APPENDIX J

SUMP AND RISER CALCULATIONS FOR SCA DESIGN

GEOSYNTEC CONSULTANTS

COMPUTATION COVER SHEET

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SUMP AND RISER CALCULATIONS FOR SCA DESIGN

INTRODUCTION

This package was prepared in support of the design of the Sediment Consolidation Area (SCA) for the Onondaga Lake Bottom Site, which will be constructed on Wastebed 13 (WB-13) to contain dredged material from the Lake. This package presents analysis of the proposed liquid management system (LMS) sump and riser of the SCA, which will utilize geotextile tubes (geo-tubes) for dewatering. The purpose of the analyses presented in this package is to:

1. Evaluate the hydraulic requirements for riser pipe perforations to handle the design inflow during post-closure conditions.
2. Evaluate the requirements for structural stability of the riser pipe, including: (i) ring compressive strain; (ii) ring bending strain; (iii) ring buckling; (iv) longitudinal (axial) strain and buckling; and (v) flotation.
3. Calculate the liquid storage volume and typical pump off and on times for the sumps during post-closure conditions.

The pumps, risers, and sumps analyzed herein are designed for the post-closure conditions. Interim liquid removal measures (e.g., temporary pumps) will be used during the operational period, as needed. These measures are not addressed in this package and will be addressed in the Sediment Management Intermediate and/or Final Design.

METHODOLOGY

The sump and riser calculations in this package include three steps: (i) sizing the riser pipe perforations to achieve the target inflow rates; (ii) calculating the dimensions and structural stability of the riser pipe; and (iii) calculating the volume of the sump.

Pipe Perforation Sizing:

The riser pipe perforations were sized to accommodate the target design inflow, following the general procedure of Bernoulli's Equation [Qian et al., 2002]. Initially, the area of each perforation A_b (ft^2) was calculated with an assumed diameter d (ft) as follows:

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$$A_b = \frac{\pi}{4} * d^2 \qquad (1)$$

Bernoulli's Equation was then used to calculate the inflow capacity of a single perforation of diameter d as follows:

$$Q_b = C * A_b * v_{ent} \qquad (2)$$

where:

Q_b = Inflow per orifice (ft^3/s)

C = Discharge coefficient, assumed to be 0.62 [Qian et al., 2002]

v_{ent} = Entry velocity (ft/s), assumed to be 0.1 ft/s [Qian et al., 2002]

The calculated maximum inflow capacity for a selected perforated pipe length L was then computed using Equation 3.

$$Q_{in} = Q_b * N_{row} * R * L \qquad (3)$$

where:

Q_{in} = Calculated maximum inflow capacity for the perforated length (ft^3/s)

N_{row} = Number of perforations per row $= \frac{360°}{\theta}$

θ = Angle between perforations within a row (°)

R = Number of rows per foot $= \frac{12}{\delta}$

δ = Offset between rows of perforations (in)

L = Perforated pipe length (ft)

The perforated pipe length L only takes into account the length between the automatic "off" elevation of the pump and the top of the sump excavation (i.e., the "on" elevation of the pump). It is noted that the perforations may extend to the bottom of the riser, however water may pond below the automatic "off" elevation of the pump. Therefore, the perforations below the automatic "off" elevation have been conservatively ignored for purposes of this calculation. The number of perforations per row and offset between rows for the proposed riser pipe was varied to analyze various possible pipe perforation options.

Pipe Structural Stability:

Calculations were performed to verify that the proposed high density polyethylene (HDPE) manhole riser pipes are able to withstand the loads applied on them with adequate factors of safety. Failure mechanisms that were checked include: (i) ring compressive strain; (ii) ring bending strain; (iii) ring buckling; (iv) longitudinal (axial) strain and buckling; and (iv) flotation. It is noted that the pipe structural stability is dependent on the standard dimension ratio (SDR) which is only available in specific values.

The radial pressure applied on the manhole riser may be approximated using the active earth pressure, computed by Equation 4 [Chevron Phillips, 2004]. It is noted that the coefficient of 1.21 in the equation is intended to account for the variability of active earth pressure around the riser. It is further noted that this equation has been modified to take into account strength reduction due to the perforations, which are assumed to extend the entire length of the riser [Qian et al., 2002]:

$$P_R = \frac{1.21 * K_A * \gamma * H}{1 - \frac{n * d}{12}} \tag{4}$$

where:

P_R	=	Radial pressure (psf);
γ	=	Soil unit weight (pcf);
H	=	Height of fill (ft);
K_A	=	Active earth pressure coefficient = $\tan^2(45 - \phi/2)$;
Φ	=	Soil friction angle (deg);
n	=	Number of perforations per lineal foot of pipe; and
d	=	Diameter of a single perforation (in).

The downdrag load applied to the manhole due to soil settlement is calculated assuming that the radially-directed pressure varies linearly with depth. Equation 5 can be used to calculate the average shear stress T_A applied to a manhole riser [Chevron Phillips, 2004]. The average shear stress may then be used in Equation 6 to calculate the downdrag load [Chevron Phillips, 2004].

$$T_A = \mu * \frac{(P_{R1} + P_{R2})}{2} \tag{5}$$

where:

T_A = Average shear stress (psf);
P_{R1} = Radial pressure at top of riser (psf);
P_{R2} = Radial pressure at base of riser (psf); and
μ = Friction coefficient between riser and soil.

$$P_D = T_A * \pi \frac{(D_O)}{12} * H \tag{6}$$

where:

P_D = Downdrag load (lb);
D_O = Outside diameter of riser (in);
T_A = Average shear stress (psf); and
H = Height of manhole fill (ft).

It is noted that μ of 0.4 is recommended for the coefficient of friction between HDPE manholes and granular or granular-cohesive backfills [Chevron Phillips, 2004]. It is further noted that the effect of water will reduce the downdrag load due to the buoyancy effect. However, the water level will fluctuate due to seasonal variations and the effect of pumping operations. Therefore, water has been conservatively ignored in calculation of the radial pressure, average shear stress, and downdrag load.

Ring Compressive Strain: Radially-directed earth loads cause ring compressive strain and bending strains in the riser. It is noted that ring compressive strain should be limited to 3.5% at 73°F [Chevron Phillips, 2004]. Equations 7 and 8 may be used to determine ring compressive thrust and ring compressive strain, respectively [Chevron Phillips, 2004].

$$N_T = \frac{P_R}{144} * (R_M) \tag{7}$$

where:

N_T = Ring compressive thrust (lb/in);
P_R = Radial pressure (psf);
R_M = Mean riser radius (in) = $\frac{ID + t}{2}$;

ID = Inside manhole diameter (in); and
t = Minimum wall thickness (in).

$$\varepsilon_T = \frac{N_T}{E * t} \tag{8}$$

where:

ε_T = Ring compressive strain (in/in);
N_T = Ring compressive thrust (lb/in);
E = Modulus of elasticity (psi); and
t = Minimum wall thickness (in).

