Appendix Y

Miscellaneous Engineering Calculations
Revised August 2013
Revised November 2013

Documents Included:

Y(a) Leachate Pipe and Pump Calculations
Revised August 2013

Y(b) Estimated Maximum Settlements Leachate Collection Pipe Profile

Y(c) Water Management Calculations
Revised November 2013

Y(d) Flood Mitigation Calculations
Revised August 2013

Y(e) Geosynthetics Design Calculations

Appendix Y(a)

Leachate Pipe and Pump
Calculations
Revised August 2013

Ameren Missouri Labadie Energy Center
Leachate Pipe and Pump Calculations
Proposed Utility Waste Landfill
Franklin County, Missouri

January 2013, Revised August 2013

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Supplemental Information for Appendix Y(a)

Leachate Pump Calculation
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1.0 Introduction

Piping proposed for use at the Ameren Missouri Labadie Utility Waste Landfill (UWL) was reviewed for capacity and resistance to crushing and buckling under various conditions. First, capacity for leachate collection piping in the cells and the leachate force mains is estimated. Second, several scenarios representing a pipe element of the leachate collection system at some phase of the UWL development was checked for resistance to crushing and buckling. Sketches of each scenario are included in Section 3.5.

2.0 Pipe Capacity

2.1 Leachate Force Main

Leachate will be pumped to storage or treatment. Leachate pump and pipe requirements are estimated in this appendix.

Assumptions:

- The worst case flow of 13.4 gpm is in the 31.4-ac Cell 1 (see Appendix O, Table O-1, Sub Appendix O-11). Prorating this over the 166.5 acres, the flow is 71 gpm.
- The longest run of pipe is anticipated to be 2500 ft (the length of the furthest Cell 3 sump in southeast corner from Pond 2).
- Leachate will be pumped to a 12-ft diameter, horizontal tank on top of the perimeter berm and a 3-ft saddle. The elevation difference will be from the bottom of the sump to 15 ft above the top of berm:

  488 elevation + 15 ft - 464.2 elevation = 38.8 ft.

The head loss is estimated using the Hazen-Williams formula

$$H_f = [(0.00208 \times L) / (D_I^{4.8655})] \times (100 \times Q / C)^{1.85}$$

Where:
h_f is the head loss (ft),
L is the length (2500 ft),
D_I is the inside diameter of the pipe (in),
Q is the rate of flow (71 gpm), and
C is the friction factor (150 for HDPE).

The inside diameter 4-in nominal diameter DR17 pipe is 3.939 in. The head loss is

$$H_f = 0.00208 \times 2500 / 3.939^{4.8655} \times (100 \times 71/150)^{1.85} = 8.3 \text{ ft}$$

The total head is: 8.3 ft + 38.8 ft = 47.1 ft

There are 2 sumps in Cell 1, so the typical pump would only need to handle a rate of 13.4 gpm / 2 = 6.7 gpm (the sumps in the other cells have smaller drainage areas, and, therefore, will have smaller flows per sump). A review of leachate pump manufacturer's literature revealed that leachate pump models are available that can produce 10 gpm or more of flow at 50 ft of head (e.g., EPG SERIES 8 Surepump™).

2.2 Leachate Collection Pipe

The leachate collection pipes in each cell are intentionally oversized. The following calculations estimate the full-flow capacity of a nominal 6-in DR 11 HDPE pipe at a 0.5 percent slope using Manning's equation.

$Q = 1.49 / n \times A \times R^{2/3} X S^{1/2}$

Where:
Q is the flow (cfs),
n is Manning's n (0.009 for HDPE),
A is the cross-sectional area of the pipe (sq ft),
R is the wetted perimeter (ft), and
S is the slope (0.5 percent or 0.005 ft/ft).
$A = \pi \times d^2 / 4$

Where d is the inside diameter. For a nominal 6-in HOPE DR 11 pipe, the inside diameter is 5.348 in or 0.446 ft.

$A = \pi \times (0.4457 \text{ ft})^2 / 4 = 0.156 \text{ sq ft}$

$P = \pi \times d = \pi \times 0.446 \text{ ft} = 1.4 \text{ ft}$

$R = A / P = 0.156 \text{ sq ft} / 1.4 \text{ ft} = 0.111 \text{ ft}$

$Q = (1.49 / 0.009) \times 0.156 \times 0.111^{2/3} \times 0.005^{1/2} = 0.42 \text{ cfs}$

0.42 cfs x 7.48 gal/cfs x 60 s/min = **190 gpm**

As previously estimated, the maximum flow in a sump is approximately 7 gpm, but the use of the lowest flow leachate pump capacity at 11.1 gpm, actual flow is significantly less than the capacity of the proposed pipe.

3.0 Crushing and Buckling Scenarios

The methods used to estimate resistance to crushing and buckling follow those published by the Plastics Pipe Institute (PPI) in its Handbook for PE Pipe (2nd Edition). A conservative CCP unit weight, 120 pounds per cubic foot (95% compaction of the Standard Proctor), was used for all crushing and buckling calculations. This unit weight is higher than reported in the typical cell material profile provided in Scenario 2 (below) because 95% compaction of the CCP is not anticipated. Therefore, the calculations and reported factors of safety are conservative.

3.1 Scenario 1

Scenario 1 represents a leachate collection pipe (DR 11) placed in a trench with rock bedding, a minimum 12 inches of aggregate protective cover, and live loads. An H20 truck, which is a 20 ton truck with properties defined by The American Association of State Highway and Transportation Officials (AASHTO), is used for modeling live loads over the pipe.

Determine Total Vertical Load

1. **Earth Load - Vertical prism loads**

Earth Load $(P_E) = w_{cover}H_{cover} + w_{bedding}H_{bedding} = (120\ pcf\ *1.0\ ft) + (125\ pcf\ *1.5\ ft)$
$= 308\ psf$

Where:
w_{waste} = Density of Aggregate Cover = 120 pcf
H_{waste} = Depth of Aggregate Cover = 1.0 ft
$w_{bedding}$ = Density of Bedding = 125 pcf
$H_{bedding}$ = Depth of Bedding = 1.5 ft

2. **Live Load - Determine loading for an H20 truck using Timoshenko's Equation for a load directly above the pipe and the Boussinesq Equation for a load straddling the pipe. Use the greater load to be conservative**.

Timoshenko's Equation

$$\text{Llve Load } (P_L) = \frac{I_f W_w}{A_c}\left(1-\frac{H^3}{(r_\gamma^2+H^2)^{1.5}}\right) = \frac{3*16{,}000lb}{1.39\,ft^2}\left(1-\frac{(2.5\,ft)^3}{[(0.665\,ft)^2+(2.5\,ft)^2]^{1.5}}\right)$$
$$= 3{,}366\ psf$$

Where:
H = Total Depth of Cover = 2.5 ft
I_f = Impact Factor = 3 (Typical for unpaved surface)
W_w = Wheel Load = 16,000 Ib (Typical value for H20 truck

A_c = Contact Area = 1.39 ft^2 (Typical value for H20 truck)

r_γ = Equivalent Radius = $\sqrt{\frac{A_c}{\pi}} = \sqrt{\frac{1.39\,ft^2}{\pi}}$ = 0.665 ft

Boussinesq Equation

Live Load (P_L) = $\frac{3I_f W_W H^3}{2\pi r^2} = \frac{3*3*16{,}000lb*(2.5\,ft)^3}{2\pi(5.6\,ft)^2}$ *= 65.0 psf*

The live load is 130 psf to account for two wheels.

Where:
H = Total Depth of Cover = 2.5 feet
I_f = Impact Fator =3 (For an unpaved surface)
W_w = Wheel Load = 16,000 Ibs (Typical value for H20 Truck)
x = Horizontal distance from wheel to center of pipe = 5 ft. (assuming truck is 10 ft wide and centered over pipe)
r = Diagonal distance from wheel to center of pipe = $\sqrt{x^2 + H^2} = \sqrt{(5\,ft)^2 + (2.5\,ft)^2}$ = 5.6 ft

3. Total Vertical Load

Total Vertical Pressure (P_{Total}) = P_E + P_L = 308 psf + 3,366 psf = 3,700 psf

<u>Calculate Ring Deflection</u>

1. **Ring Deflection – Determine whether the ring deflection is less than the allowable 5% using Spangler's Modified Iowa Formula.**

Ring Deflection = $\left(\frac{\Delta X}{D_M}\right) = \frac{1}{144}\left(\frac{K_{BED}L_{DL}P_E + K_{BED}P_L}{\frac{2E}{3}\left(\frac{1}{DR-1}\right)^3 + 0.061F_S E'}\right)$

$= \frac{1}{144}\left(\frac{(0.1*1.5*308\,psf) + (0.1-3{,}366\,psf)}{\left(\frac{2*21{,}000}{3}\right)*\left(\frac{1}{11-1}\right)^3 + (0.061*0.85*3{,}000\,psi)}\right)$ *= 0.016 or 1.6%*

1.6 % < 5 %, therefore the ring deflection is within the acceptable range.

Where:
K_{BED} = Bedding Factor = 0.1 (Typical Value)
L_{DL} = Deflection Lag Factor = 1.5 (Typical Value)
P_E = 308 psf (Greater Value Calculated Above)
P_L = 3,366 psf (Calculated Above)
E = Apparent Modulus of Elasticity of Pipe Material = 21,000 psi (Assume 100 yrs, 73°F)
E' = Modulus of Soil Reaction = 3,000 psi (Assume compacted crushed rock)
F_s = Soil Support Factor = 0.85 (When: $\frac{E_N^{'}}{E'}$ = 0.2 and $\frac{B_d}{D_O}$ = 3)
DR = Dimension Ratio = 11

Crushing and Buckling Forces

1. **Compressive Stress - Determine whether the compressive stress is less than the allowable 800 psi.**

Compressive Stress (S) = $\frac{P_{Total} * DR}{288} = \frac{3{,}700\,psf * 11}{288}$ = *141 psi*

141 psi < 800 psi, the compressive stress value is within the acceptable range.

Where:
P_{Total} = 3,700 psf (Previously calculated)
DR = Dimension Ratio = 11

2. **Allowable Constrained Buckling Pressure - Determine if the buckling pressure is greater than P_{TOTAL} (3,700 psf) using Luscher's Equation.**

Constrained Buckling Pressure (P_{WC}) = $\frac{5.65}{N}\sqrt{RB'E'*\frac{E}{12(DR-1)^3}}$

= $\frac{5.65}{2}\sqrt{0.80*0.227*3{,}000\,psi*\frac{21{,}000\,psi}{12(11-1)^3}}$ = *87.2 psi = 12,550 psf*

12,550 psf > 3,700 psf, the buckling pressure is within the acceptable range

Where:
N = Safety Factor = 2

R = Buoyancy Reduction Factor = 1 - 0.33 $\frac{H_{GW}}{H}$ = 1 – 0.33 $\frac{1.5\,ft}{2.5\,ft}$ = 0.80

H_{GW} = Groudwater Height Above Pipe = 1.5 ft assuming a maximum 1 ft allowed on liner plus an addition 0.5 ft.
H = Cover Above Pipe = 2.5 ft

B' = Soil Support Factor = $\frac{1}{1+4e^{-0.065H}}$ = $\frac{1}{1+4e^{-0.065*2.5}}$ = 0.227

E = Apparent Modulus of Elasticity of Pipe Material = 21,000 psi (Assume 100 yrs, 73°F)
E' = Modulus of Soil Reduction = 3,000 psi (Assuming compacted crushed rock)

3.2 Scenario 2

Scenario 2 represents a leachate collection pipe as in Scenario 1, except under the loading conditions of the UWL at full capacity.

Determine Total Vertical Load

1. **Earth load - Vertical prism loads**

Earth Load (P_E)= $w_{waste}H_{waste}$ + $w_{soil}H_{soil}$ + $w_{bedding}H_{bedding}$

= $(120\,pcf * 98\,ft) + (120\,pcf * 2\,ft) + (125\,pcf * 1.5\,ft) = 12{,}188\,psf$

Where:
w_{waste} = Density of Waste = 120 pcf
H_{waste} = Depth of Waste = 98 ft
W_{soil} = Density of Waste = 120 pcf
H_{soil} = Depth of Waste = 2.0 ft
$w_{bedding}$ = Density of Bedding = 125 pcf
$H_{bedding}$ = Depth of Bedding = 1.5 ft

2. **Live Load – No Live Load Exists**

P_L = 0 psf

3. **Total Vertical Load**

Total Vertical Pressure (P_T) = P_L + P_E = 0 psf + 12,188 psf = 12,188 psf

Calculate Ring Deflection

1. **Rigidity Factor – Use the Watkins- Gaube Method to find Rigidity Factorm Deformation Factor, and Soil Stress. From this, Ring Deflection can be found and should be less than the allowable 5%.**

Rigidity Factor (R_f) = $\frac{12E_s(DR-1)^3}{E} = \frac{12*3{,}491psi*(11-1)^3}{21{,}000psi}$ *= 1,995*

Where:

E_s = Secant Modulus of Soil = $M_s\frac{(1+\mu)(1-2\mu)}{(1-\mu)} = 4{,}700psi\frac{(1+0.3)(1-2*0.3)}{(1-0.3)}$

= 3,491 psi

Assuming, M_s = 4,700 psi and μ = 0.3, based on typical values.

DR = Dimension Ratio = 11

E = Apparent Modulus of Elasticity of Pipe Material = 21,000 psi (Assume 100 yrs, 73°F)

2. **Deformation Factor – For Rigidity Factor of 1,995**

Deformation Factor (DF) = 1.5

3. **Soil Strain**

Soil Strain (ε_s) = $\frac{P_E}{144E_s} = \frac{12{,}188psi}{144*3{,}491psi}$ *= 0.024 or 2.4 %*

Where:
P_E = 12,188 psi (previously calculated)
E_s = 3,491 psi (previously calculated)

4. **Ring Deflection – Determine whether Ring Deflection is less than the allowable 5%.**

Ring Deflection $\left(\frac{\Delta X}{D_M}\right)$ *= ε_s(%)*DF = 2.4% * 1.5 = 3.6%*

Since 3.6% < 9%, the ring deflection is within acceptable range.

<u>Calculate Hoop Stress</u>

1. **Hoop Thrust Stiffness Ratio –**

Hoop Stress Stiffness Ratio (S_A) = $\frac{1.43 M_S r_{cent}}{EA}$ = $\frac{1.43 * 4{,}700\, psi * 3.095 in}{21{,}000\, psi * 0.60 in}$ = *1.65*

Where:
E = Apparent Modulus of Elasticity of Pipe Material = 21,000 psi (Assume 100 yrs, 73°F)
A = Pipe Thickness = 0.60 in
M_S = 4,700 psi (Typical Value, From Table 3.12)
r_{cent} = radius to pipe centroid = 3.095 in

2. **Vertical Arching Factor –**

Vertical Arching Factor (VAF) = $0.88 - 0.71\left(\frac{S_A - 1}{S_A + 2.5}\right)$ = $0.88 - 0.71\left(\frac{1.65 - 1}{1.65 + 2.5}\right)$ = *0.769*

3. **Hoop Stress – Determine if Hoop Stress is less than the allowable 800 psi using the radial directed earth pressure (P_{RD})**

Radial Directed Earth Pressure (P_{RD}) = $VAF * P_E$ = $0.769 * 12{,}188\, psf$ = *9,373 psf*

Where:
P_E = Vertical Earth Load = 11,403 psf (calculated above)

Hoop Stress (S) = $\frac{(P_{RD} + P_L) DR}{288}$ = $\frac{(9{,}373\, psf + 0\, psf) * 11}{288}$ = *358 psi*

358 psi < 800 psi, therefore the hoop stress is within the acceptable range

Where:
P_L = 0 psf (No live load)
DR = Dimension Ratio = 11

3.3 Scenario 3

Scenario 3 represents a sump riser (DR 17) on the side slope, bedded in a trench, and under a live load. Loads were treated as if they were normal to the pipe. This is a larger pipe that contains the sump and pump discharge pipe.

