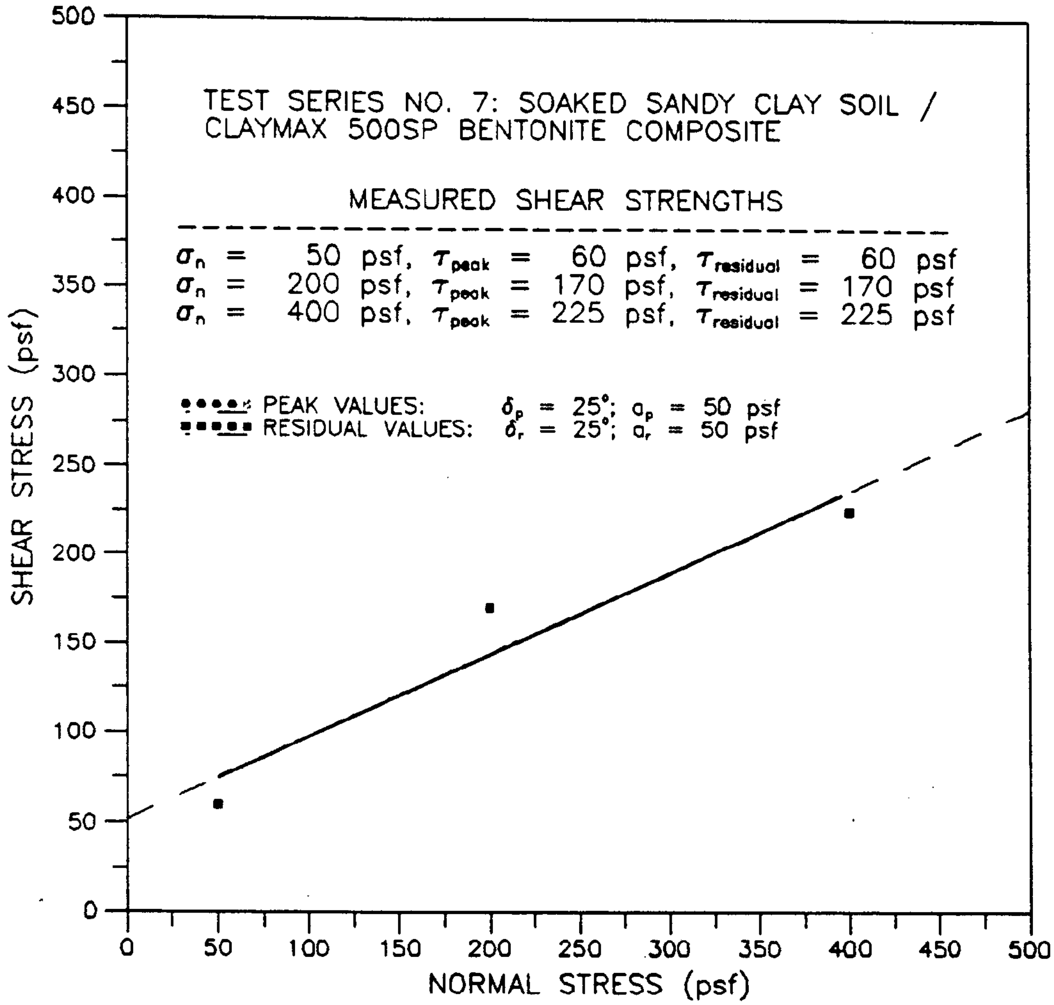
JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: NOVEMBER 1992

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	B-13
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

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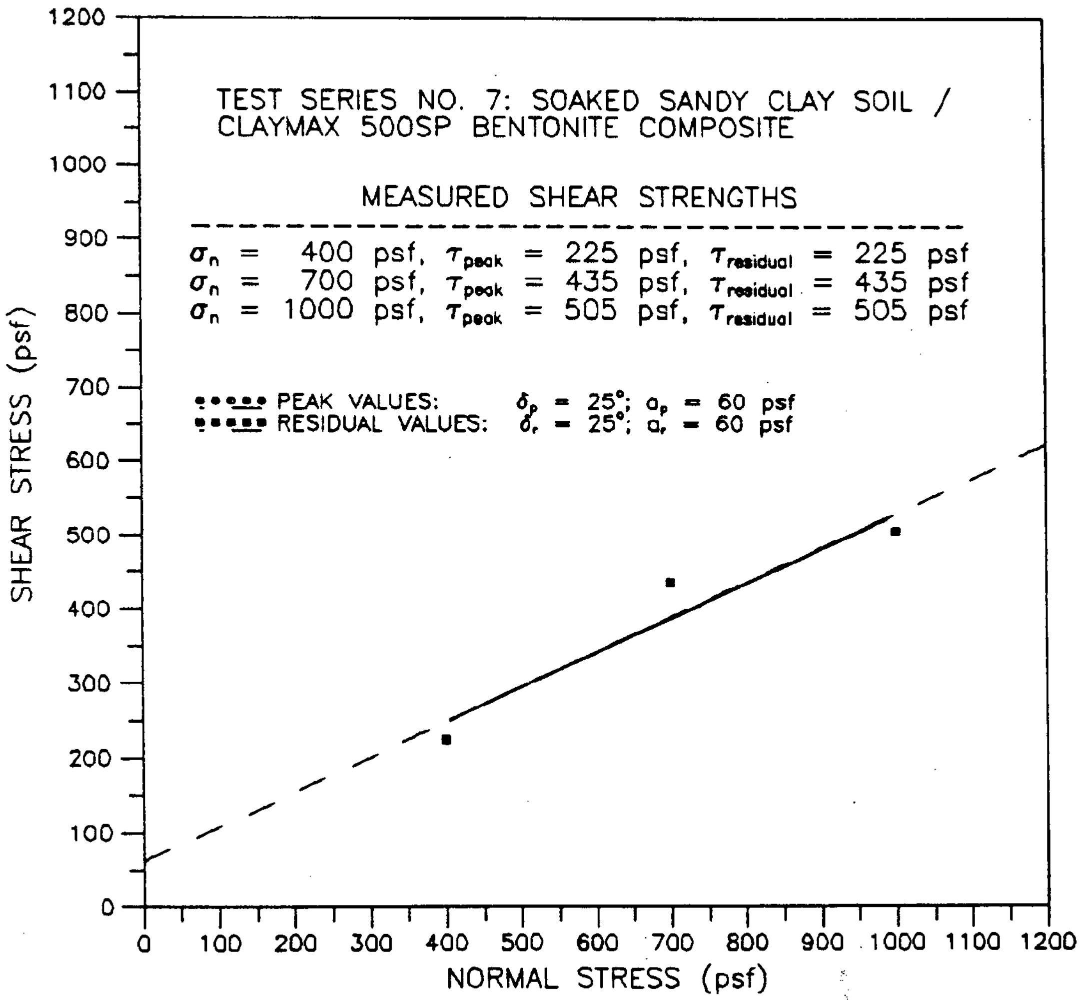
JAMES CLEM CORPORATION
INTERFACE DIRECT SHEAR TESTING



NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: NOVEMBER 1992

FIGURE NO.	B-14
PROJECT NO.	GL3160
DOCUMENT NO.	GEL92211
PAGE NO.	

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