Ring Bending: Ring deflection often occurs during installation, causing a bending moment within the riser. It is noted that ring bending strain can be added to ring compressive strain to calculate the combined strain, which should be limited to 5% at 73°F [Chevron Phillips, 2004]. Equations 9 and 10 may be used to determine ring bending moment and ring bending strain, respectively, assuming 2% deflection during installation [Chevron Phillips, 2004].

$$M_E = 0.25 * C_O * D_M * N_T \tag{9}$$

where:

M_E = Ring bending moment (lb-in/in);
C_O = Correction for 2% deflection = 0.02;
D_M = Mean riser diameter (in); and
N_T = Ring compressive thrust (lb/in).

$$\varepsilon_B = \frac{6 * M_E}{E * t^2} \tag{10}$$

where:

ε_B = Ring bending strain (in/in);
M_E = Ring bending moment (lb-in/in);
E = Modulus of elasticity (psi); and
t = Minimum wall thickness (in).

Ring Buckling: If ring compressive thrust exceeds a critical buckling value, the manhole riser may buckle. Ring buckling is calculated differently depending on whether the riser is above or below the water level. It is noted that the water level for this project will vary seasonally and based on operational conditions, therefore both calculations were performed and the more critical result was used in analysis. For a manhole riser completely above the water level, the critical ring compressive thrust may be calculated using Equation 11 [Chevron Phillips, 2004].

$$N_{CR} = 0.3 * R_H * t * E^{1/3} * E_S^{2/3} \tag{11}$$

where:

N_{CR}	=	Critical ring compressive thrust, no water (lb/in);
R_H	=	Geometry factor = 1.0 if relative stiffness (RS) is less than 0.005;
t	=	Minimum riser wall thickness (in);
E	=	Modulus of elasticity of soil (psi);
E_S	=	Young's Modulus of soil (psi);
RS	=	Relative stiffness = $\dfrac{0.22 * E * t^3}{E_S * R_M^3}$; and
R_M	=	Mean radius of riser (in).

If the manhole riser or a portion of the riser is below the water level, the critical compressive thrust may be calculated with Equation 12 [Chevron Phillips, 2004].

$$N_{CRW} = 0.82 * \sqrt{\frac{R * B' * E' * E * t^3}{D_M}} \tag{12}$$

where:

N_{CRW}	=	Critical ring compressive thrust with water (lb/in);
D_M	=	Mean diameter (in);
R	=	Buoyancy reduction = $1 - 0.33 * \dfrac{H'}{H}$;
H'	=	Height of water from invert (ft);
H	=	Height of manhole fill (ft);
E	=	Modulus of elasticity of soil (psi);
E'	=	Modulus of soil reaction (psi);
t	=	Minimum riser wall thickness (in); and
B'	=	$\dfrac{1}{1 + 4 * e^{(-0.065 * H)}}$.

It is noted that the ring compressive thrust N_T should not exceed 50% of the critical ring compressive thrust N_{CR} or N_{CRW} [Chevron Phillips, 2004]. It is further noted that while the manhole top and bottom may increase the stiffness and lower the amount of buckling observed, this is conservatively ignored for purposes of this calculation.

Longitudinal (Axial) Strain and Buckling: Longitudinal compressive strain may be caused by downdrag loads, dead loads (i.e., manhole weight), and any live loads (i.e., equipment and personnel on top of the manhole). Longitudinal compressive strains can be calculated using Equation 13 [Chevron Phillips, 2004].

$$\varepsilon_A = \frac{P_D + P_L + P_W}{E * \pi * D_M * t} \qquad (13)$$

where:

ε_A = Longitudinal compressive strain (in/in);
P_D = Downdrag force (lb);
P_L = Live load (lb);
P_W = Dead load (lb);
E = Modulus of elasticity of soil (psi);
D_M = Mean riser diameter (in); and
t = Minimum riser wall thickness (in).

Local wall buckling can occur if the longitudinal strain exceeds the critical longitudinal strain, calculated by Equation 14 [Chevron Phillips, 2004].

$$\varepsilon_{CR} = \frac{2 * t}{D_M * \sqrt{3 * (1 - \mu^2)}} \qquad (14)$$

where:

ε_{CR} = Critical longitudinal compressive strain (in/in);
D_M = Mean riser diameter (in);
μ = Poisson's ratio for HDPE; and
t = Minimum riser wall thickness (in).

It is noted that the use of granular soil or granular-cohesive soil surrounding the manhole will provide an additional safety factor not considered in this calculation; therefore the use of an additional safety factor is not required [Chevron Phillips, 2004]. Therefore, ε_A must be less than ε_{CR}. Additionally, ε_A must be less than 3.5%. It is noted that if the calculated strain ε_A fails to meet either condition, a thicker riser (i.e., smaller SDR) should be selected.

Flotation: When a manhole riser is surrounded by water, the buoyancy force may cause flotation if the downward forces (i.e., manhole weight, friction, etc.) are not sufficient to resist the buoyancy force due to the water. It is noted that flotation will only occur if water is between the riser bottom and the liner. Based on current designs, the riser will be placed directly on top of the liner, so flotation is not expected to be an issue. However, to be conservative, the effect of flotation has been calculated. The net downward (resisting) force can be calculated using Equation 15 [Chevron Phillips, 2004]. It is noted that a factor of safety of 1.5 is selected for both the weight of the anti-flotation slab and the soil weight.

$$F_{DOWN} = W_{MH} + W_{CC} + \frac{W_{AF}}{FS_{AF}} + \frac{W_{AFS}}{FS_{AFS}} \tag{15}$$

where:

- F_{DOWN} = Total downward force (lb);
- W_{MH} = Manhole weight (lb);
- W_{CC} = Weight of manhole cover (lb);
- W_{AF} = Weight of anti-flotation slab (lb);
- FS_{AF} = Factor of safety for anti-flotation slab weight;
- W_{AFS} = Weight of soil above anti-flotation slab (lb); and
- FS_{AFS} = Factor of safety for soil weight above anti-flotation slab.

Manhole weight and the weight of manhole cover can be obtained from the manufacturer of the riser and/or calculated from design drawings. It is noted that because the manhole cover will be installed after construction of the manhole, its weight has been conservatively neglected (i.e., assumed to be zero) for purposes of the analyses herein. It is additionally noted that for purposes of flotation calculations, the manhole weight has been conservatively neglected (i.e., assumed to be zero). The weight of the anti-flotation slab can be computed using Equation 16 [Chevron Phillips, 2004].

$$W_{AF} = (L_{AF} * w_{AF} * \frac{t_{AF}}{12} * (\gamma_C - \gamma_W)) \tag{16}$$

where:

W_{AF} = Weight of anti-flotation slab (lb);
L_{AF} = Length of anti-flotation slab (ft);
w_{AF} = Width of anti-flotation slab (ft);
t_{AF} = Thickness of anti-flotation slab (in);
γ_C = Unit weight of concrete (pcf); and
γ_W = Unit weight of water (pcf).