Determine Total Vertical Load

1. Earth Load - Vertical prism loads

*Earth Load (P_E)= $w_{cover}H_{cover}$ + $w_{bedding}H_{bedding}$ = (120 pcf *1.0 ft) + (125 pcf *1.0 ft)*
= 245 psf

Where:
w_{waste} = Density of Aggregate Protective Cover = 120 pcf
H_{waste} = Depth of Aggregate Protective Cover = 1.0 ft
$w_{bedding}$ = Density of Bedding = 125 pcf
$H_{bedding}$ = Depth of Bedding = 1.0 ft

2. Live Load - Determine loading for a 6,000 lb (3-ton) skid steer directly above the pipe using Timoshenko's Equation. According to the PPI Handbook, the load of a wheel directly over the pipe will be greater than two wheels straddling the pipe when there is less than 4ft of cover.

Timoshenko's Equation

$$\text{LIve Load } (P_L) = \frac{I_f W_w}{A_c}\left(1-\frac{H^3}{(r_\gamma^2+H^2)^{1.5}}\right) = \frac{3*1{,}500\,lb}{0.89\,ft^2}\left(1-\frac{(2.0\,ft)^3}{[(0.53\,ft)^2+(2.0\,ft)^2]^{1.5}}\right)$$

= 489 psf

Where:
H = Total Depth of Cover = 2.0 ft
I_f = Impact Factor = 3 (Typical for unpaved surface)
W_w = Wheel Load = 6,000 lb/ 4 tires = 1,500 Ib
A_c = Contact Area = 0.66 ft * 1.33 ft = 0.89 ft^2

r_γ = Equivalent Radius = $\sqrt{\frac{A_c}{\pi}} = \sqrt{\frac{0.89\,ft^2}{\pi}}$ = 0.53 ft

3. Total Vertical Load

Total Vertical Pressure (P_T) = P_L + P_E = 489 psf + 245 psf = 734 psf

Calculate Ring Deflection

1. Ring Deflection – Determine whether the ring deflection is less than the allowable 5% using Spangler’s Modified Iowa Formula.

Ring Deflection = $\left(\frac{\Delta X}{D_M}\right) = \frac{1}{144}\left(\frac{K_{BED}L_{DL}P_E + K_{BED}P_L}{\frac{2E}{3}\left(\frac{1}{DR-1}\right)^3 + 0.061F_S E'}\right)$

$= \frac{1}{144}\left(\frac{(0.1*1.5*245\,psf) + (0.1-489\,psf)}{\left(\frac{2*21{,}000}{3}\right)*\left(\frac{1}{17-1}\right)^3 + (0.061*0.3*3{,}000\,psi)}\right)$ = *0.010 or 1.0%*

1.0 % < 5 %, therefore the ring deflection is within the acceptable range.

Where:
K_{BED} = Bedding Factor = 0.1 (Typical Value)
L_{DL} = Deflection Lag Factor = 1.5 (Typical Value)
P_E = 245 psf (Calculated Above)
P_L = 489 psf (Calculated Above)
E = Apparent Modulus of Elasticity of Pipe Material = 21,000 psi (Assume 100 yrs, 73°F)
E' = Modulus of Soil Reaction = 3,000 psi (Assume compacted crushed rock)
F_s = Soil Support Factor = 0.3 (When: $\frac{E_N'}{E'}$ = 0.2 and $\frac{B_d}{D_O}$ = 1.5)
DR = Dimension Ratio = 17

Crushing and Buckling Forces

1. **Compressive Stress - Determine whether the compressive stress is less than the allowable 800 psi.**

Compressive Stress (S) = $\frac{P_{Total} * DR}{288} = \frac{734\,psf * 17}{288}$ = *43 psi*

43 psi < 800 psi, the compressive stress value is within the acceptable range.

Where:
P_{Total} = 734 psf (Previously calculated)
DR = Dimension Ratio = 17

2. **Allowable Constrained Buckling Pressure - Determine if the buckling pressure is greater than P_{TOTAL} (734 psf) using Luscher's Equation.**

Constrained Buckling Pressure (P_{WC}) = $\frac{5.65}{N}\sqrt{RB'E'*\frac{E}{12(DR-1)^3}}$

= $\frac{5.65}{2}\sqrt{1.0*0.222*3{,}000\,psi*\frac{21{,}000\,psi}{12(17-1)^3}}$ *= 47.7 psi = 6,869 psf*

6,869 psf > 734 psf, the buckling pressure is within the acceptable range

Where:
N = Safety Factor = 2
R = Buoyancy Reduction Factor = 1 - 0.33 $\frac{H_{GW}}{H}$ = 1 – 0.33 $\frac{0\,ft}{2.0\,ft}$ = 1.0

H_{GW} = Groudwater Height Above Pipe = 0 ft because there will be no standing water on the slope
H = Cover Above Pipe = 2.0 ft

B' = Soil Support Factor = $\frac{1}{1+4e^{-0.065H}}$ = $\frac{1}{1+4e^{-0.065*2.0}}$ = 0.222
E = Apparent Modulus of Elasticity of Pipe Material = 21,000 psi (Assume 100 yrs, 73°F)

E' = Modulus of Soil Reduction = 3,000 psi (Assuming compacted crushed rock)

Calculate Allowable Live Load Pressure

1. **Allowable Live Load Pressure – Calculate live load pressure for a shallow cover situation. The pressure calculated should be less than the live load.**

Allowable Live Load Pressure (P_{LA}) = $\frac{12w(KH)^2}{ND_o}+\frac{7387.2(I)}{ND_o{}^2C}\left(S-\frac{wD_oH}{288A}\right)$

= $\frac{12*120\,pcf\,(2.46*2\,ft)^2}{2*18in}+\frac{7387.2*0.094}{2*(18in)^2*0.53in}\left(3{,}000\,psi-\frac{120\,pcf\,*18in*2\,ft}{288*1.06in}\right)$

= 7,006 psf

734 psf < 7,006 psf, the allowable live load is in the acceptable range

Where:
w = Average Density of Cover Material = 120 pcf
H = Depth of Cover = 2 ft

K = Passive Earth Pressure Coefficient = $\frac{1+\sin\phi}{1-\sin\phi}$ = $\frac{1+\sin(25)}{1-\sin(25)}$ = 2.46

ϕ = 25° for a loose silty material

N = Safety Factor = 2

D_o = Outside Diameter of Pipe = 18 in

A = Pipe Wall Thickness = 1.06 in (Based in DR of 17)

C = Outer Fiber Wall of Centroid = 0.5t = 0.5*1.06 in = 0.53 in

S = Material Yield Strength = 3,000 psi

I = Pipe Wall Moment of Inertia = $\frac{t^2}{12}$ = $\frac{(1.06in)^2}{12}$ = 0.094

3.4 Scenario 4

Scenario 4 represents a pipe (DR 17) in the perimeter berm for carrying leachate to a holding tank.

<u>Determine Total Vertical Load</u>

1. **Earth Load – Vertical prism loads**

   *Earth Load (P_E)= $w_{soil}H_{soil}$ = (120 pcf *4.0 ft) = 480 psf*

   Where:
   w_{soil} = Density of Soil = 120 pcf
   H_{soil} = Depth of Soil Cover = 4.0 ft

2. **Live Load - Determine loading for an H20 truck using Timoshenko's Equation for a load directly above the pipe and the Boussinesq Equation for a load straddling the pipe. Use the greater load to be conservative**.

   Timoshenko's Equation

   LIve Load (P_L) = $\frac{I_f W_w}{A_c}(1-\frac{H^3}{(r_\gamma^2+H^2)^{1.5}}) = \frac{3*16{,}000lb}{1.39\,ft^2}(1-\frac{(4.0\,ft)^3}{[(0.665\,ft)^2+(4.0\,ft)^2]^{1.5}})$

   = 1,384 psf

   Where:
   H = Total Depth of Cover = 4.0 ft
   I_f = Impact Factor = 3 (Typical for unpaved surface)
   W_w = Wheel Load = 16,000 Ib (Typical value for H20 truck
   A_c = Contact Area = 1.39 ft^2 (Typical value for H20 truck)

r_γ = Equivalent Radius = $\sqrt{\frac{A_c}{\pi}} = \sqrt{\frac{1.39\,ft^2}{\pi}}$ = 0.665 ft

Boussinesq Equation

Live Load (P_L) = $\frac{3I_f W_W H^3}{2\pi r^2} = \frac{3*3*16{,}000lb*(4.0\,ft)^3}{2\pi(6.4\,ft)^2}$ *= 137.0 psf*

The live load is 274 psf to account for two wheels.

Where:
H = Total Depth of Cover = 4.0 feet
I_f = Impact Fator =3 (For an unpaved surface)
W_w = Wheel Load = 16,000 Ibs (Typical value for H20 Truck)
x = Horizontal distance from wheel to center of pipe = 5 ft. (assuming truck is 10 ft wide and centered over pipe)
r = Diagonal distance from wheel to center of pipe = $\sqrt{x^2 + H^2} = \sqrt{(5\,ft)^2 + (4.0\,ft)^2}$ = 6.4 ft

3. **Total Vertical Load**

Total Vertical Pressure (P_{Total}) = P_E + P_L = 480 psf + 1,384 psf = 1,864 psf

<u>Calculate Ring Deflection</u>

1. **Ring Deflection – Determine whether the ring deflection is less than the allowable 5% using Spangler's Modified Iowa Formula.**

Ring Deflection = $\left(\frac{\Delta X}{D_M}\right) = \frac{1}{144}\left(\frac{K_{BED}L_{DL}P_E + K_{BED}P_L}{\frac{2E}{3}\left(\frac{1}{DR-1}\right)^3 + 0.061F_S E'}\right)$

$= \frac{1}{144}\left(\frac{(0.1*1.5*480\,psf) + (0.1-1{,}384\,psf)}{\left(\frac{2*21{,}000}{3}\right)*\left(\frac{1}{17-1}\right)^3 + (0.061*0.85*2{,}000\,psi)}\right)$ *= 0.013 or 1.3%*

1.3 % < 5 %, therefore the ring deflection is within the acceptable range.

Where:
K_{BED} = Bedding Factor = 0.1 (Typical Value)
L_{DL} = Deflection Lag Factor = 1.5 (Typical Value)
P_E = 480 psf (Greater Value Calculated Above)
P_L = 1,384 psf (Calculated Above)
E = Apparent Modulus of Elasticity of Pipe Material = 21,000 psi (Assume 100 yrs, 73°F)
E' = Modulus of Soil Reaction = 2,000 psi (Assume compacted coarse grained soil)
F_s = Soil Support Factor = 0.85 (When: $\frac{E'_N}{E'}$ = 0.2 and $\frac{B_d}{D_O}$ = 3)
DR = Dimension Ratio = 17

Crushing and Buckling Forces

1. **Compressive Stress - Determine whether the compressive stress is less than the allowable 800 psi.**

Compressive Stress (S) = $\frac{P_{Total} * DR}{288}$ = $\frac{1{,}864\,psf * 17}{288}$ = *110 psi*

110 psi < 800 psi, the compressive stress value is within the acceptable range.

Where:
P_{Total} = 1,864 psf (Previously calculated)
DR = Dimension Ratio = 17

2. **Allowable Constrained Buckling Pressure - Determine if the buckling pressure is greater than P_{TOTAL} (1,864 psf) using Luscher's Equation.**

Constrained Buckling Pressure (P_{WC}) = $\frac{5.65}{N}\sqrt{RB'E' * \frac{E}{12(DR-1)^3}}$

= $\frac{5.65}{2}\sqrt{1.0 * 0.245 * 2{,}000\,psi * \frac{21{,}000\,psi}{12(17-1)^3}}$ = *40.9 psi = 5,890 psf*

5,890 psf > 1,864 psf, the buckling pressure is within the acceptable range

Where:
N = Safety Factor = 2
R = Buoyancy Reduction Factor = 1 - 0.33 $\frac{H_{GW}}{H}$ = 1 – 0.33 $\frac{0\,ft}{4.0\,ft}$ = 1.0

H_{GW} = Groudwater Height Above Pipe = 0 ft because there will be no standing water on the slope

H = Cover Above Pipe = 4.0 ft

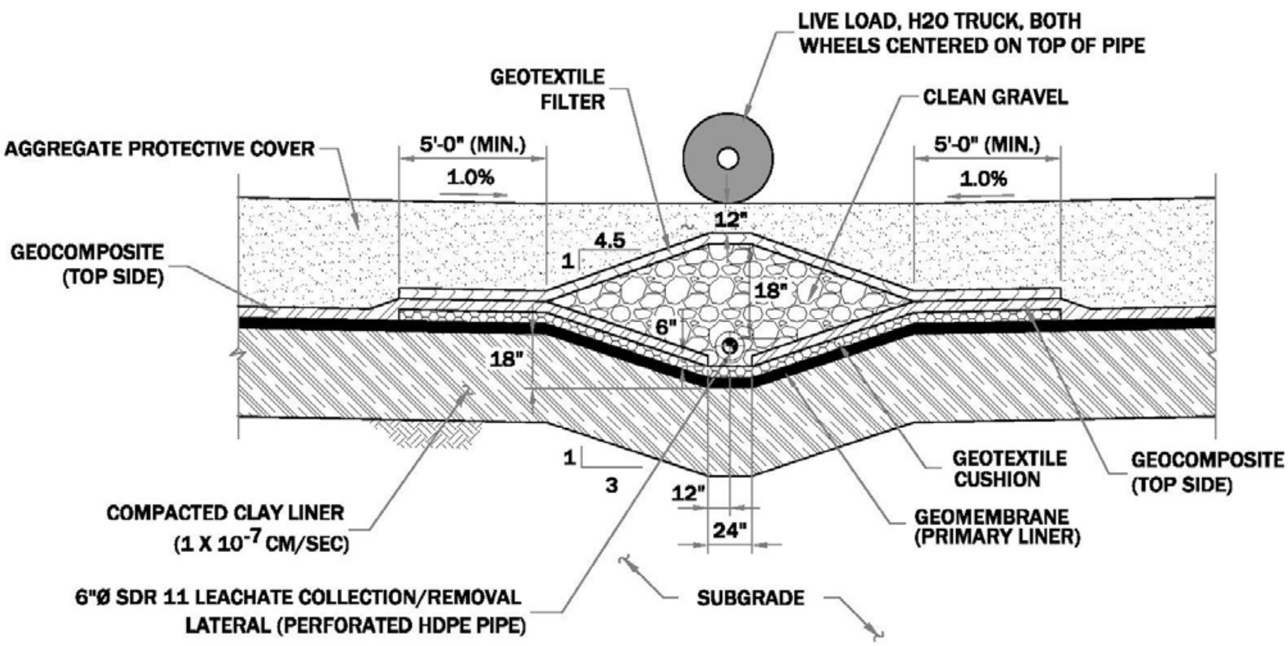
B' = Soil Support Factor = $\frac{1}{1+4e^{-0.065H}}$ = $\frac{1}{1+4e^{-0.065*4.0}}$ = 0.245

E = Apparent Modulus of Elasticity of Pipe Material = 21,000 psi (Assume 100 yrs, 73°F)

E' = Modulus of Soil Reduction = 2,000 psi (Assuming compacted coarse grained soil)

3.5 Scenario Sketches

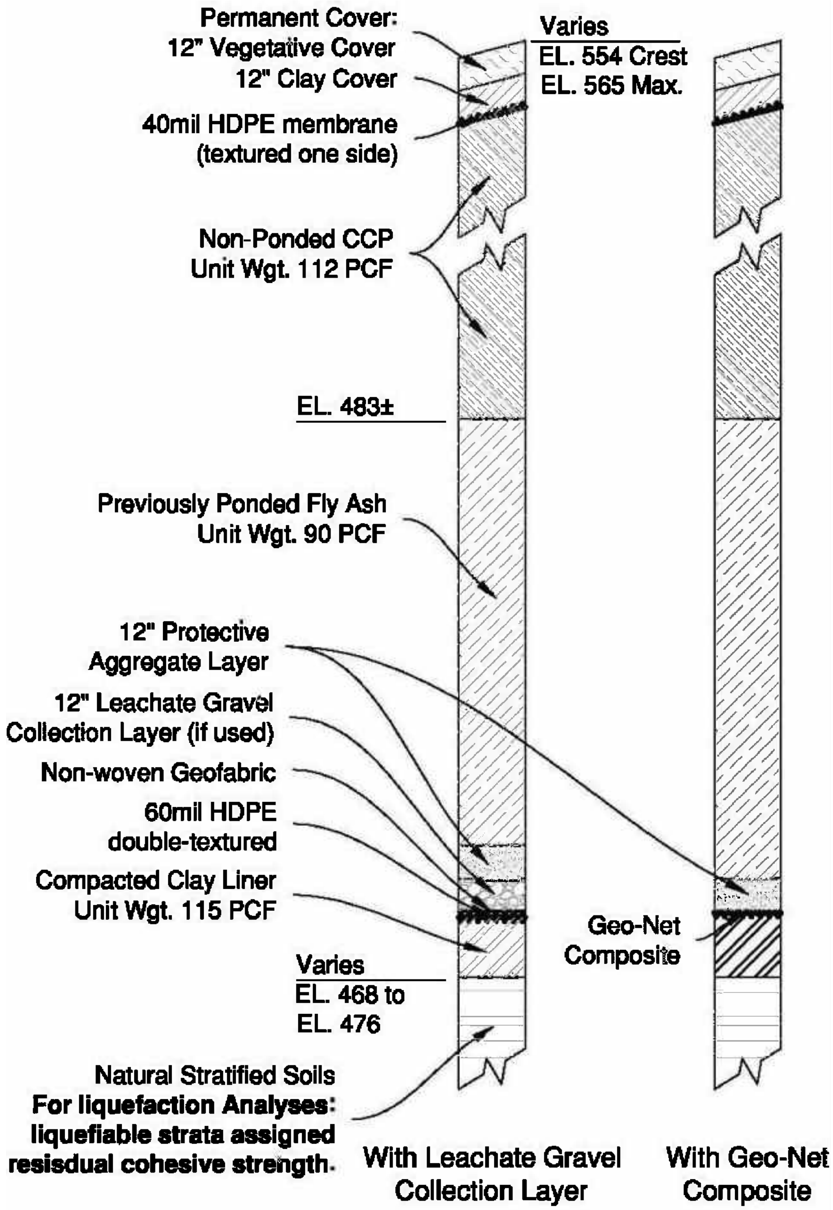
Scenario 1



LEACHATE COLLECTION LATERAL TRENCH (ALTERNATE)

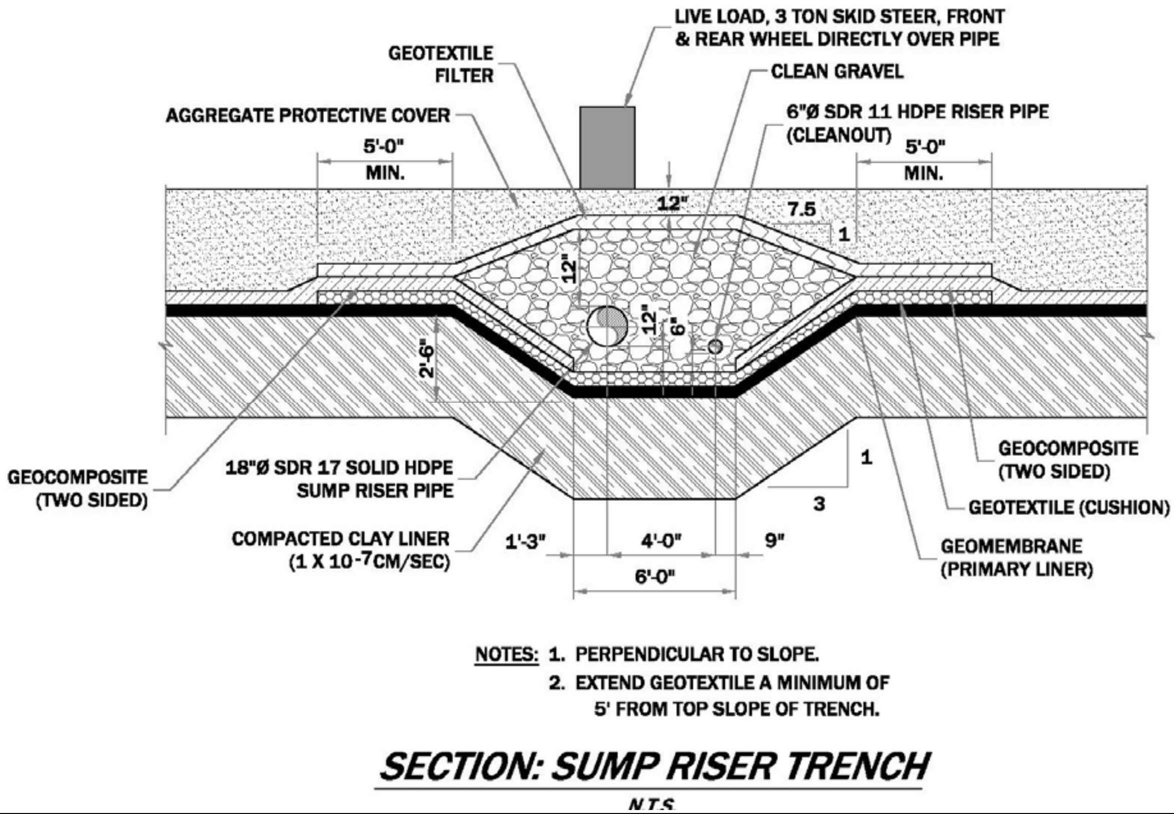
N.T.S.

Scenario 2

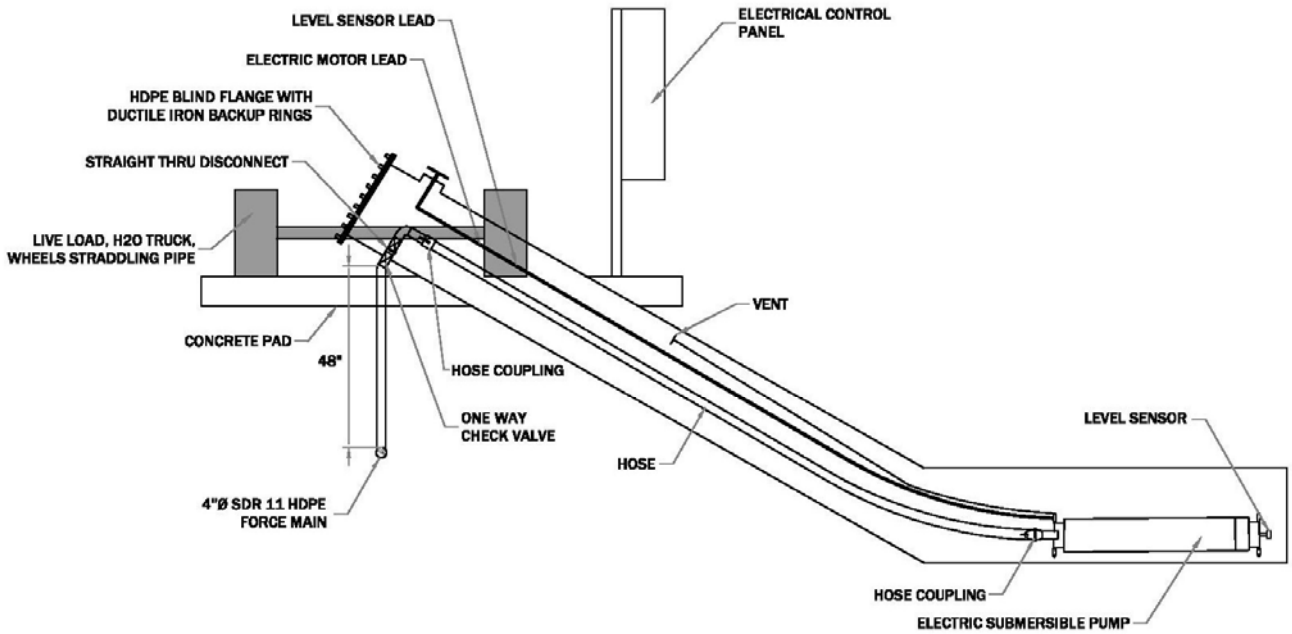


Typical Cell Material Profile
NOT TO SCALE

Scenario 3


LIVE LOAD, 3 TON SKID STEER, FRONT & REAR WHEEL DIRECTLY OVER PIPE
GEOTEXTILE FILTER
CLEAN GRAVEL
AGGREGATE PROTECTIVE COVER
6"Ø SDR 11 HDPE RISER PIPE (CLEANOUT)
5'-0" MIN.
5'-0" MIN.
12"
7.5
1
12"
12"
6"
2'-6"
GEOCOMPOSITE (TWO SIDED)
18"Ø SDR 17 SOLID HDPE SUMP RISER PIPE
COMPACTED CLAY LINER (1 X 10-7CM/SEC)
1
3
GEOCOMPOSITE (TWO SIDED)
GEOTEXTILE (CUSHION)
GEOMEMBRANE (PRIMARY LINER)
1'-3"
4'-0"
9"
6'-0"
NOTES: 1. PERPENDICULAR TO SLOPE.
2. EXTEND GEOTEXTILE A MINIMUM OF 5' FROM TOP SLOPE OF TRENCH.
SECTION: SUMP RISER TRENCH
N.T.S.

Scenario 4



TYPICAL LEACHATE PUMP STATION
WITH LEACHATE FORCE MAIN
N.T.S.

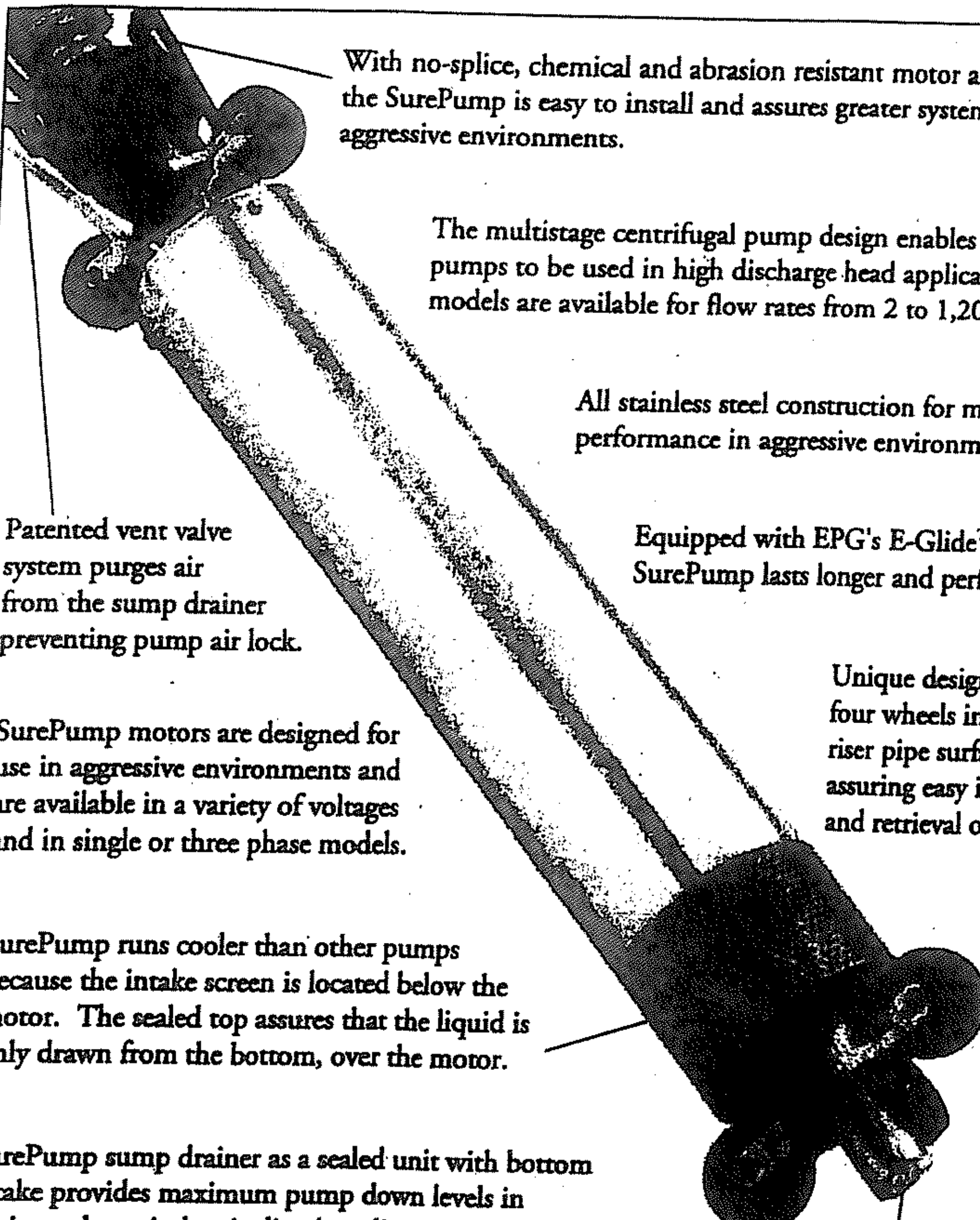
Supplemental Information for Appendix Y(a)

Pipe and Pump Calculations

From Sioux Const. Permit Mod. Report Feb 2012

Horizontal Wheeled Sump Drainer

More details at www.epgco.com



With no-splice, chemical and abrasion resistant motor and sensor leads, the SurePump is easy to install and assures greater system integrity in aggressive environments.

The multistage centrifugal pump design enables smaller diameter pumps to be used in high discharge head applications. SurePump models are available for flow rates from 2 to 1,200 gpm.

All stainless steel construction for maximum performance in aggressive environments.

Patented vent valve system purges air from the sump drainer preventing pump air lock.

Equipped with EPG's E-Glide™ bearings, the SurePump lasts longer and performs better.

Unique design places at least four wheels in contact with riser pipe surface at all times assuring easy installation and retrieval of the pump.

SurePump motors are designed for use in aggressive environments and are available in a variety of voltages and in single or three phase models.

SurePump runs cooler than other pumps because the intake screen is located below the motor. The sealed top assures that the liquid is only drawn from the bottom, over the motor.