Equation 17 is used to calculate the weight of the soil above the anti-flotation slab [Chevron Phillips, 2004]. It is noted that for the soil weight calculation, there will not be soil directly above the riser; therefore, the plan area of the riser has been subtracted from the total area.

$$W_{AFS} = (L_{AF} * w_{AF} - A_{OVER}) * (\gamma * (H_{AF} - H_W) + H_W * (\gamma - \gamma_W)) \qquad (17)$$

where:

W_{AFS} = Weight of soil above slab (lb);
L_{AF} = Length of slab (ft);
w_{AF} = Width of slab (ft);
A_{OVER} = Plan area of riser directly on top of slab (ft^2);
γ = Soil unit weight (pcf);
γ_W = Unit weight of water (pcf);
H_{AF} = Height of soil above slab (ft); and
H_W = Height of water above slab (ft).

The upward forces lifting the manhole off the ground include the buoyant force acting on the manhole, as shown in Equation 18 [Chevron Phillips, 2004].

$$F_{UP} = \pi * \frac{D_O^2}{576} * \gamma_W * H_W \qquad (18)$$

where:

F_{UP} = Uplift force (lb);
D_O = Outside diameter of riser (in);
γ_W = Unit weight of water; and
H_W = Height of water (ft).

The overall factor of safety against flotation can be calculated using Equation 19 [Chevron Phillips, 2004]. It is noted that for purposes of this calculation, FS values have been applied to the downward (i.e., resisting) forces individually, therefore, an overall safety factor of 1.0 is be considered acceptable [Chevron Phillips, 2004].

$$FS = \frac{F_{DOWN}}{F_{UP}} \tag{19}$$

where:

FS = Factor of safety against flotation;
F_{DOWN}= Total downward force (lb); and
F_{UP} = Total upward force (lb).

It is noted that the ability of the manhole cover to carry personnel loads and equipment without excessive deformations (i.e., flexural strength) is an important safety consideration. It is recommended that certification be obtained from the manhole riser manufacturer for a minimum allowable live load of about 750 lbs for the manhole cover. Clear signs should be posted at the cover to indicate allowable live loads.

Sump Volume and Pump Sizing:

The storage volume of the sump is calculated as the volume between the pump off level and the top of the low permeability soil liner outside the sump area. This volume is a combination of open area inside the riser pipes and pore volume of areas outside the risers that are filled with gravel.

Pump on time is calculated by dividing the storage volume by the selected pumping rate minus the design inflow rate. Pump off time is calculated by dividing the storage volume by the inflow rate to the sump.

INPUT PROPERTIES

The calculated liquid inflow rate for the entire SCA was 0.4 gal/min, as presented in Appendix I, "Evaluation of Hydraulic Performance for SCA Design". It is noted that this inflow rate has been computed for post-closure conditions (i.e., after placement of the final cover system). However, it is noted that placement of the geomembrane in the final cover system may not occur immediately after construction. Therefore, a target inflow rate of 15 gal/min has been selected to represent conditions following the fourth year of construction and during closure. This target inflow rate is based on calculations presented in the Addendum to Appendix I, "Evaluation of Hydraulic Performance for SCA Design" and corresponds to a 21 inch interim soil cover layer. This target inflow of 15 gal/min is much greater than the calculated target inflow of 0.4 gallons per minute under post-closure conditions. It is further noted that although each sump will have a backup riser, they were not considered in the flow calculations; however, the backup risers and additional temporary pumps can be used during closure, if necessary. Additional temporary pumps can also be used to dewater the risers anytime before the final cover geomembrane is installed. Therefore, the use of 15 gal/min rather than 0.4 gal/min to size the sump, risers, and pumps is considered to be conservative.

As presented in Figure 1, the liner grading plan indicates that the total water infiltration will be split between the western sump area and the eastern sump area. Based on the relative areas, it appears that approximately 67% of the infiltration will drain to the western sump and approximately 33% to the eastern sump area. Therefore, the assumed target liquid inflow rate for the individual sump areas is 10 gal/min. The perforation sizing and pipe structural analyses performed in this package assume that both the western and eastern sump areas will be designed identically to be able to handle the target inflow of 10 gal/min. The volume and pump on and off time calculations use a target liquid inflow rate of 10 gal/min for the western sump area and 5 gal/min for the eastern sump area. It is noted that for the post-closure condition, the total inflow rate of 0.4 gal/min is expected to follow the same 67%/33% split. Therefore, the volume and pump off and on time calculations use a target liquid inflow rate of 0.27 gal/min for the western sump area and 0.13 gal/min for the eastern sump area.

It is further assumed that each sump area will consist of two separate riser pipes, a main and a backup, with a pump in the main riser and a 10 ft offset between each riser pipe. The sump is designed with a depth of 5 ft. As presented in Figure 2, the pump was assumed to require a 6 inch concrete pad (for anti-flotation purposes), a 3 inch HDPE riser bottom and a 6 inch working space between the bottom of the riser and the pump bottom. The pump was assumed to have an automatic off elevation of 6 inches above the bottom of the pump (for post-closure conditions) or 20 inches (for conditions during closure). This was done to facilitate the use of pumps with different capacities and minimum water level requirements under different flow conditions to minimize the water levels within the sumps. The perforated length is designated as L (measured in ft above the automatic off elevation). During closure, L is designated as the difference between the automatic off elevation and the top of the sump (i.e., 2 ft, 1 in). Under post-closure conditions, a lower capacity piston pump can be used, which allows the use of an automatic on elevation (L) of 6 in (0.5 ft), to minimize the water level in the sump during the post-closure period. The walls of the excavated sump area were assumed to be sloped at a 2.5 horizontal:1

vertical (H:V) slope. Each riser pipe was assumed to be approximately 5 ft in diameter. In addition, a 17.5 ft offset between the edge of the riser pipe and the start of the sump side slopes was provided. The area of the pumps is small relative to the total sump area and has been neglected with respect to the sump volume calculation.

It is assumed that the riser pipes will be SDR 26, with an inside diameter of 57.85 inches and a nominal outside diameter of 63 inches, as shown in Table 1. The riser pipes are assumed to be HDPE with a long-term modulus of elasticity at 73°F of 28,200 psi, as shown in Table 2 [Chevron Phillips, 2004]. A reduction factor of 0.76 has been conservatively applied to account for stress concentrations [August et al., 1997]. The Poisson's ratio of HDPE is assumed to be 0.45 for long-term and 0.35 for short-term [Chevron Phillips, 2004]. It is assumed that there will not be any riser stubouts. The manhole weight is assumed to be approximately 8000 lb for purposes of downdrag calculations, based on the estimated weight per lineal foot shown in Table 1 and a 40 ft riser. It is noted that for purposes of anti-flotation calculations, the manhole weight is neglected, as discussed previously. A square anti-flotation slab with length and width of 7.25 ft and a thickness of 6.0 inches will be constructed from concrete (γ_C=150 pcf). It is assumed that the riser will be bolted to the anti-flotation slab.

The design live load for purposes of downdrag calculations is considered to be 1000 lb. It is noted that the weight of two people (assumed to be 250 lb per person), the pump (assumed 100 lb), and other cap equipment such as bolts and flanges (assumed 100 lb) is less than 1000 lb, therefore this assumption is considered reasonable. However, it is noted that the allowable live load on the cap may be less than 1000 lb due to flexural stability of the cap itself. Therefore, the allowable live load should be selected based on manufacturer recommendations.

It is assumed that the riser pipes are surrounded by a layer of gravel throughout their entire height. The maximum riser height is considered to be 40 ft. It is noted that after the dredged material in the geo-tube undergoes settlement, the total thickness of geo-tubes is expected to be less than 30 ft, therefore the use of a 40 ft riser is considered to be conservative. The water is considered to rise to a maximum of 15.5 ft above the base. This corresponds to an elevation of 434 ft, the same height as the lowest berm elevation near the sumps. It is noted that this condition is not expected to occur, however this represents the worst-case scenario of the SCA flooding with water during operations and is therefore expected to be conservative.

Based on discussions with Parsons, two types of gravel, #2 and #3A, are available from nearby quarry sites for use in the SCA drainage layer, including the sump areas. The sieve analysis test results for these two types of gravel are presented in Table 3, and the associated grain size distribution curves are in Figure 3. Gravel type #3A has a larger D_{85} value (D_{85}=1.4 in) than type #2 (D_{85}=0.9 in), therefore it has been assumed that #3A will be selected for the drainage layer to provide more drainage capacity. The unit weight and drained friction angle of the gravel are considered to be 120 pcf and 38 degrees, respectively, following recommendations from Appendix G, “Slope Stability Analyses for SCA Design”. A typical value of 0.4 is chosen for the porosity of the gravel. The elastic modulus for coarse grained soils is assumed to be 250 tsf, as shown in Table 4 [USACE, 1990].

CALCULATIONS

A sample calculation spreadsheet for the pipe perforation sizing is included in Attachment 1, following the methodology described above. The calculation worksheets for the pipe structural stability and sump volume are also provided in Attachment 1.

RESULTS OF ANALYSIS

The results of the pipe perforation sizing are summarized in Table 5. Multiple combinations of perforation angle and row offset resulted in acceptable calculated FS values. Choosing a pipe perforation length of 2.08 ft (based on a total excavation depth of 5 ft), pipe perforation angle of 45 degrees and row offset of 3 inches results in a calculated inflow capacity of 10.1 gal/min. A conceptual diagram of the proposed design is presented in Figure 4.

Based on the pipe perforation dimensions chosen, the structural stability appears to be acceptable, as summarized in Table 6. Specifically, the calculated ring compressive strain, combined ring strain, and ring buckling satisfy the target criteria of less than 3.5%, less than 5.0%, and less than 50%, respectively. In addition, the calculated longitudinal strain is less than the critical longitudinal strain (i.e., 5.2%) and less than the set limit of 3.5%. The calculated FS value against flotation with the anti-flotation slab satisfies the chosen target FS.

The results of the sump volume calculations are summarized in Table 7. Based on the results presented previously, a sump depth of 5 ft was selected, with the two risers separated by 10 ft. This configuration resulted in a total storage volume of 28,000 gallons for each of the sump areas and a filling time of approximately 47 hours for the western sump area and 94 hours for the eastern sump area under conditions during closure (i.e., inflow of 10 gal/min to the western sump, 5 gal/min to the eastern sump, and a pump on level of 2.08 ft above the automatic off elevation of the pump). For a selected pumping rate of 30 gal/min, the western sump will have a pump on time of approximately 23.5 hours under the design condition (i.e., inflow of 10 gal/min). For a selected pumping rate of 20 gal/min, the eastern sump will have a pump on time of approximately 31 hours under the design condition (i.e., 5 gal/min).

For the post-closure conditions (i.e., inflow of 0.27 gal/min to the western sump, 0.13 gal/min to the eastern sump, and a pump on level of 6 in above the automatic off elevation of the pump), the configuration resulted in a fill time of approximately 302 hours for the western sump and 628 hours for the eastern sump. For a selected pumping rate of 2 gal/min, the western sump will have a pump on time of approximately 47 hours under post-closure conditions (i.e., inflow of 0.27 gal/min). For a selected pumping rate of 2 gal/min, the eastern sump will have a pump on time of approximately 44 hours under the design condition (i.e., 0.13 gal/min).

SUMMARY AND CONCLUSIONS

This package presents calculations to support the design of the sump areas of the proposed SCA within WB-13. Three types of calculations were performed: (i) evaluation of the hydraulic requirements for pipe perforations to handle the required inflow during post-closure conditions; (ii) evaluation of the requirements for structural stability of the riser pipe, including ring compressive strain, ring bending strain, ring buckling, longitudinal (axial) strain and buckling, and flotation; and (iii) calculation of liquid storage volume and filling time for the sump area during post-closure conditions.

Based on the calculations, it is recommended that a SDR 26 riser pipe with an inside diameter of 57.85 inches and a nominal outside diameter of 63 inches, with a perforation angle of 45 degrees, perforation row offset of 3 inches, and a total SOLW excavation depth of 5 ft be selected. This resulted in a calculated inflow capacity of 10.1 gal/min, exceeding the target design inflow of 10 gal/min for design conditions and 0.27 gal/min for post-closure conditions. It is noted that the calculated strains did not exceed the target strains. In addition, the calculated FS of the riser pipe satisfied the target FS with regards to flotation with the inclusion of a 7.25 ft x 7.25 ft x 6.0 inches square concrete anti-flotation slab beneath the riser.

For the western sump area, the calculated sump storage volume of 28,000 gallons resulted in a pump off time of 47 hours and pump on time of 23.5 hours using the design inflow rate of 10 gal/min and a pumping rate of 30 gal/min. Additionally, the calculated sump storage volume of 4,900 gallons resulted in a pump off time of 302 hours (approximately 13 days) and pump on time of 47 hours using the design post-closure inflow rate of 0.27 gal/min and a pumping rate of 2 gal/min.

The eastern sump area with a calculated sump storage volume of 28,000 gallons, a design inflow rate of 5 gal/min, and a pumping rate of 20 gal/min, resulted in a calculated pump off time of 94 hours and pump on time of 31 hours. Additionally, the calculated sump storage volume of 4,900 gallons resulted in a pump off time of 628 hours (approximately 26 days) and pump on time of 44 hours using the design post-closure inflow rate of 0.13 gal/min and a pumping rate of 2 gal/min.