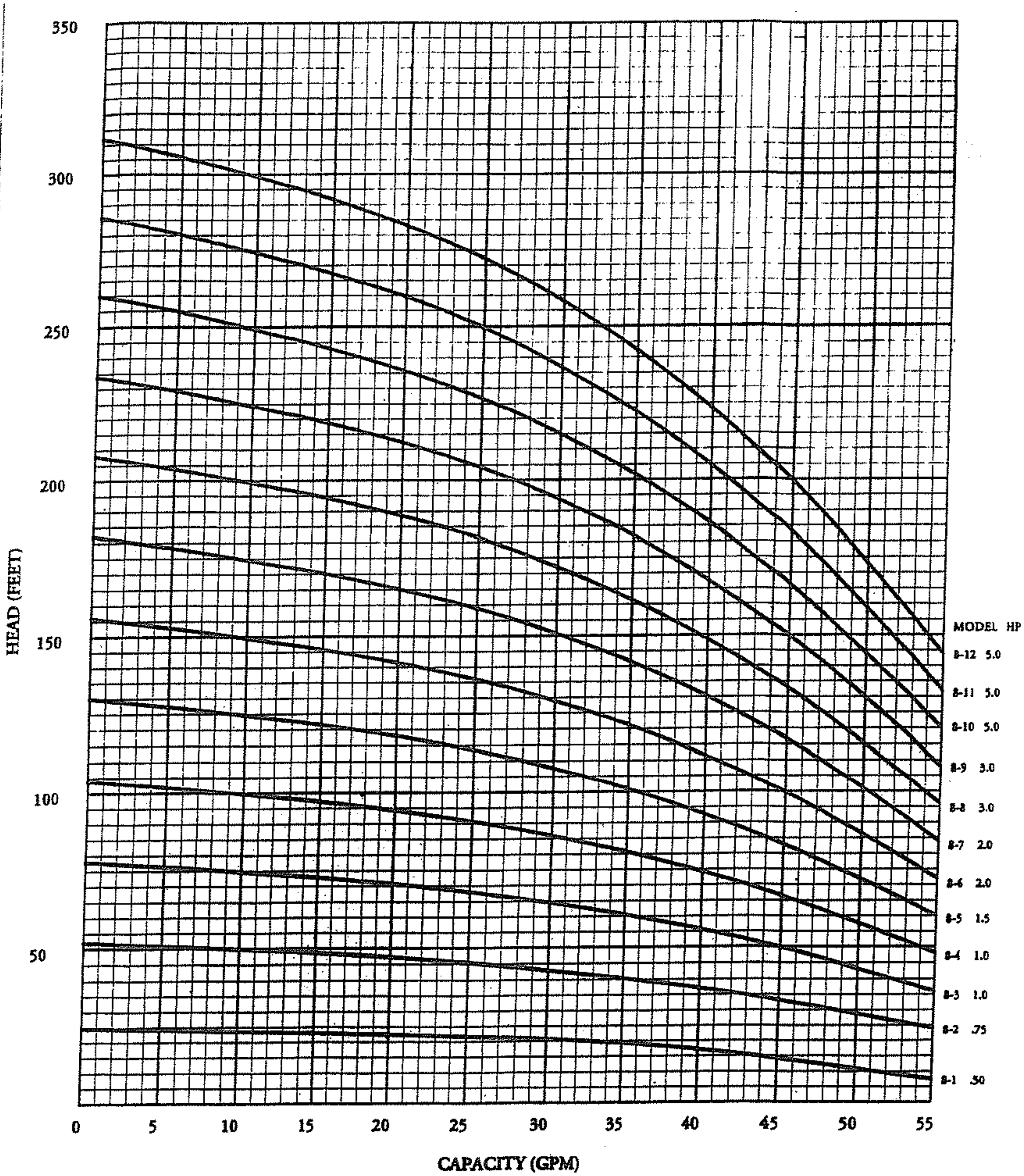
SurePump sump drainer as a sealed unit with bottom intake provides maximum pump down levels in horizontal, vertical or inclined applications.

The patented submersible level sensor is mounted along the central axis of the sump drainer, is removable from the bottom and assures accurate, repeatable level control.

SERIES 8 SurePump™

Flow Range 20-50 GPM

60 Hz



DATA SUBJECT TO CHANGE WITHOUT NOTICE

Pump Capacities

If the following curves do not meet your needs, please call us at 800-443-7426 and ask for an application specialist. Custom pumps in additional sizes, flow rates and head are available.

SurePump™

Curve	Model	Flow Range
0571[illegible]-0000	SERIES 1	1 to 7 [illegible]
05770-0000	SERIES 1.5	3 to 10 GPM
05771-0000	SERIES 2	4 to 14 GPM
05772-0000	SERIES 3	10 to 20 GPM
05773-0000	SERIES 5	15 to 30 GPM
05774-0000	SERIES 8	20 to 50 GPM
05775-0000	SERIES 12	35 to 75 GPM
05776-0000	SERIES 15	45 to 95 GPM

Curve	Model	Flow Range
0577[illegible]	SERIES 17	[illegible] GPM
05778-0000	SERIES 30	50 to 200 GPM
057[illegible]	[illegible]	75 to [illegible] GPM
05780-0000	SERIES 60	50 to 400 GPM
057[illegible]	[illegible] 77	75 to [illegible] GPM
05782-0000	SERIES 95	95 to 680 GPM
057[illegible]3-0000	SERIES 125	125 to [illegible] GPM

The SurePumps are available in the following configurations:

WSDPT: Wheeled Sump Drainer with integral level sensor for side slope riser applications

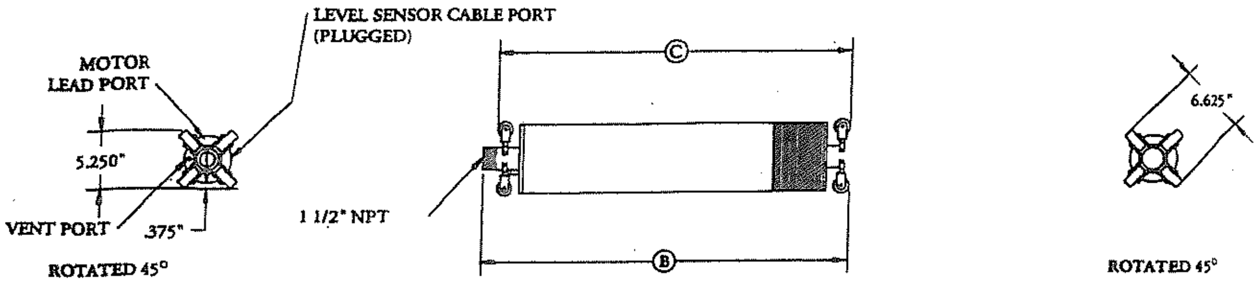
WSD: Wheeled Sump Drainer without integral level sensor for side slope riser applications

VSDPT: Vertical Sump Drainer with integral level sensor

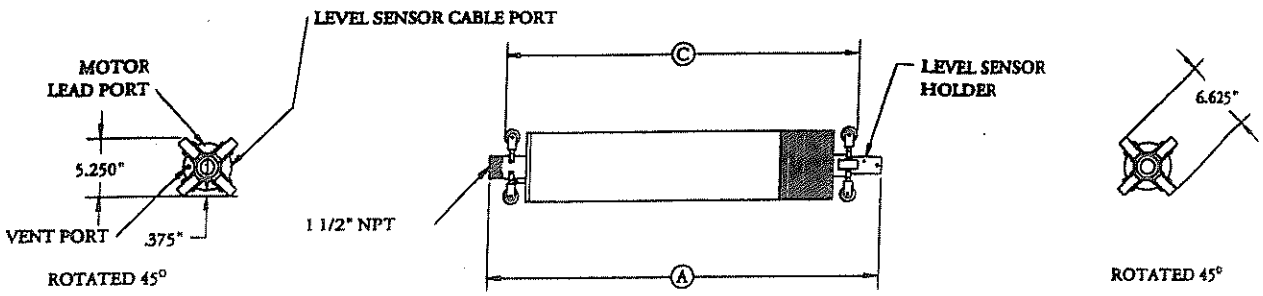
VSD: Vertical Sump Drainer with no level sensor

TSP: Submersible Pump

SERIES 8 SIZE 4 WHEELED SUMP DRAINER



WSD



WSDPT

MODEL	HP	PHASE	A	B	C	*APPROX. SHIPPING WEIGHT	
						WSD	WSDPT
8-1	0.50	1	32.72	31.60	30.85	62.79	67.79
8-1	0.50	3	32.72	31.60	30.85	62.79	67.79
8-2	0.75	1	35.50	34.38	33.63	68.95	68.95
8-2	0.75	3	35.50	34.38	33.63	68.95	68.95
8-3	1.00	1	38.24	37.12	36.37	75.08	80.08
8-3	1.00	3	38.24	37.12	36.37	75.08	80.08
8-4	1.00	1	39.89	38.77	38.02	77.36	82.36
8-4	1.00	3	39.89	38.77	38.02	77.36	82.36
8-5	1.50	1	43.41	42.29	41.54	86.09	91.09
8-5	1.50	3	41.54	40.42	39.67	79.64	84.64
8-6	2.00	1	46.56	45.44	44.69	92.53	97.53
8-6	2.00	3	45.06	43.94	43.19	88.37	93.37

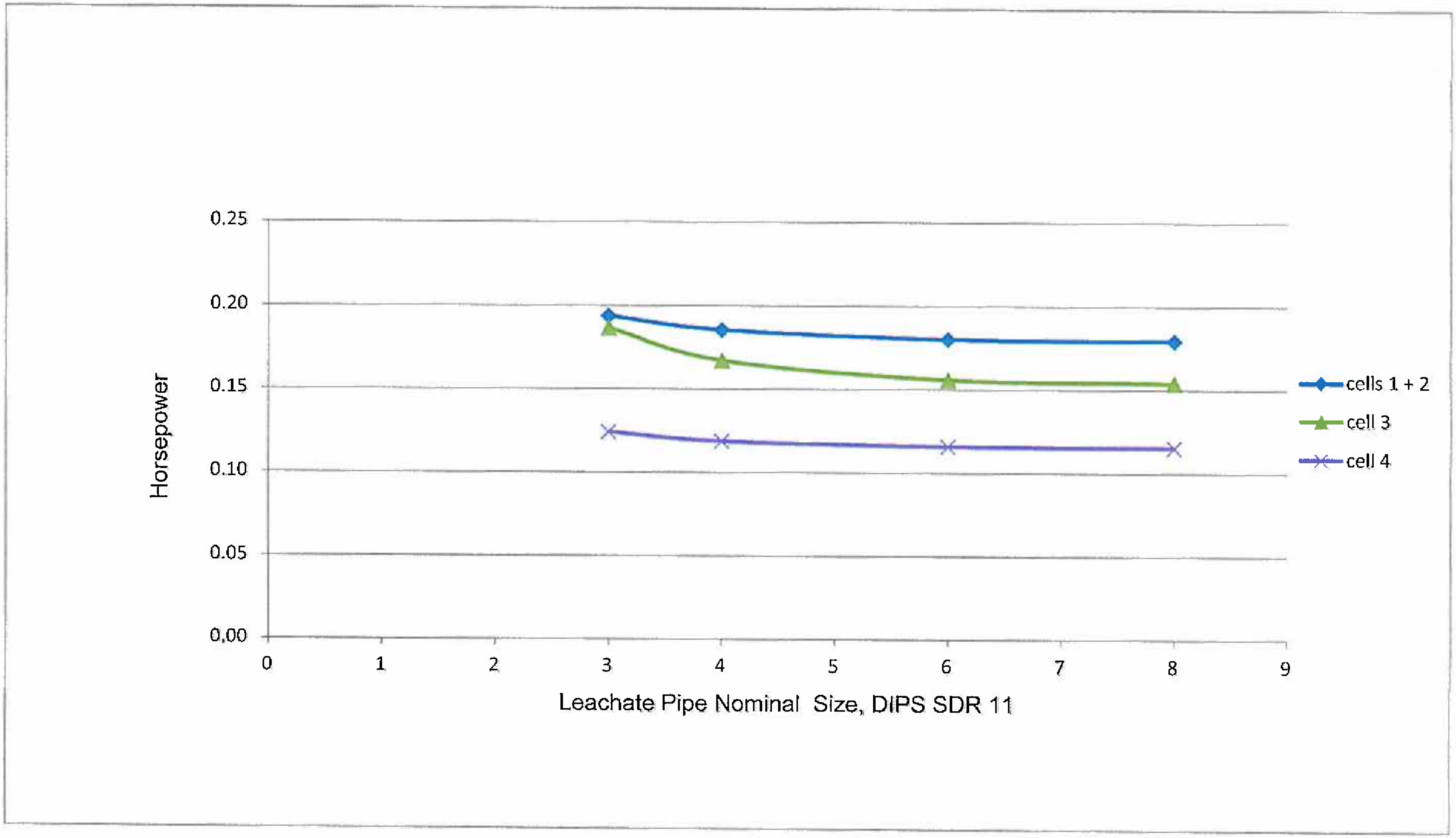
MODEL	HP	PHASE	A	B	C	*APPROX. SHIPPING WEIGHT	
						WSD	WSDPT
8-7	2.00	1	48.21	47.09	46.34	94.81	99.81
8-7	2.00	3	46.71	45.59	44.84	90.65	95.65
8-8	3.00	1	58.36	57.24	56.49	126.69	131.69
8-8	3.00	3	55.36	54.24	53.49	113.36	118.36
8-9	3.00	1	60.01	58.89	58.14	128.97	133.97
8-9	3.00	3	57.01	55.89	55.14	115.64	120.64
8-10	5.00	1	67.66	66.54	65.79	150.91	155.91
8-10	5.00	3	61.66	60.54	59.79	131.25	136.25
8-11	5.00	1	69.31	68.19	67.44	153.19	158.19
8-11	5.00	3	63.31	62.19	61.44	133.53	138.53
8-12	5.00	1	70.96	69.84	69.09	155.47	160.47
8-12	5.00	3	64.96	63.84	63.09	135.81	140.81

NOTE: ALL DIMENSIONS ARE IN INCHES.

*SHIPPING WEIGHT INCLUDES
WSD: CRATE, 50' OF 14-4 MOTOR LEAD, 50' OF 1/8" SS CABLE.
WSDPT: CRATE, 50' OF 14-4 MOTOR LEAD, 50' OF 1/8" SS CABLE, LEVEL SENSOR AND CABLE.

Ameren Missouri Labadie UWL
Leachate Pump Calculation

Pump Horsepower for leachate lines for cells to leachate holding at Pond 1



Prepared byGREDELL Engineering Resoruces, Inc.