Different pump capacities and on/off times may be used as appropriate based on actual field conditions, as long as performance requirements are met. It is further noted that a smaller SDR ratio is expected to reduce the strains and increase the calculated FS values and therefore pipes with a smaller SDR ratio than SDR of 26 used in this calculation may be used as appropriate.

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Written by:	Joseph Sura	Date:	6/16/2010	Reviewed by:	Fan Zhu/R. Kulasingam	Date:	6/18/2010
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Tables

Written by: **Joseph Sura** Date: **6/16/2010** Reviewed by: **Fan Zhu/R. Kulasingam** Date: **6/18/2010**

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Table 1: Typical Pipe Properties [IscoIndustries, 2009]

PE 3608/3408 IPS HDPE PIPE SIZES

Pressure Rating		DR 17 (100psi)			DR 19 (89psi)			DR 21 (80psi)			DR 26 (65psi)			DR 32.5 (50psi)		
Nominal Size	Actual O.D.	Min. wall	Average I.D.	Weight lb/lf	Min. wall	Average I.D.	Weight lb/lf	Min. wall	Average I.D.	Weight lb/lf	Min. wall	Average I.D.	Weight lb/lf	Min. wall	Average I.D.	Weight lb/lf
3/4"	1.050"	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
1"	1.315"	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
1 1/4"	1.660"	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
1 1/2"	1.900"	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
2"	2.375"	0.140"	2.079"	0.429	---	---	---	---	---	---	---	---	---	---	---	---
3"	3.500"	0.206"	3.064"	0.932	---	---	---	---	---	---	---	---	---	---	---	---
4"	4.500"	0.265"	3.939"	1.540	0.237"	3.998"	1.387	0.214"	4.046"	1.262	0.173"	4.133"	1.030	0.138"	4.206"	0.831
5"	5.375"	0.316"	4.705"	2.197	0.283"	4.775"	1.980	0.256"	4.832"	1.801	0.207"	4.937"	1.470	0.165"	5.024"	1.186
5"	5.563"	0.327"	4.869"	2.353	0.293"	4.942"	2.120	0.265"	5.001"	1.929	0.214"	5.109"	1.574	0.171"	5.200"	1.270
6"	6.625"	0.390"	5.799"	3.338	0.349"	5.886"	3.007	0.315"	5.956"	2.736	0.255"	6.085"	2.233	0.204"	6.193"	1.801
7"	7.125"	0.419"	6.236"	3.860	0.375"	6.330"	3.478	0.339"	6.406"	3.165	0.274"	6.544"	2.582	0.219"	6.660"	2.083
8"	8.625"	0.507"	7.549"	5.657	0.454"	7.663"	5.097	0.411"	7.754"	4.637	0.332"	7.922"	3.784	0.265"	8.062"	3.053
10"	10.750"	0.632"	9.409"	8.788	0.566"	9.551"	7.918	0.512"	9.665"	7.204	0.413"	9.873"	5.878	0.331"	10.049"	4.742
12"	12.750"	0.750"	11.160"	12.362	0.671"	11.327"	11.138	0.607"	11.463"	10.134	0.490"	11.710"	8.269	0.392"	11.918"	6.671
14"	14.000"	0.824"	12.254"	14.905	0.737"	12.438"	13.429	0.667"	12.587"	12.218	0.538"	12.858"	9.970	0.431"	13.087"	8.044
16"	16.00"	0.941"	14.005"	19.467	0.842"	14.215"	17.540	0.762"	14.385"	15.959	0.615"	14.695"	13.022	0.492"	14.956"	10.506
18"	18.00"	1.059"	15.755"	24.638	0.947"	15.992"	22.199	0.857"	16.183"	20.198	0.692"	16.532"	16.480	0.554"	16.826"	13.296
20"	20.00"	1.176"	17.506"	30.418	1.053"	17.768"	27.406	0.952"	17.981"	24.936	0.769"	18.369"	20.346	0.615"	18.695"	16.415
22"	22.00"	1.294"	19.256"	36.805	1.158"	19.545"	33.162	1.048"	19.779"	30.172	0.846"	20.206"	24.619	0.677"	20.565"	19.863
24"	24.00"	1.412"	21.007"	43.801	1.263"	21.322"	39.465	1.143"	21.577"	35.907	0.923"	22.043"	29.299	0.738"	22.434"	23.638
26"	26.00"	1.529"	22.758"	51.406	1.368"	23.099"	46.316	1.238"	23.375"	42.141	1.000"	23.880"	34.385	0.800"	24.304"	27.742
28"	28.00"	1.647"	24.508"	59.618	1.474"	24.876"	53.716	1.333"	25.173"	48.874	1.077"	25.717"	39.879	0.862"	26.174"	32.174
30"	30.00"	1.765"	26.259"	68.439	1.579"	26.653"	61.664	1.429"	26.971"	56.105	1.154"	27.554"	45.779	0.923"	28.043"	36.934
32"	32.00"	1.882"	28.009"	77.869	1.684"	28.429"	70.160	1.524"	28.770"	63.835	1.231"	29.391"	52.086	0.985"	29.913"	42.023
34"	34.00"	2.000"	29.760"	87.907	1.789"	30.206"	79.204	1.619"	30.568"	72.064	1.308"	31.228"	58.814	1.046"	31.782"	47.440
36"	36.00"	2.118"	31.511"	98.553	1.895"	31.983"	88.796	1.714"	32.366"	80.791	1.385"	33.065"	65.922	1.108"	33.652"	53.186
42"	42.00"	2.471"	36.762"	134.141	2.211"	37.314"	120.861	2.000"	37.760"	109.966	1.615"	38.575"	89.727	1.292"	39.260"	72.392
48"	48.00"	2.824"	42.014"	175.205	2.526"	42.644"	157.857	2.286"	43.154"	143.629	1.846"	44.086"	117.194	1.477"	44.869"	94.552
54"	54.00"	3.176"	47.266"	222.547	2.842"	47.975"	199.791	2.571"	48.549"	182.298	2.077"	49.597"	148.324	1.662"	50.478"	119.668
63"	62.99"	---	---	---	---	---	---	3.000"	56.631"	247.800	2.423"	57.854"	202.010	1.938"	58.881"	162.980

Selected Pipe

NOTE:
- Items highlighted in Blue indicates standard stocking items that are more readily available.
- Pressures are based on using water at 23°C (73°F).
- Average inside diameter calculated using nominal OD and minimum wall plus 6% for use in estimating fluid flows. Actual ID will vary.
- Service factors should be utilized to compensate for the effect of liquids other than water, and for other temperatures.
- Other piping sizes or DR's may be available upon request.
- Standard Lengths: 40' for 2"-24" / 50' for 26" and larger / Coils available for 3/4"-6"(8" by special order)

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Notes:
1. These are typical commercially available HDPE pipe sizes.
2. The pipe chosen has an outside diameter of 63 in, based on the nominal outside diameter of 5 ft as discussed in the package.

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Table 2: Modulus of Elasticity Values for HDPE [Chevron Phillips, 2004]

***Table 2-2* Modulus of Elasticity for HDPE, psi**

LOAD	TEMPERATURE			
DURATION	**0° F**	**73° F**	**120° F**	**140°F**
Short-term	260,000	110,000	65,000	50,000
1 hour	148,000	69,600	36,900	28,400
10 hours	122,000	57,500	30,500	23,500
1000 hours	92,800	43,700	23,200	17,800
10 years	67,100	31,600	16,800	12,900
50 years	59,900	28,200	15,000	11,500

Notes:
1. The Modulus of Elasticity chosen is for 50 years (long-term) at 73°F.
2. A reduction factor of 0.76 has been applied to the chosen reduction factor [August et. al, 1997]

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Table 3: Sieve Analysis Test Results for Potential Gravel Sources

Sieve Size	#3A's	#2
3"	100.0%	100.0%
2"	100.0%	100.0%
1 1/2"	95.1%	100.0%
1"	14.5%	91.5%
3/4"	5.8%	65.2%
1/2"	3.4%	13.3%
3/8"	2.4%	0.8%
1/4"	1.8%	0.4%
#4		
1/8"		
#8		
#10		
#16		
#20		
#30		
#40		
#50		
#80		
#100		
#200		

Note:
1. These results were provided to Geosyntec by Parsons.

Table 4: Typical Elastic Modulus Values for soils [USACE, 1990]

Typical Elastic Moduli

Soil	E_s, tsf
Clay	
Very soft clay	5 - 50
Soft clay	50 - 200
Medium clay	200 - 500
Stiff clay, silty clay	500 - 1000
Sandy clay	250 - 2000
Clay shale	1000 - 2000
Sand	
Loose sand	100 - 250
Dense sand	$\boxed{250}$ - 1000
Dense sand and gravel	1000 - 2000
Silty sand	250 - 2000

Notes:

1. It is assumed that the riser pipes will be surrounded by gravel, which may or may not be heavily compacted. Therefore, a value of 250 tsf has been assumed to represent the worst expected scenario.

Table 5: Pipe Perforation Summary

Assumed Hole Diameter *d* (in)	Perforated Pipe Length *L* (ft)	Perforation Angle θ (°)	Row Offset *δ* (in)	Computed Inflow Capacity (gal/min)
1.00	3.0	45	2	21.9
1.00	2.08	45	3	10.1
1.00	3.0	45	3	14.6
1.00	4.0	45	3	19.4
1.00	4.0	60	2	21.9
1.00	4.0	60	3	14.6

Notes:

1. This table summarizes acceptable combinations of perforated pipe length L, perforation angle θ, and row offset δ to meet the selected target inflow capacity of 10 gal/min.
2. The selected pipe has a perforated length of 2.08 ft (25 in), perforation angle of 45°, row offset of 3 in and a calculated inflow capacity of 10.1 gal/min.

Table 6: Mechanical Stability Summary

Calculation	Target	Calculated	OK?
Ring Compressive Strain (%)	< 3.5%	0.84%	Yes
Combined Ring Strain (%)	< 5.0%	1.46%	Yes
Ring Buckling (%)	< 50%	15.41%	Yes
Longitudinal Strain (%)	< 3.5% < 5.2%	2.87%	Yes
Flotation with slab (FS)	> 1.0	3.82	Yes

Note:

1. These values are calculated following the methodology of Chevron Phillips [2004], as described in the package.

Table 7A: Sump Volume Summary – Design 15 gal/min flow rate

	Q_{DESIGN} (gal/min)	Pump Off Time[2] (hr)	Pump Rate (gal/min)	Pump Time (hr)
Sump 1	10	47.0	30	23.5
Sump 2	5	93.9	20	31.3

Table 7B: Sump Volume Summary – Post-Closure

	Q_{DESIGN} (gal/min)	Pump Off Time[2] (hr)	Pump Rate (gal/min)	Pump Time (hr)
Sump 1	0.27	302.2	2	47.2
Sump 2	0.13	627.6	2	43.6

Notes:

1. These values are calculated assuming an excavation depth of 5 ft, pipe perforation angle of 45 degrees, and perforation row offset of 3 inches, as described in the package. It is noted that for post-closure conditions, an automatic on level of 6 inches (0.5 ft) was assumed to minimize the water level within the sump.
2. This is the time taken for the liquid level to go from the minimum to maximum level within the sump area.
3. Different pump capacities and on/off times may be used as appropriate based on actual field conditions, as long as performance requirements are met.

Figures

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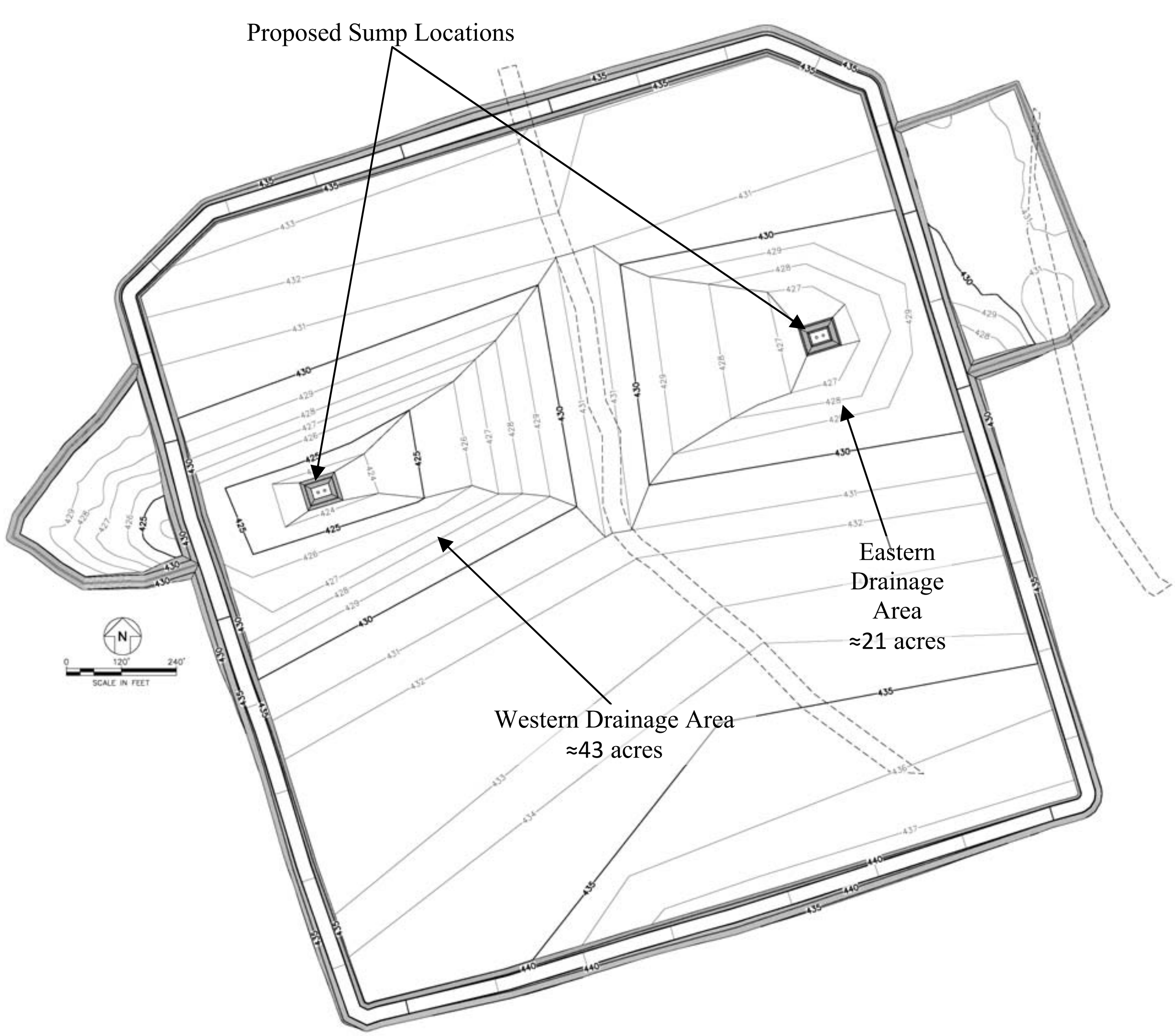