January 2013

GREDELL Engineering Resources, Inc.
ENVIRONMENTAL ENGINEERING LAND - AIR - WATER
Telephone (573) 659-9078

Date: 11/6/2012		Page No: 1 of 2	
Client: Reitz & Jens			
Prepared By: FP	Checked By:	Project: Ameren MO Labadie CCPLF	
Subject: Check Calculations of Pipe Size and Pump Power for Leachate Collection			

Area of Cell 1 31.4 Ac

Measured	Sf	Ac		Ac Corrected
680, 1370, 0.5	465800	14.98	31.4 / 30.38	= 15.5
140, (1370+1300)/2	186900	Sump 2		
390, (1300+1130)/2	413100	15.4	31.4 / 30.38	15.9
520, 990, 0.5	257400	Sump 1		
		30.38		31.4

Flow from HELP Model OGE3 R003

Peak 612.5 gpad → 0.00095 cfs/Ac

Annual 209.8 gpad → 0.00032 cfs/Ac

	Ac	cfs/sump	Total flow
Sump 1	15.9 × 0.00095	.0151	0.0632
Sump 2	15.5	.0147	0.0481
	Add Cell 2	.0334	

4" SDR 11 DIPS Effective Diameter = 3.89" = 0.32' ISCO

$$\text{Area} = \frac{\pi d^2}{4} = 0.082 \text{ sf}$$

$$\text{Velocity} = Q/A = 0.0632/0.082 = 0.77 \text{ fps}$$

$$\text{Reynold's No} = \frac{Vd}{V_K}$$ where V_K = Kinematic Viscosity = 0.00001217 ft²/sec at 60° Vermand Fluid mechanics

$$R_E = \frac{(0.77)(0.32)}{1.217 \times 10^{-5}} = 20400$$

Printed on Recycled Paper

Date: 11/6/2012		Page No: 2 of 2	
Client: Reitz & Jens			
Prepared By: FD	Checked By:	Project: Ameren MO Labadie CCP LF	
Subject: Check Calculations of Pipe Size and Pump Power for Leachate Collection			

Velocity and friction head

e = roughness $= 7 \times 10^{-5}$ ft

$d = 0.32$ ft

$e/d = \frac{7 \times 10^{-5}}{3.2 \times 10^{-1}} = 2.2 \times 10^{-4} = 0.000202$

entering Moody's Diagram for $Re = 20{,}400$ and $e/d = 0.0002$ $f = 0.025$

$$\left(1 + f\frac{L}{d}\right)\frac{v^2}{2g} = \left(1 + \frac{0.025\,(140)}{0.32}\right)\frac{(0.77)^2}{2(32.2)} =$$

g = gravity constant 32.2 ft/sec^2

$$= \frac{(1 + 10.8)(0.59)}{64.4} = 0.11 \text{ ft}$$

$$= \left(1 + \frac{0.028\,(140)}{0.324}\right)\frac{(0.765)^2}{64.4}$$

$$(1 + 12.2)\frac{0.585}{64.4} = 0.12 \text{ ft}$$

Pipe Type:	DIPS
Pipe Diameter:	3
Pipe SDR:	SDR 11
Flow Rate:(GPM)	28.3
Pipe Length:(ft)	140

ISCO INDUSTRIES

Pipe I. D.:(in)	3.21	
Wall Thickness:(in)	0.360	
Pressure Rating:	160	
Flow Velocity:(ft/sec)	1.12	$= (q^{0.408709}/d^2)$
Head loss: (ft/100 feet)	0.164	$= .2083(100/C)^{1.852} x (q^{1.852}/d^{4.8655})$
System Pressure loss: (psi)	0.10	

Calculate Reset Print Form

Disclamier: The calculations in this program are, to the best of our knowledge, current and represent calculations normally used to size high-density polyethylene pipe. ISCO Industries, LLC does not accept responsibility for the use and/or application of these programs. Each project or applicatin has its own set of conditions and input variables. The intrepretation and use of these input

Pipe Type:	DIPS
Pipe Diameter:	4
Pipe SDR:	SDR 11
Flow Rate:(GPM)	28.3
Pipe Length:(ft)	140

ISCO INDUSTRIES

Pipe I. D.:(in)	3.89	
Wall Thickness:(in)	0.436	
Pressure Rating:	160	
Flow Velocity:(ft/sec)	0.76	$= (q^{0.408709}/d^2)$
Head loss: (ft/100 feet)	0.065	$= .2083(100/C)^{1.852} x (q^{1.852}/d^{4.8655})$
System Pressure loss: (psi)	0.04	

Calculate | Reset | Print Form

Disclamier: The calculations in this program are, to the best of our knowledge, current and represent calculations normally used to size high-density polyethylene pipe. ISCO Industries, LLC does not accept responsibility for the use and/or application of these programs. Each project or applicatin has its own set of conditions and input variables. The intrepretation and use of these input

Pipe Type:	DIPS
Pipe Diameter:	6
Pipe SDR:	SDR 11
Flow Rate:(GPM)	28.3
Pipe Length:(ft)	140

Pipe I. D.:(in)	5.60	
Wall Thickness:(in)	0.627	
Pressure Rating:	160	
Flow Velocity:(ft/sec)	0.37	$= (q^{0.408709}/d^2)$
Head loss: (ft/100 feet)	0.011	$= .2083(100/C)^{1.852} x (q^{1.852}/d^{4.8655})$
System Pressure loss: (psi)	0.01	

Calculate Reset Print Form

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Pipe Type:	DIPS
Pipe Diameter:	8
Pipe SDR:	SDR 11
Flow Rate:(GPM)	28.3
Pipe Length:(ft)	140

ISCO INDUSTRIES

Pipe I. D.:(in)	7.34	
Wall Thickness:(in)	0.823	
Pressure Rating:	160	
Flow Velocity:(ft/sec)	0.21	$= (q^{0.408709}/d^2)$
Head loss: (ft/100 feet)	0.003	$= .2083(100/C)^{1.852} x (q^{1.852}/d^{4.8655})$
System Pressure loss: (psi)	0.00	

Calculate Reset Print Form

Disclamier: The calculations in this program are, to the best of our knowledge, current and represent calculations normally used to size high-density polyethylene pipe. ISCO Industries, LLC does not accept responsibility for the use and/or application of these programs. Each project or applicatin has its own set of conditions and input variables. The intrepretation and use of these input

Ameren MO Labadie Energy Center
Computer Worksheet
Leachate Pumping to Holding Tank(s)

e, roughness=	0.00007	ft.		Technical literature from Chevron Petroleum Company
Flow pre acre	0.0009	cfs/acre	From HELP Model for Operational Condition, run OGE3R003	
		Gallon / acre / d	612.5	209.8
Tank on saddles	10,000	cfs/acre	0.000947743	0.000324631
length	30	Flow condition	peak	average
height	4	Sample	OGE3R003	OGE3R003

	DIPS	SDR 11		
	3	4	6	8
Diameter in inches	3.21	3.89	5.6	7.34
Diameter in feet	0.27	0.32	0.47	0.611666667
Area in square feet	0.056	0.083	0.171	0.294

		Area for sump		Distances to tanks		Flow of leachate		
		Measured	Corrected	Total Distance	Incremental distance			
		Acres		feet				
cell 1		31.4		.		Flow/ sump , cfs	Total flow at sump,cfs	Total flow at sump,gpm
	1	15.4	15.9	140	140	0.02	0.06	28.33
	2	15	15.5	630	490	0.01	0.05	21.56
cell 2		35.2						
	1	11.1	10.9	2300	1670	0.01	0.03	14.97
	2	11.6	11.4	2800	500	0.01	0.02	10.32
	3	13.1	12.9	3200	400	0.01	0.01	5.48
			35.2					
cell 3		57.1						
	1	8.7	8.9	2850	2850	0.01	0.05	24.29
	2	10.3	10.6	3350	500	0.01	0.05	20.49
	3	10.3	10.6	3840	490	0.01	0.04	15.99
	4	10.0	10.2	4330	490	0.01	0.03	11.48
	5	10.2	10.4	4820	490	0.01	0.02	7.15
	6	6.3	6.4	5310	490	0.01	0.01	2.72
			57.1					
cell 4		42.8						
	1	8.8	8.4	1650	1650	0.01	0.04	18.20
	2	9.7	9.3	2150	500	0.01	0.03	14.61
	3	11.8	11.4	2660	510	0.01	0.02	10.64
	4	14.2	13.7	3160	500	0.01	0.01	5.81

		3.21	3.89	5.6	7.34	Reynolds's Number 3.21	3.89	5.6	7.34
cell 1	1	1.12	0.76	0.37	0.21	25,000	20,000	14,000	11,000
	2	0.85	0.58	0.28	0.16	19,000	16,000	11,000	8,000
cell 2						-	-	-	-
	1	0.59	0.40	0.20	0.11	13,000	11,000	7,000	6,000
	2	0.41	0.28	0.13	0.08	9,000	7,000	5,000	4,000
	3	0.22	0.15	0.07	0.04	5,000	4,000	3,000	2,000
cell 3	1	0.96	0.66	0.32	0.18	21,000	17,000	12,000	9,000
	2	0.81	0.55	0.27	0.16	18,000	15,000	10,000	8,000
	3	0.63	0.43	0.21	0.12	14,000	11,000	8,000	6,000
	4	0.46	0.31	0.15	0.09	10,000	8,000	6,000	4,000
	5	0.28	0.19	0.09	0.05	6,000	5,000	4,000	3,000
	6	0.11	0.07	0.04	0.02	2,000	2,000	1,000	1,000
cell 4	1	0.72	0.49	0.24	0.14	16,000	13,000	9,000	7,000
	2	0.58	0.39	0.19	0.11	13,000	11,000	7,000	6,000
	3	0.42	0.29	0.14	0.08	9,000	8,000	5,000	4,000
	4	0.23	0.16	0.08	0.04	5,000	4,000	3,000	2,000

		e/D 3.21	3.89	5.6	7.34	Friction factor 3.21	3.89	5.6	7.34
cell 1	1	0.000261682	0.000215938	0.00015	0.00011444	0.026	0.028	0.028	0.028
	2	0.000261682	0.000215938	0.00015	0.00011444	0.027	0.031	0.032	0.033
cell 2	1	0.000261682	0.000215938	0.00015	0.00011444	0.029	0.033	0.034	0.035
	2	0.000261682	0.000215938	0.00015	0.00011444	0.032	0.038	0.037	0.037
	3	0.000261682	0.000215938	0.00015	0.00011444	0.036	0.042	0.042	0.042
cell 3	1	0.000261682	0.000215938	0.00015	0.00011444	0.027	0.029	0.03	0.032
	2	0.000261682	0.000215938	0.00015	0.00011444	0.029	0.03	0.03	0.032
	3	0.000261682	0.000215938	0.00015	0.00011444	0.03	0.032	0.032	0.038
	4	0.000261682	0.000215938	0.00015	0.00011444	0.032	0.034	0.038	0.04
	5	0.000261682	0.000215938	0.00015	0.00011444	0.038	0.038	0.044	0.044
	6	0.000261682	0.000215938	0.00015	0.00011444	0.05	0.05	0.065	0.065
cell 4	1	0.000261682	0.000215938	0.00015	0.00011444	0.028	0.028	0.032	0.034
	2	0.000261682	0.000215938	0.00015	0.00011444	0.031	0.031	0.032	0.035
	3	0.000261682	0.000215938	0.00015	0.00011444	0.032	0.034	0.035	0.04
	4	0.000261682	0.000215938	0.00015	0.00011444	0.038	0.04	0.065	0.065

		Incremental head loss for velocity and friction				Total head including elevation head with diameter of tank and 1 foot for tank saddle				Horsepower			
										3	4	6	8
		3.21	3.89	5.6	7.34	3.21	3.89	5.6	7.34	3.21	3.89	5.6	7.34
cell 1	1	0.29	0.12	0.02	0.01	27.05	25.89	25.15	25.04	0.19	0.19	0.18	0.18
	2	0.57	0.25	0.04	0.01	26.77	25.78	25.13	25.03				
cell 2	1	1.00	0.43	0.07	0.02	26.19	25.52	25.09	25.02				
	2	0.16	0.07	0.01	0.00	25.20	25.09	25.01	25.00				
	3	0.04	0.02	0.00	0.00	25.04	25.02	25.00	25.00				
cell 3	1	4.16	1.71	0.29	0.08	30.37	27.19	25.37	25.10	0.19	0.17	0.16	0.15
	2	0.57	0.22	0.04	0.01	26.21	25.49	25.08	25.02				
	3	0.35	0.14	0.02	0.01	25.65	25.26	25.05	25.01				
	4	0.19	0.08	0.01	0.00	25.30	25.12	25.02	25.01				
	5	0.09	0.03	0.01	0.00	25.10	25.04	25.01	25.00				
	6	0.02	0.01	0.00	0.00	25.02	25.01	25.00	25.00				
cell 4	1	1.41	0.54	0.10	0.03	26.94	25.75	25.14	25.04	0.12	0.12	0.12	0.12
	2	0.31	0.12	0.02	0.01	25.54	25.21	25.04	25.01				
	3	0.17	0.07	0.01	0.00	25.23	25.09	25.02	25.01				
	4	0.06	0.02	0.01	0.00	25.06	25.02	25.01	25.00				

CIVIL ENGINEERING HANDBOOK

LEONARD CHURCH URQUHART, C.E., Editor-in-Chief

Consulting Engineer, Porter, Urquhart, McCreary & O'Brien
Newark, N.J.; Los Angeles, San Francisco, and Sacramento, Calif.

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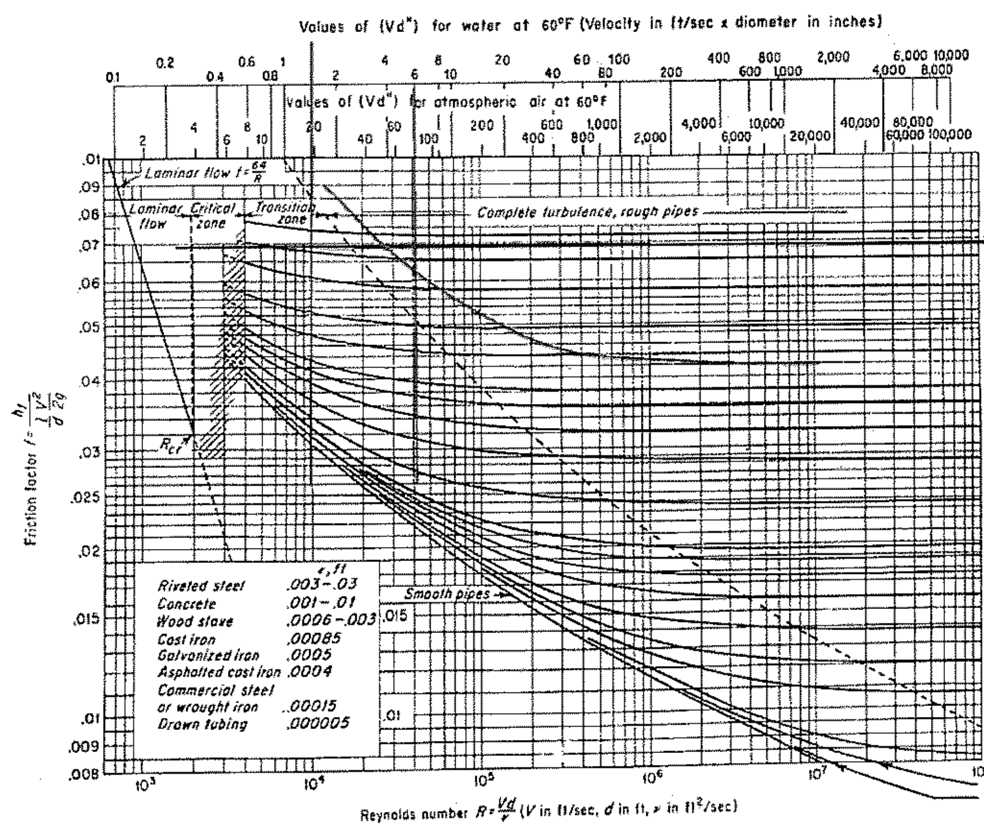


Fig. 61. Pipe friction coefficients.

Moody's chart. It is taken from the reference below.
ed to find friction factors for water in long pipes.

Appendix Y(b)

Estimated Maximum Settlements
Leachate Collection Pipe Profile

Ameren Missouri Labadie Energy Center
Utility Waste Landfill
Franklin County, MO
January 2013

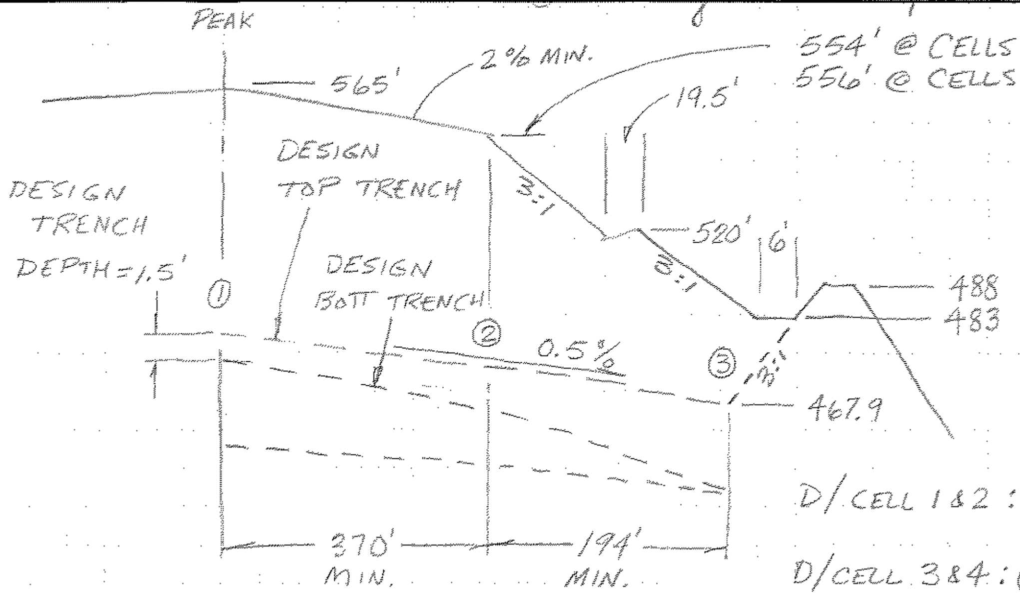
Estimated Maximum Settlements for Leachate Collection Pipe Profile Appendix Y(b)

A graphical analysis of the effect of long-term settlement due to the weight of utility waste on the leachate collection pipe is shown on the following page and detail number 1 on sheet 18 of the drawings. Analysis indicated that long-term settlement would cause a negative pipe slope within the approximately 180 feet of pipe length upstream of the leachate collection sump. The negative slope occurs because the maximum long-term settlement is estimated to be approximately 2.2 feet on the interior of the landfill, while it is estimated to be approximately 0.8 feet at the leachate collection sump. If the pipe were installed with a 0.5% slope running all the way to the sump, future settlement could reduce the final pipe slope to about -0.2%.

To mitigate this risk, it is proposed to slightly steepen the design slope of the leachate collection trenches from the head of the collection pipe to the sumps in order to provide a minimum post-construction, post-settlement pipe grade of 0.5%. The proposed design trench bottom grades are elevation 465.0 at the sump low point and elevation 467.0 at an inflection point 200 feet upstream from the sump low point. Laying out the trenches and sumps for excavation to these fixed elevations will provide a maximum installed leachate collection line slope of 0.6% from the head of the line to the inflection point, and a slope of 1.0% from the inflection point to the sumps.

GREDELL Engineering Resources, Inc.
ENVIRONMENTAL ENGINEERING — LAND - AIR - WATER
Telephone (573) 659-9078

Date: 11/15/12	Page No: 1 of 1		
Client: Reitz & Jens			
Prepared By: BD	Checked By:	Project: Labadie UWL	
Subject: Leachate Trench Grading to Compensate UWL Settlement			



See Appendix J for Settlement Estimates

D/CELL 1 & 2 : (554 - 483) 3 + 19.5 + 6 - (483 - 46.8) 3 = 193.5'

D/CELL 3 & 4 : (556 - 483) 3 + 19.5 + 6 - (483 - 468) 3 = 199.5'

LOCATION →	①		②		③	
TOP TRENCH PRE-S EL.	470.72		468.87		467.90	
BOTT TRENCH PRE-S EL.	469.22		467.37		466.40	(BASED ON 0.5% INITIAL SLOPE ON BOTT TRENCH)
MAX. EST. S	2.17		2.17		1.42	
MIN. EST. S	1.67		1.50		0.83	
BOTT TRENCH MAX. POST-S EL.*	467.55		465.87		465.57	* w/o DESIGN GRADE COMPENSATION
SLOPE, %		0.45		0.15 ← UNACCEPTABLE		
BOTT TRENCH MIN. POST-S EL.*	467.05		465.20		464.98	* w/o DESIGN GRADE COMPENSATION
SLOPE, %		0.50		0.11 ← UNACCEPTABLE		
MAX ΔS PER X-SECTS		0.42'		0.83'		
PROP. BOTT TRENCH PER MAX ΔS	469.22	0.61%	466.95	0.96%	465.04	
THEOR. POST-S EL. PER MAX ΔS	467.05	0.50%	465.20	0.50%	464.21	

FOR DETAIL 1/18 :
SET DESIGN BOTT TR @ ② = 467.0'
" " " " @ ③ = 465.0'
REPORT SLOPE ① → ② @ 0.6% (design)
REPORT SLOPE ② → ③ @ 1.0%

Appendix Y(c)

Water Management
Calculations
Revised November 2013

Ameren Missouri Labadie Energy Center
Utility Waste Landfill
Franklin County, MO
January 2013, Revised November 2013

Appendix Y(c)
Water Management Calculations

Leachate and stormwater are planned to be managed on site. The following calculations describe the capacity needed for water storage and pumping.

1. Leachate Flow

Cell 1 HELP Model results for leachate flow:

- Operational condition is worst case (Appendix O, Sub Appendix O -11).

- For the Operational condition for Cell 1:

    - Geocomposite drainage layer on the bottom and side slope:

        - Peak Daily Volume is: 13.4 gpm or 19,296 gpd.
        - Average Annual Volume is: 321,394 cu. ft./yr or 2,404,000 gallons per year.

    - Aggregate material drainage layer on the bottom and a geocomposite layer on the side slope:

        - Peak daily leachate flow is: 11.7 gpm or 16,848 gpd.
        - Average Annual Volume is: 320,708 cu. ft./yr or 2,399,000 gallons per year.

For Cell 3 at 57.1 acres; estimate the leachate volumes by pro-rating maximum peak daily flows using the ratio of the size of Cell 3 (57.1 acres) to Cell 1 (31.4 acres) or 1.819 (rounded).

- Peak daily leachate flow is: 21.3 gpm or 30,672 gpd.
- Average Annual Volume is: 584,616 cu. ft./yr or 4,367,000 gallons per year.

2. Stormwater Flow

Cell 1 HELP Model results for stormwater flow:

- Peak daily stormwater runoff is: 1,683,913 gpd

Estimating the maximum daily stormwater runoff using the ratio of Cell 3 to Cell 1:

- Peak daily stormwater runoff is: 3,063,000 gpd.

3. Estimate the Volume of Onsite Reuse of Leachate and/or Stormwater Runoff

Onsite water (leachate or stormwater runoff) usage:

- Reuse for CCP Moisture Conditioning Estimate:
  - The daily CCP generation rate for the first five years: 2,300,000 CY. The ratio of fly ash to bottom ash is 70% to 30%.
  - Therefore, the annual volume of fly ash generation is:
  - (2,300,000 CY / 5) x 0.70 = 322,000 CY/ year:
  - At 22% moisture by volume for conditioning, the water usage is 70,840 CY per year or 1,912,680 cubic feet (cf) or 14,306,846 gallons per year for CCP moisture conditioning.
  - This equates to an average daily water demand of 39,200 gpd, or an average flow rate of 27 gpm.

- Usage for Dust Control on Haul Roads:
  - Onsite water usage for dust control on onsite haul roads:
  - Assume an application rate of 0.25 inches (depth) per hour applied 6 hours per day or 1.5 inches per day.
  - Assume a road width of 12 feet and a ¼ mile of onsite haul road.
  - Volume is 12' x 1320' x0.25"/hr x 6 hr/day/(12"/1') = 1,980 cf per day
  - Volume is 1,980 cf/day x 7.48 gallons/cf = 14,810 gpd for dust control
  - Volume is 14,810 gpd x 5 days/wk x 52 weeks/yr = 3,850,600 gallons per year for dust control.
  - This equates to an average daily water demand of 10,550 gpd, or an average flow rate of 7.3 gpm.

Estimated Total Volume of Potential Onsite Reuse: 18,157,446 gallons per year or approximately 34.3 gpm.

4. Leachate Storage

The estimated required onsite leachate tank storage volume is calculated for the average annual volume from the HELP model results:

- Cell 1 Initial = 4.2 gpm x 1440 = 6,048 gpd = 2,207,520 gallons per year
- Cell 3 Initial = 6,048 x 57 ac/31 ac = 10,998 gpd = 4,014,312 gallons per year
- Cell 1 Operational = 4.6 gpm x 1440 = 6,624 gpd = 2,417,760 gallons per year
- Cell 3 Operational = 6,624 x 57 ac/ 31 ac = 12,180 gpd = 4,445,559 gallons per year

Therefore, a 10,000 gallon onsite storage tank will provide for an average of 0.8 days storage of the average annual leachate flow for Cell 3. One (1), 10,000 gallon horizontal tank would be 8 feet in diameter by 30 feet long. One or more tanks can be utilized based on the actual leachate flow and the demand for onsite reuse.

Backup leachate management will be at an off site POTW.

Backup stormwater management will be through the Labadie Energy Center's plant stormwater management system, which will be dependent on current NPDES operating permit requirements.

Appendix Y(d)

Flood Mitigation
Calculations
Revised August 2013

Ameren Missouri Labadie Energy Center
Proposed Utility Waste Landfill
Franklin County, Missouri
January 2013, Revised August 2013

Appendix Y(d)
Flood Mitigation Calculations

<u>Pumping Rates for Flood Water Protection – Cell 3</u>

Known:

Average Area of Cell 3 between floor and 480 ft. elev. = 49 ac

Average Bottom Elevation of Cell 3 from CADD surface = 471.2 ft

100-year Flood Elevation = 484 ft

Depth of water is estimated using the method described in Figure 7 of Appendix J. The density of water is substituted for the density of CCP to estimate the water fill depth need to protect against uplift during a flood. The inside toe of the slopes where the gravel drainage layer terminates is considered the critical location in the liner system that is most sensitive to hydrostatic uplift. The end-of-construction ballast against uplift at this location is equal to 2-feet of clay liner and 1-foot of protective cover. With estimated densities of 115 pounds per cubic foot (pcf) and 125 pcf, respectively, the ballast of 355 pounds per square foot (psf) at this location is the lowest at any point on the liner. Required elevations are determined by adding "H" values plus liner and cover thickness to elevation 466 feet.

$$H_{\text{Inside Cell}} = (H_{\text{Outside Cell}} \text{ x } 62.4 \text{ pcf x } 1.1 - 355 \text{ psf}) / 62.4 \text{ pcf}$$

$$H_{\text{Outside Cell}} = 484 \text{ ft} - 466 \text{ ft (lowest bottom of liner elevation)} = 18 \text{ ft}$$

$$H_{\text{Inside Cell}} = (18 \text{ ft x } 62.4 \text{ pcf x } 1.1 - 355 \text{ psf}) / 62.4 \text{ pcf} = 14.1 \text{ ft (elev. 483.1 ft)}$$

49 ac x 43,560 sf/ac x (483.1 - 471.2 ft) = 25,399,836 cf

25,399,836 cf x 7.48 gal/cf = 189,990,773 gal

Assume pumping will occur for 10 days, 24 hours per day:

10 days x 1,440 min/day = 14,400 min

Pumping rate = 189,990,773 gal / 14,400 min = 13,194 gpm

A pumping rate of 13,194 gpm, pumping 24 hours per day, is required to fill Cell 3 in 10 days for 100-year flood protection. High capacity pumps and power equipment necessary for pumping are readily available from equipment dealers and contractors within the St. Louis metropolitan area in the event of a major flood.

Fill Volume for Flood Mitigation

For each cell of the UWL, when there is an impending flood event that creates floodwater levels that exceed the minimum elevation of CCPs inside the active cell, CCPs will be placed at an accelerated rate in the active cell until it reaches an elevation sufficient to counterbalance uplift pressure during a flood. Again using the method described in Figure 7 of Appendix J, the minimum elevation of CCP's is determined as follows:

$$H_{CCP} = (H_{Outside\ Cell} \text{ x } 62.4 \text{ pcf x } 1.1 - 355 \text{ psf}) / 93.0 \text{ pcf}$$

$$H_{Outside\ Cell} = 484 \text{ ft} - 466 \text{ ft (lowest bottom of liner elevation)} = 18 \text{ ft}$$

$$H_{CCP} = (18 \text{ ft x } 62.4 \text{ pcf x } 1.1 - 355 \text{ psf}) / 93.0 \text{ pcf} = 9.5 \text{ ft (elev. 478.5 ft)}$$

A fill elevation of 478.5 feet provides sufficient ballast to resist the uplift pressure on the clay liner created by 100-year flood elevation of 484 feet, with a factor-of-safety of 1.1.

Fill volumes for each cell are estimated in the attached Table. Cell 3 has the largest estimated fill volume of 578,000 CY at elevation 478.5 ft. At a rate of 10,000 CY/day, it would take 58 days to fill to elevation 478.5 ft.

Flood Mitigation Culvert Design for Stormwater Ponds

The maximum anticipated rate of floodwater rise is estimated at 5-feet in 24-hours at the proposed site. To mitigate this flood risk, it is proposed to install pipe culverts with the capacity to intake water at a rate that will raise the pond levels at least 5-feet in 24 hours while limiting excess uplift head on the liner to less than 3-feet. The proposed pipe culverts were modeled with their flowline at elevation 472 feet, and a maximum headwater at the inlet of 2-feet.

The maximum volume in any 5-foot elevation interval in the stormwater ponds occurs in Pond 2. From elevation 478 feet to 483 feet, the volume is 19.8 acre-feet (see Table N-8, Appendix N). Based on a water elevation rise of 5 feet per day, the required inflow rate through a culvert in cubic feet per second (cfs) is:

$$(19.8 \text{ acre-feet/day})*(43{,}560 \text{ ft}^2/\text{acre})*(1 \text{ day}/24 \text{ hours})*(1 \text{ hour}/3600 \text{ sec}) = 10.0 \text{ cfs}$$

Based on the assumption of 2 feet of headwater on the pipe inlet at all times and an inflow discharge value of 10.0 cfs, the proposed diameter for a HDPE pipe culvert is 24 inches. Based on a pond berm design with a 12-foot top width at 488 elevation, 3:1 side slopes, and a culvert pipe at 472 elevation, the culvert pipe will be approximately 110 feet in length. A "duckbill" elastomeric valve is proposed to be installed on the culvert outlet to prevent backflow and subsequent loss of water. Additionally, a mechanical check valve is proposed to be installed in the pipe to control flow into the stormwater pond and to provide redundant backflow protection.

Solution of culvert design is by determination of flow under given headwater and tailwater conditions. The two critical conditions of flow through the proposed culvert are full pipe flow and partial pipe flow. These two conditions can be analyzed by their controlling element; inlet and/or outlet control.

Full pipe flow is a critical condition with submerged inlet and free fall outlet. This condition can be defined through a capacity equation given by:

$$q = a\sqrt{2gH} / \sqrt{1 + Ke + Kdv + KcL}$$

Where:

q=flow capacity (cfs)
a=conduit cross-sectional area (ft^2)
H=head causing flow (ft.) = 2' – 0.6*pipe diameter = 0.8'
Ke=entrance loss coefficient
Kc=friction loss coefficient from pipe
Kdv= duckbill valve friction loss coefficient
L=length of conduit (ft.)
g=acceleration due to gravity (32 ft/s^2)

$$q = \pi (1)^2 * \sqrt{2*32*.8}) / \sqrt{1+.78+1.0+(0.0165*110)})$$

q=10.50 cfs

Friction loss due to the mechanical check valve does exist, however the loss values are negligible. Under the conditions of full pipe flow, a 24-inch diameter design culvert is acceptable since the pipe discharge, q (10.5 cfs) is greater than the calculated minimum pond inflow requirement of 10.0 cfs.