Figure 1: Grading Plan of Top of Low Permeability Soil Layer

Note:

1. Approximately 67% of the SCA area will drain to the western sump area, and approximately 33% of the SCA area will drain to the eastern sump area.
2. This figure does not include settlement.

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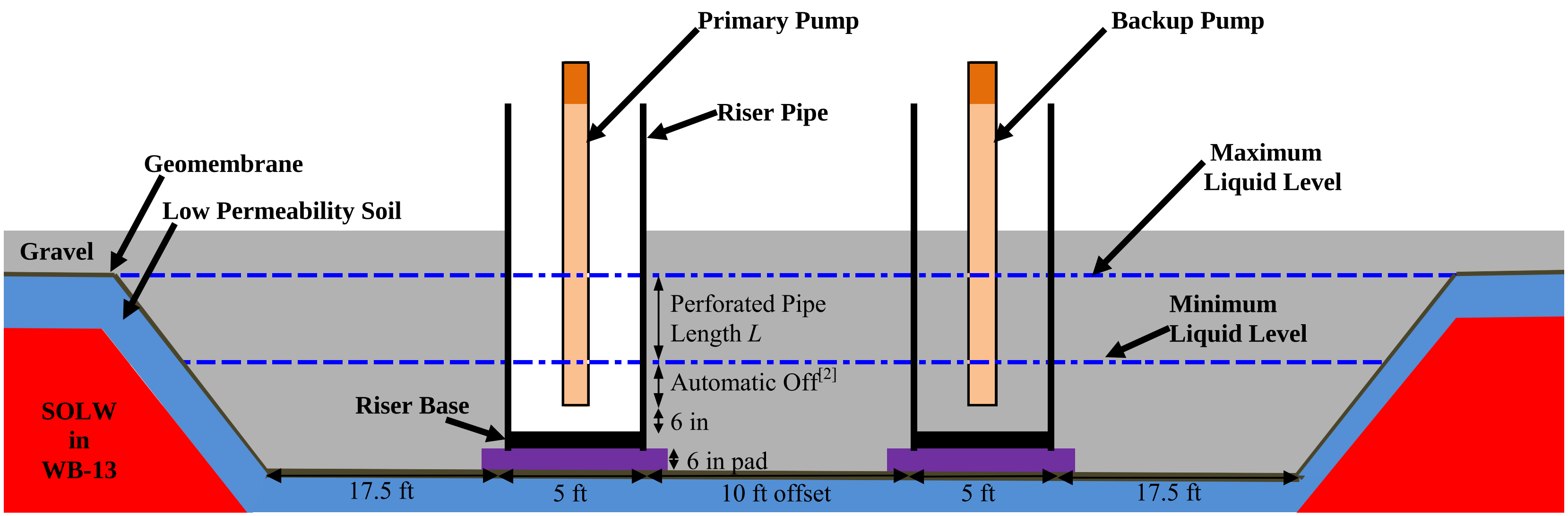

Figure 2: Conceptual View of Sump Area (not to scale)

Notes:

1. The backup riser is conservatively assumed to be filled with gravel, therefore no extra sump volume capacity is considered.
2. The total SOLW excavation depth is 5 ft. An automatic off height of 6 in is sufficient for several commercially available piston pumps for post-closure conditions and an automatic off height of 20 in is sufficient for several commercially available electrical pumps during closure.
3. The side slopes are 2.5 horizontal:1 vertical.
4. The width of the sump bottom is considered to be 5 ft for the riser and 17.5 ft offset from the side slopes, for a total of 40 ft. The length of the sump bottom is 55 ft, as shown on this figure.
5. The low permeability soil liner has a minimum thickness of 1.5 ft near the sump area.