Under submerged inlet and submerged outlet conditions, H=2 ft. and the outlet flow capacity using the above equation is 16.6 cfs, which exceeds the 10 cfs minimum pond inflow requirement.

The second critical flow condition is orifice controlled partial flow. This condition is illustrated by a submerged inlet and a free fall outlet. This condition can be defined by a capacity equation given as:

$$q = aC\sqrt{2gh}$$

Where:

q=flow capacity (cfs)
a=conduit cross-sectional area (ft^2)
C=coefficient for a sharp-edged orifice (0.6)
g=acceleration due to gravity (32 ft/s^2)
h= head to the center of the orifice (ft.)

$$q = \pi (1)^2 * 0.6 * \sqrt{2*32*1}$$

q=15.1 cfs

Under the conditions of orifice controlled partial flow, a 24-inch diameter culvert is acceptable since $q_{outflow}$ (15.1 cfs) is greater than the required q_{inflow} (10.0 cfs). The value of h=1 foot is the minimum value for a 24" culvert under the specified condition. As h increases, the outflow capacity increases, which continues to satisfy the condition of outflow capacity > inflow capacity.

Ameren Missouri Labadie Energy Center
Proposed Utility Waste Landfill
Flood Elevation vs. Fill Volumes

Appendix Y(d)
January 2013

Cell	100-yr Flood Elevation (ft)	Required CCP Elevation (ft)	Mean EL. Cell Floor	Floor Area (acres)	Area at 480 EL. (acres)	Volume to Fill Cell (cy)	Time to Fill with Varying Daily Disposal Rates (days)				
							1,000 CY/day	2,000 CY/day	4,000 CY/day	8,000 CY/day	10,000 CY/day
1	484	478.5	471.1	24.9	27.0	311,000	311	156	78	39	32
2	484	478.5	471.5	31.2	33.8	368,000	368	184	92	46	37
3	484	478.5	471.2	46.9	51.3	578,000	578	289	145	73	58
4	484	478.5	471.5	37.7	40.8	444,000	444	222	111	56	45

Notes

Volumes are estimates only, based on:

- Areas from permit drawings.
- The mean cell floor elevations were determined from CADD surfaces.
- Cell fill volumes were estimated using the average-end-area method.
- For the purposes of this table, it was estimated that the minimum CCP elevation to prevent hydrostatic uplift of the liner is 478.5 ft.
- For the purposes of this table, the cell areas at 478.5 ft and 480 ft are considered equivalent.

Appendix Y(e)

Geosynthetic Design Calculations

Ameren Missouri Labadie Power Plant
Utility Waste Landfill
Franklin County, Missouri
January 2013

Appendix Y(e)
Geosynthetics Design Calculations

The following anchor trench and slope stability design is based on three-foot horizontal to one-foot vertical slope utilizing a 60-mil HDPE textured geomembrane, a 250-mil Geocomposite with double sided 6 ounce per square yard non-woven needlepunched geotextile, and a 40-mil geomembrane. The calculations were performed through use of the equations provided in the book "Designing with Geosynthetics". Three conditions were analyzed: bottom liner slope stability, anchor trench design for the utility waste landfill's bottom liner and internal tensile stress within the bottom liner side slope layers.

Reference:

1. Koerner, R.M., Designing with Geosynthetics, 5th Edition, Prentice Hall, Upper Saddle River, New Jersey, 2005
2. Koerner, R.M., Designing with Geosynthetics, 2nd Edition, Prentice Hall, Upper Saddle River, New Jersey, 1990
3. Coduto, D.P., Geotechnical Engineering Principles and Practices, Prentice Hall, Upper Saddle River, New Jersey, 1999
4. Held, R.J., Soil Survery of Franklin County, Missouri, United States Department of Agriculture: Soil Conservation Service, 1989
5. *GSE HD Smooth Geomembrane*; Product Data Sheet; GSE Lining Technology, LLC: Houston, TX, REV 5MAR2012.
6. *GSE HD Textured Geomembrane*; Product Data Sheet; GSE Lining Technology, LLC: Houston, TX, REV 09APR2012.
7. *GSE FabriNet HF Geocomposite*; Product Data Sheet; GSE Lining Technology, LLC: Houston, TX, REV 01MAY2012.

Prepared by:
GREDELL Engineering Resources, Inc.

January 2013

Appendix Y(e) Notes

Slope Stability of Liner, Anchor Trench Pullout, and Liner Layer Stress Calculation

Calculations Required:

1. Failure due to sliding of leachate collection protective cover.
2. Failure due to anchor trench pullout of geomembrane and geocomposite.
3. Failure due to tensile stress in liner layers.

1. Side Slope Cover Material Stability on 3(H):1(V) Slope

From Koerner (5th Edition) the stability of the system is achieved if all interface friction angles (δ) are greater than the slope angle (β). The Factor of Safety (F.S.) will be determined by the use of Equation 5.22 (pg. 492, 5th Ed.) where δ is the lowest numerical interface friction angle. Interface friction angles are taken from Table 5.6, Koerner, 2nd Edition, and Table 5.7, Koerner, 5th Edition.

$\beta = \tan^{-1}(\frac{1}{3}) = 18.43^{\circ}$

$\text{F.S.} = \dfrac{\tan\delta}{\tan\beta}$

$\delta_{clay\text{-}geomembrane} = 26^{\circ} > 18.43^{\circ}$

$\delta_{geomembrane\text{-}geotextile} = 32^{\circ} > 18.43^{\circ}$

$\delta_{geotextile\text{-}protectivecover} = 30^{\circ} > 18.43^{\circ}$

$\text{F.S.} = \dfrac{\tan 26^{\circ}}{\tan 18.43^{\circ}} = 1.5$

The slope is stable with a F.S. of 1.5.

2. Anchor Trench Depth and Runout Calculations

Check design detail to determine if proposed runout and anchor trench depth provides adequate F.S.

Koerner gives detailed equations for calculating required depth and runout on pgs. 500-506 (5th Ed.). Rearranging Eq. 5.26, one can solve for runout length (L_{RO}), anchor trench depth (d_{AT}), or allowable stress (T_{allow}). The allowable stress was solved for and input to a spreadsheet to expedite calculations. The equation was used as follows:

$$T_{allow} = \frac{d^{AT}[0.5 * \gamma_{AT}(K_P - K_A)] + d_{AT}[\sigma(K_P - K_A)] + L_{RO}[\sigma_n(\tan\delta_u + \tan\delta_L)]}{[\cos\beta - \sin\beta\tan\delta_L]}$$

Attached to these calculations are printouts of the inputs and results for this calculation.

In order to determine certain friction angles some assumptions were made about the material to be used for berm construction which affects the anchor trench soil as well as the cover soil on top of the liner runout. It was assumed that stock piled soil from the top 18 inches of onsite soil would be used.

Onsite soils are predominately Blake-Waldron Complex classification as determined using the cares website. USDA soil survey of Franklin County, Missouri (1989) defines Blake-Waldron as CL, CL-CH soil with plasticity indices ranging from 10-45 within the top 24", giving an average of approximately 26.0. For calculation purposes P_I was chosen to be 30.0. Using Fig. 13.17 from Coduto (pg. 489), this gives an effective friction angle of approximately 27°. This soil will also have a compacted unit weight of approximately 115 lb/ft^3.

The interface friction angle between the geomembrane and the material directly above and below it must be taken from published data until more site specific data are known. δ_L for the geomembrane-CCL inferface will be selected from Table 5.6 from Koener 2nd Edition. Detail 5/17 on Sheet 17 shows the geometry of the designed anchor trench and runout.

To determine if the liner or geocomposite will pullout of the anchor trench the calculated T_{allow} was compared to the T_{Design} obtained from the manufacturer's specifications. If $T_{allow} > T_{Design}$ the liner (or composite) will yield before anchor trench pullout occurs.

For 60 mil textured HDPE Geomembrane:

$$T_{Design} = 22 \text{ kN/m}$$
$$T_{allow} = 44.88 \text{ kN/m}$$
$$T_{allow} > T_{Design}, \text{ therefore no pullout}$$

$$\text{F.S.} = \frac{T_{allow}}{T_{Design}} = 2.0$$

For 250 mil Geocomposite with 6 oz/sq yd non-woven, needle-punched Geotextile:

$$T_{Design} = 9.60 \text{ kN/m}$$
$$T_{allow} = 49.31 \text{ kN/m}$$
$$T_{allow} > T_{Design}, \text{ therefore no pullout}$$

$$\text{F.S.} = \frac{T_{allow}}{T_{Design}} = 5.1$$

<u>3. Tensile Stress Calculations within Liner Layers</u>

$N = W\cos(\beta)$
$W = W_C - T_C$
$\beta = slopeangle = \tan^{-1}(1/3) = 18.43°$

$$H_C = HeightofCover = 2.0'$$
$$L_S = LengthofSlope = 53.8'$$
$$\gamma_c = 130\,pcf$$
$$\phi = 26°$$

$W_C = H_c L_s \gamma_c$

Tc=$\sigma_H \tan\phi H_C = K_0 \sigma_U (\tan\phi) H_C$

Tc = (1-sin(26))(2')(130 pcf)(2')(tan(26))= 142 lb/ft
Wc = (2')(53.8')(130 pcf) = 14,000 lb/ft

W = 14,000 – 142 = 13,858 lb/ft
N = 13,858*cos(18.43) = 13,147 lb/ft

a.) Shear Forces in Geocomposite

$$F_{above} = N * \tan(\delta_u)$$

$$F_{below} = N * \tan(\delta_L)$$

$$F_a = (13{,}147)\tan 25 = 6130.5\,lb/ft$$
$$F_b = (13{,}147)\tan 32 = 8215.2\,lb/ft$$

Therefore Geocomposite is not in tension

b.) Shear Forces in Geomembrane

$$F_{above} = F_{belowfromcomposite} = 8215.2\,lb/ft$$
$$F_{below} = N\tan\delta_u = (13{,}147)\tan 26 = 6412.2\,lb/ft$$
$$F_{above} > F_{below}$$

Therefore the Geomembrane is in tension

$$\sigma_{n,\max} = (\gamma_{ccp})(H_{ccp}) = (134\,pcf)(100') = 13{,}400\,psf = 93.1\,psi$$

$$\sigma_{all,membrane} = \frac{131\,lb/in}{0.06\,in} = 2138\,psi$$

F.S.=$\dfrac{\sigma_{all}}{\sigma_{\max}} = \dfrac{2183}{93.1} = 23.4$

Therefore the geomembrane is acceptable.

THESE CALCULATIONS FOR THE GEOSYNTHETICS IN THE COVER ARE SUPPLEMENTAL TO THOSE FOR THE LINER. THEY APPLY THE SAME METHODS.

① SIDE SLOPE COVER MATERIAL STABILITY

THE PREVIOUSLY CALCULATED SLOPE ANGLE IS 18.43°.

THE CCP IS ASSUMED TO BE SIMILAR TO SAND FOR THE PURPOSE OF ESTIMATING THE FRICTION ANGLE BETWEEN IT AND THE GEOMEMBRANE.

$$\delta_{GEOMEMBRANE-CCP} = 22° > 18.43°$$ (SAND FRICTION ANGLE ~ 30°)

$$F.S. = \frac{\tan 22°}{\tan 18.43°} = 1.2$$

$$\frac{\tan 30°}{\tan 18.43°} = 1.7$$

② ANCHOR TRENCH DEPTH AND RUNOUT

ANCHOR TRENCH AND RUNOUT CALCULATIONS WERE PREVIOUSLY CONDUCTED FOR 60-MIL HDPE. THE SAME METHODS WERE APPLIED TO THE 40-MIL GEOMEMBRANE [SEE ATTACHED PRINTOUT]

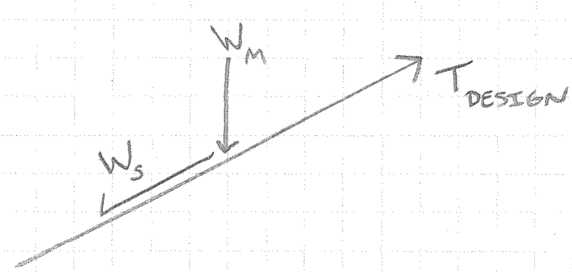
$$T_{ALLOW} = 24.72 \text{ kN/m}$$

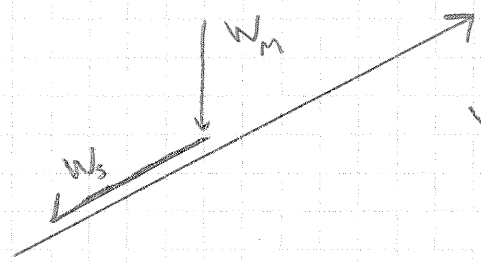
$$T_{DESIGN} = 15 \text{ kN/m}$$

$$T_{ALLOW} > T_{DESIGN}$$

$$F.S. = \frac{24.72 \text{ kN/m}}{15 \text{ kN/m}} = 1.6$$

THE LENGTH OF THE GEOMEMBRANE MUST BE SHORT ENOUGH SO ITS WEIGHT DOES NOT PULL OUT OF THE TRENCH





$W_S \leq T_{DESIGN}$

$$W_S = W_M \sin 18.43^\circ$$

$$W_M = \gamma t L$$

$$\approx 59 \text{ pcf} (40 \text{ mil}) \left(\frac{1 \text{ ft}}{12{,}000 \text{ mil}} \right) = (0.2 \text{ psf}) L$$

$$W_S = (0.2 \text{ psf}) L (\sin 18.43^\circ) = (0.06 \text{ psf}) L$$

$$T_{DESIGN} = 15{,}000 \text{ N/m} \left(\frac{1 \text{ lb}}{4.448222 \text{ N}} \right) \left(\frac{0.3048 \text{ m}}{\text{ft}} \right)$$

$$T_{DESIGN} = 1028 \text{ lb/ft}$$

$$(0.06 \text{ psf}) L \leq 1028 \text{ lb/ft}$$

$$\boxed{L \leq 17{,}133 \text{ ft}}$$

Ameren Missouri
Labadie Energy Center UWL
Anchor Trench & Runout Calculations for 40 mil Geomembrane

Design Data & Material Properties	
Allowable Stress in Geosynthetic (kPa), σ_{ALLOW}	14,763.8
Thickness of Geosynthetic (m), t_g	0.0010
Side Slope Ratio (V:H), 1:	3.00
Side Slope Angle (degrees), β	18.43
Angle of Shearing Resistance between Geosynthetic & adjacent material BELOW Geosynthetic (degrees), δ_L	22.00
Angle of Shearing Resistance between Geosynthetic & adjacent material ABOVE Geosynthetic (degrees), δ_U	0.00
Unit Weight of Runout Cover Material (kN/m^3), γ_{CM}	18.07
Thickness of Runout Cover Material (m), t_{CM}	0.61
Applied Normal Stress from Cover Material (kPa), σ_n	11.02
Unit Weight of Soil in Anchor Trench (kN/m^3), γ_{AT}	18.07
Angle of Shearing Resistance of Fill Soil in Trench (degrees), Φ_A (Typically the same as Φ_P)	22.00
Angle of Shearing Resistance of Soil in Trench Wall (degrees), Φ_P (Typically the same as Φ_A)	22.00
Allowable Force in Geosynthetic (kN/m), T_{DESIGN}	15.00
Active Earth Pressure from Trench Fill, K_A	0.45
Passive Earth Pressure from Trench Wall, K_P	2.20

Calculate Length of Runout (L_{RO}) for Given Depth of Anchor Trench (d_{AT})	
Depth of Anchor Trench (m), d_{AT}	
Length of Geosynthetic Runout Required (m), L_{RO}	0.00

Calculate Depth of Anchor Trench (d_{AT}) for Given Length of Runout (L_{RO})	
Length of Geosynthetic Runout (m), L_{RO}	
Depth of Anchor Trench Required (m), d_{AT}	0.00

Calculate Allowable Force in Geosynthetic and Factor of Safety	
Length of Geosynthetic Runout (m), L_{RO}	0.61
Depth of Anchor Trench (m), d_{AT}	0.61
Allowable Force in Geosynthetic (kN/m), T_{ALLOW}	24.72
Factor of Safety, F.S.	1.6

Ameren Missouri
Labadie Energy Center UWL
Anchor Trench & Runout Calculations for 60 mil Geomembrane

Design Data & Material Properties	
Allowable Stress in Geosynthetic (kPa), σ_{ALLOW}	15,333.3
Thickness of Geosynthetic (m), t_g	0.0015
Side Slope Ratio (V:H), 1:	3.00
Side Slope Angle (degrees), β	18.43
Angle of Shearing Resistance between Geosynthetic & adjacent material BELOW Geosynthetic (degrees), δ_L	26.00
Angle of Shearing Resistance between Geosynthetic & adjacent material ABOVE Geosynthetic (degrees), δ_U	0.00
Unit Weight of Runout Cover Material (kN/m^3), γ_{CM}	18.07
Thickness of Runout Cover Material (m), t_{CM}	0.91
Applied Normal Stress from Cover Material (kPa), σ_n	16.52
Unit Weight of Soil in Anchor Trench (kN/m^3), γ_{AT}	18.07
Angle of Shearing Resistance of Fill Soil in Trench (degrees), Φ_A (Typically the same as Φ_P)	27.00
Angle of Shearing Resistance of Soil in Trench Wall (degrees), Φ_P (Typically the same as Φ_A)	27.00
Allowable Force in Geosynthetic (kN/m), T_{DESIGN}	23.00
Active Earth Pressure from Trench Fill, K_A	0.38
Passive Earth Pressure from Trench Wall, K_P	2.66

Calculate Length of Runout (L_{RO}) for Given Depth of Anchor Trench (d_{AT})	
Depth of Anchor Trench (m), d_{AT}	
Length of Geosynthetic Runout Required (m), L_{RO}	0.00

Calculate Depth of Anchor Trench (d_{AT}) for Given Length of Runout (L_{RO})	
Length of Geosynthetic Runout (m), L_{RO}	
Depth of Anchor Trench Required (m), d_{AT}	0.00

Calculate Allowable Force in Geosynthetic and Factor of Safety	
Length of Geosynthetic Runout (m), L_{RO}	0.61
Depth of Anchor Trench (m), d_{AT}	0.61
Allowable Force in Geosynthetic (kN/m), T_{ALLOW}	44.88
Factor of Safety, F.S.	2.0

Ameren Missouri
Labadie Energy Center UWL
Anchor Trench & Runout Calculations for 250 mil Geocomposite

Design Data & Material Properties	
Allowable Stress in Geosynthetic (kPa), σ_{ALLOW}	1,523.8
Thickness of Geosynthetic (m), t_g	0.0063
Side Slope Ratio (V:H), 1:	3.00
Side Slope Angle (degrees), β	18.43
Angle of Shearing Resistance between Geosynthetic & adjacent material BELOW Geosynthetic (degrees), δ_L	32.00
Angle of Shearing Resistance between Geosynthetic & adjacent material ABOVE Geosynthetic (degrees), δ_U	0.00
Unit Weight of Runout Cover Material (kN/m^3), γ_{CM}	18.07
Thickness of Runout Cover Material (m), t_{CM}	0.91
Applied Normal Stress from Cover Material (kPa), σ_n	16.52
Unit Weight of Soil in Anchor Trench (kN/m^3), γ_{AT}	18.07
Angle of Shearing Resistance of Fill Soil in Trench (degrees), Φ_A (Typically the same as Φ_P)	27.00
Angle of Shearing Resistance of Soil in Trench Wall (degrees), Φ_P (Typically the same as Φ_A)	27.00
Allowable Force in Geosynthetic (kN/m), T_{DESIGN}	9.60
Active Earth Pressure from Trench Fill, K_A	0.38
Passive Earth Pressure from Trench Wall, K_P	2.66

Calculate Length of Runout (L_{RO}) for Given Depth of Anchor Trench (d_{AT})	
Depth of Anchor Trench (m), d_{AT}	
Length of Geosynthetic Runout Required (m), L_{RO}	0.00

Calculate Depth of Anchor Trench (d_{AT}) for Given Length of Runout (L_{RO})	
Length of Geosynthetic Runout (m), L_{RO}	
Depth of Anchor Trench Required (m), d_{AT}	0.00

Calculate Allowable Force in Geosynthetic and Factor of Safety	
Length of Geosynthetic Runout (m), L_{RO}	0.61
Depth of Anchor Trench (m), d_{AT}	0.61
Allowable Force in Geosynthetic (kN/m), T_{ALLOW}	49.31
Factor of Safety, F.S.	5.1

PRODUCT DATA SHEET

GSE HD Smooth Geomembrane

METRIC

GSE HD is a smooth high density polyethylene (HDPE) geomembrane manufactured with the highest quality resin specifically formulated for flexible geomembranes. This product is used in applications that require excellent chemical resistance and endurance properties.

[*]

AT THE CORE:
An HDPE geomembrane used in applications that require excellent chemical resistance and endurance properties.

Product Specifications

These product specifications meet GRI GM 13

Tested Property	Test Method	Frequency	Minimum Average Value				
			0.75 mm	1.00 mm	1.50 mm	2.00 mm	2.50 mm
Thickness, (minimum average), mm Lowest individual reading	ASTM D 5199	every roll	0.750 0.675	1.00 0.90	1.50 1.35	2.00 1.80	2.50 2.25
Density, g/cm³	ASTM D 1505	90,000 kg	0.940	0.940	0.940	0.940	0.940
Tensile Properties (each direction) Strength at Break, N/mm Strength at Yield, N/mm Elongation at Break, % Elongation at Yield, %	ASTM D 6693, Type IV Dumbbell, 50 mm/min G.L. 50 mm G.L. 33 mm	9,000 kg	 20 11 700 12	 27 15 700 12	 40 22 700 12	 53 29 700 12	 67 37 700 12
Tear Resistance, N	ASTM D 1004	20,000 kg	93	125	187	249	311
Puncture Resistance, N	ASTM D 4833	20,000 kg	240	320	480	640	800
Carbon Black Content, % (Range)	ASTM D 1603*/4218	9,000 kg	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0
Carbon Black Dispersion	ASTM D 5596	20,000 kg	Note(1)	Note(1)	Note(1)	Note(1)	Note(1)
Notch Constant Tensile Load, hr	ASTM D 5397, Appendix	90,000 kg	300	300	300	300	300
Oxidative Induction Time, min	ASTM D 3895, 200°C; O_2, 1 atm	90,000 kg	>100	>100	>100	>100	>100
TYPICAL ROLL DIMENSIONS							
Roll Length(2), m			341	265	171	131	104
Roll Width(2), m			6.86	6.86	6.86	6.86	6.86
Roll Area, m²			2,341	1,819	1,171	899	711

NOTES:
- (1)Dispersion only applies to near spherical agglomerates. 9 of 10 views shall be Category 1 or 2. No more than 1 view from Category 3.
- (2)Roll lengths and widths have a tolerance of ±1%.
- GSE HD Smooth is available in rolls weighing approximately 1,800 kg.
- All GSE geomembranes have dimensional stability of ±2% when tested according to ASTM D 1204 and LTB of <-77° C when tested according to ASTM D 746.
- *Modified.

GSE is a leading manufacturer and marketer of geosynthetic lining products and services. We've built a reputation of reliability through our dedication to providing consistency of product, price and protection to our global customers.

Our commitment to innovation, our focus on quality and our industry expertise allow us the flexibility to collaborate with our clients to develop a custom, purpose-fit solution.

[DURABILITY RUNS DEEP] For more information on this product and others, please visit us at GSEworld.com, call 800.435.2008 or contact your local sales office.

GSE ENVIRONMENTAL™

PRODUCT DATA SHEET

GSE HD Textured Geomembrane

METRIC

GSE HD Textured is a co-extruded textured high density polyethylene (HDPE) geomembrane available on one or both sides. It is manufactured from the highest quality resin specifically formulated for flexible geomembranes. This product is used in applications that require increased frictional resistance, excellent chemical resistance and endurance properties.

[*]

AT THE CORE:
An HDPE geomembrane used in applications that require increased frictional resistance, excellent chemical resistance and endurance properties.

Product Specifications

These product specifications meet GRI GM13

Tested Property	Test Method	Frequency	Minimum Average Value				
			0.75 mm	1.00 mm	1.50 mm	2.00 mm	2.50 mm
Thickness, (minimum average), mm Lowest individual reading	ASTM D 5994	every roll	0.750 0.675	1.00 0.90	1.50 1.35	2.00 1.80	2.50 2.25
Density, g/cm^3 , (min.)	ASTM D 1505	90,000 kg	0.940	0.940	0.940	0.940	0.940
Tensile Properties (each direction) Strength at Break, N/mm Strength at Yield, N/mm Elongation at Break, % Elongation at Yield, %	ASTM D 6693, Type IV Dumbbell, 50 mm/min G.L. 50 mm G.L. 33 mm	9,000 kg	 8 11 100 12	 10 15 100 12	 16 22 100 12	 21 29 100 12	 26 37 100 12
Tear Resistance, N	ASTM D 1004	20,000 kg	93	125	187	249	311
Puncture Resistance, N	ASTM D 4833	20,000 kg	200	267	400	534	667
Carbon Black Content, % (Range)	ASTM D 1603*/4218	9,000 kg	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0
Carbon Black Dispersion	ASTM D 5596	20,000 kg	Note(1)	Note(1)	Note(1)	Note(1)	Note(1)
Asperity Height, mm	ASTM D 7466	second roll	0.40	0.45	0.45	0.45	0.45
Notch Constant Tensile Load(2), hr	ASTM D 5397, Appendix	90,000 kg	300	300	300	300	300
Oxidative Induction Time, min	ASTM D 3895, 200°C; O_2, 1 atm	90,000 kg	>100	>100	>100	>100	>100
TYPICAL ROLL DIMENSIONS							
Roll Length(3), m	Double-Sided Textured Single-Sided Textured		253 308	213 238	158 165	122 125	101 101
Roll Width(3), m			6.86	6.86	6.86	6.86	6.86
Roll Area, m^2	Double-Sided Textured Single-Sided Textured		1,736 2,113	1,461 1,633	1,084 1,132	837 858	693 693

NOTES:
- (1)Dispersion only applies to near spherical agglomerates. 9 of 10 views shall be Category 1 or 2. No more than 1 view from Category 3.
- (2)NCTL for GSE HD Textured is conducted on representative smooth geomembrane samples.
- (3)Roll lengths and widths have a tolerance of ±1%.
- GSE HD Textured is available in rolls weighing approximately 1,800 kg.
- All GSE geomembranes have dimensional stability of ±2% when tested according to ASTM D 1204 and LTB of <-77° C when tested according to ASTM D 746.
- *Modified.

GSE is a leading manufacturer and marketer of geosynthetic lining products and services. We've built a reputation of reliability through our dedication to providing consistency of product, price and protection to our global customers.

Our commitment to innovation, our focus on quality and our industry expertise allow us the flexibility to collaborate with our clients to develop a custom, purpose-fit solution.

GSE ENVIRONMENTAL™

[DURABILITY RUNS DEEP] For more information on this product and others, please visit us at GSEworld.com, call 800.435.2008 or contact your local sales office.

PRODUCT DATA SHEET

GSE FabriNet HF Geocomposite

METRIC

GSE FabriNet HF geocomposite consists of a 6.3 mm thick GSE HyperNet HF geonet heat-laminated on one or both sides with a GSE nonwoven needle-punched geotextile. The geotextile is available in mass per unit area range of 200 g/m² to 540 g/m². The geocomposite is designed and formulated to perform drainage function under a range of anticipated site loads, gradients and boundary conditions.

[*]

AT THE CORE:

A 6.3 mm thick GSE HyperNet HF geonet heat-laminated on one or both sides with a nonwoven needle-punched geotextile.

Product Specifications

Tested Property	Test Method	Frequency	Minimum Average Roll Value		
Geocomposite			200 g/m²	270 g/m²	335 g/m²
Transmissivity(2), m²/sec Double-Sided Composite Single-Sided Composite	ASTM D 4716	1/50,000 m²	 5×10^{-4} 1.5×10^{-3}	 5×10^{-4} 1.5×10^{-3}	 3×10^{-4} 1×10^{-3}
Ply Adhesion, g/cm	ASTM D 7005	1/4,600 m²	178	178	178
Geonet Core(3) – GSE HyperNet HF					
Transmissivity(2), m²/sec	ASTM D 4716		3×10^{-3}	3×10^{-3}	3×10^{-3}
Density, g/cm³	ASTM D 1505	1/4,600 m²	0.94	0.94	0.94
Tensile Strength (MD), N/mm	ASTM D 5035/7179	1/4,600 m²	9.6	9.6	9.6
Carbon Black Content, %	ASTM D 1603(6)/4218	1/4,600 m²	2.0	2.0	2.0
Geotextile(3,4)					
Mass per Unit Area, g/m²	ASTM D 5261	1/8,300 m²	200	270	335
Grab Tensile, N	ASTM D 4632	1/8,300 m²	710	975	1,155
Puncture Strength, N	ASTM D 4833	1/8,300 m²	395	525	725
AOS, US sieve(1) (mm)	ASTM D 4751	1/50,000 m²	0.212	0.180	0.150
Permittivity, (sec⁻¹)	ASTM D 4491	1/50,000 m²	1.5	1.3	1.0
Flow Rate, lpm/m²	ASTM D 4491	1/50,000 m²	4,480	3,865	3,050
UV Resistance, % retained	ASTM D 4355 (after 500 hours)	once per formulation	70	70	70
NOMINAL ROLL DIMENSIONS					
Geonet Core Thickness, mm	ASTM D 5199	1/4,600 m²	6.3	6.3	6.3
Roll Width(5), m			4.5	4.5	4.5
Roll Length(5), m	Double-Sided Composite Single-Sided Composite		70.1 79.2	64.0 79.2	64.0 76.2
Roll Area, m²	Double-Sided Composite Single-Sided Composite		321 362	293 362	293 348

[Product specifications continued on back]

[*]

AT THE CORE:
A 250 mil thick HyperNet HF geonet heat-laminated on one or both sides with a nonwoven needlepunched geotextile.

Product Specifications [continued]

NOTES:
- [(1)]AOS in mm is a maximum average roll value.
- [(2)]Gradient of 0.1, normal load of 10,000 psf, water at 70˚F between steel plates for 15 minutes. Contact GSE for performance transmissivity value for use in design.
- [(3)]Component properties prior to lamination.
- [(4)]Refer to geotextile product data sheet for additional specifications.
- [(5)]Roll widths and lengths have a tolerance of \1%.
- [(6)]Modified.