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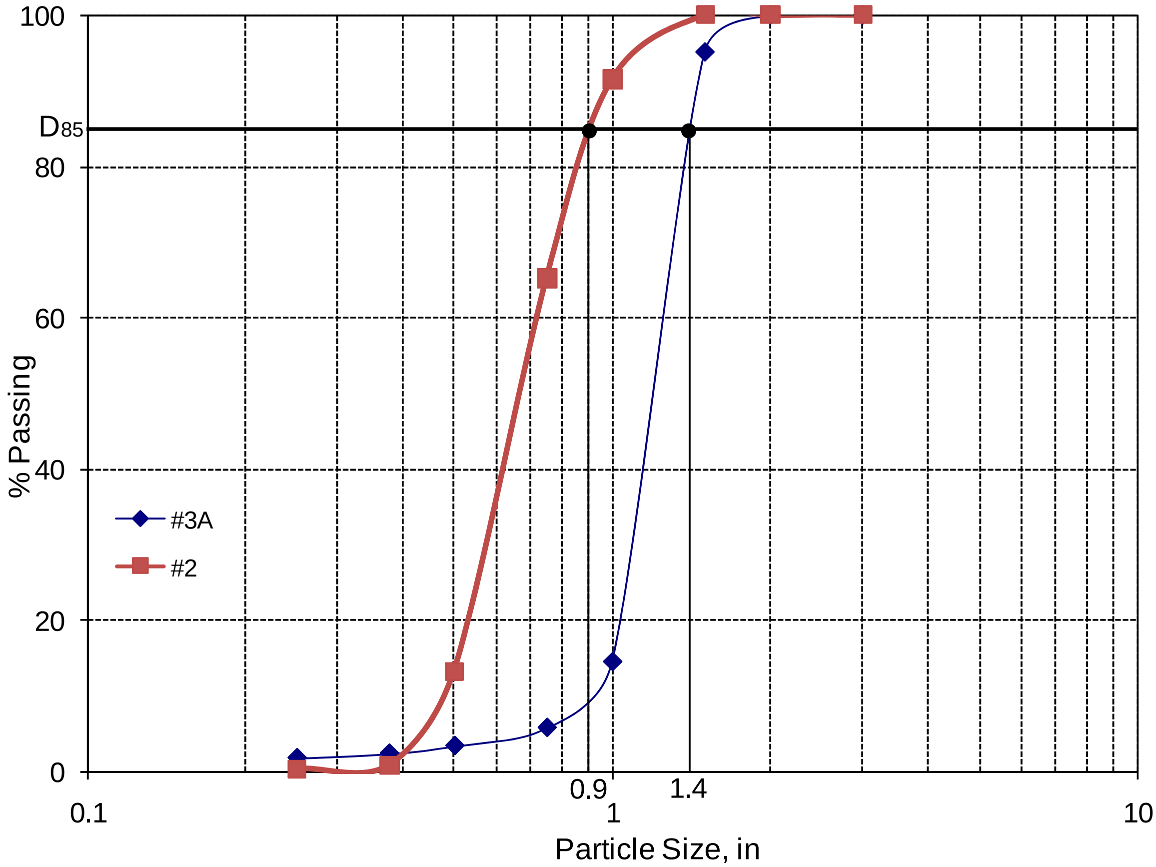

Figure 3: Grain Size Distribution Curves

Note:

1. These grain size distribution curves are based on sieve analysis results for potential gravel sources provided to Geosyntec by Parsons.

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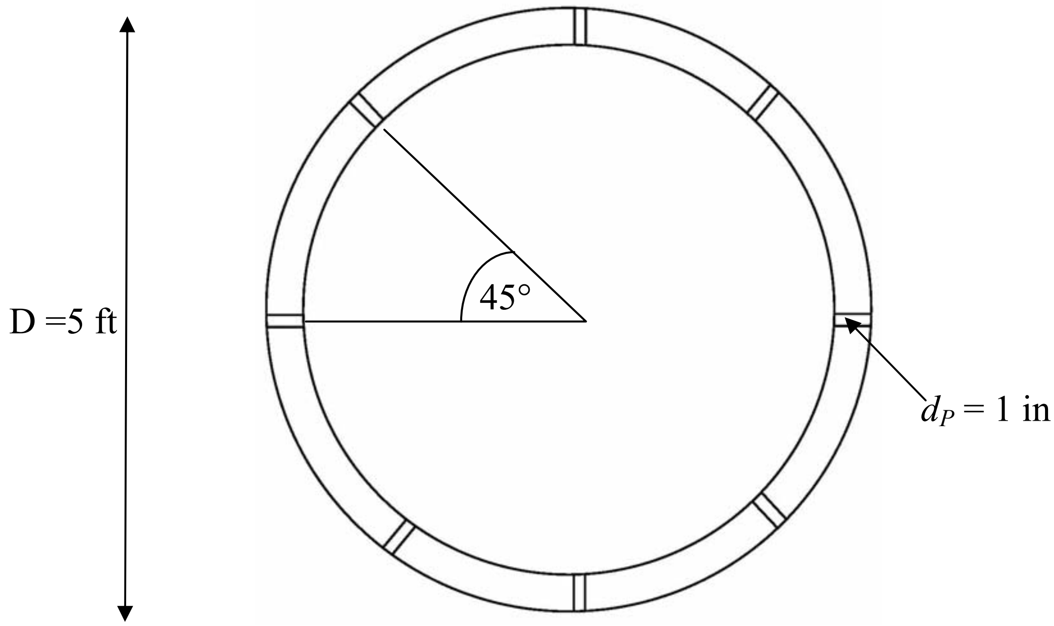

Figure 4a: Conceptual View of Proposed Perforated Pipe Section Design (Plan View)

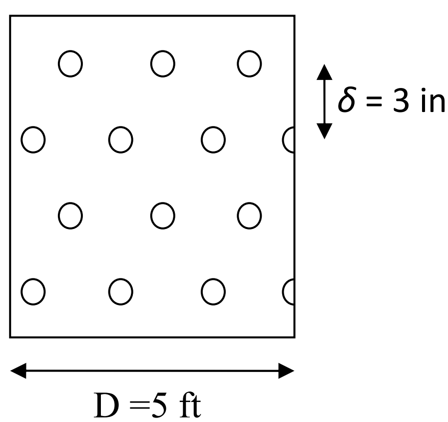

Figure 4b: Conceptual View of Proposed Perforated Riser Pipe Section Design (Side View)

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Attachment 1: Calculation Spreadsheets

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Bernoulli's Equation to calculate pipe perforations

Finding Area of Each Perforation [1]	$A_b=\pi/4*d^2$		
Diameter of Perforation [d] =	1.00	in =	8.33E-02 ft
Area of Perforation [A_b] =	5.45E-03	ft^2	
Bernoulli Equation [2]	$Q_b=C*A_b*v_{ent}$		
Entry Velocity [v_{ent}] =	0.1	ft/s	
Discharge Coefficient [C] =	0.62		
Inflow per Orifice [Q_b] =	3.38E-04	ft^3/s =	2.03E-02 ft^3/min
Pipe Design			
Perforation offset angle [θ] =	45	degrees	
#Perforations in each row [N_{row}] =	8		
Offset between rows [δ] =	3.00	in	
Number of Rows [R] =	4		
Pipe Length [L] =	2.08	ft	
Maximum Flow[3]	$Q_{in} = Q_b * N_{row} * R * L$		
Maximum Inflow Rate [Q_{in}] =	1.35	ft^3/min =	**10.12** gal/min

Note:

1. Qian et al. [2002]

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PROPOSED MANHOLE RISER DESIGN
Onondaga Lake - SCA Design

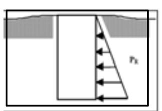
Pipe Type:

Name:		Performance Pipe
Nominal Inside Diameter	ID =	**57.854**
Minimum wall thickness (in)	t =	**2.423**
Reference Outside Diameter (in)	D_O =	**62.99**
Mean Diameter (in)	D_M =	**60.42**
Inside-Diameter Dimension Ratio	IDR =	**23.9**
Manhole Weight (lb)	W =	**8080.4** Obtained from manufacturer IscoPipe
Height of GW above base (ft)	H_w =	**15.5**
Modulus of elasticity (psi)	E =	**21,432** 50 year duration at 73°F [Chevron Phillips, 2004], applied reduction factor=0.76 August et al., 1997]
Diameter of stubouts (in)	D_S =	**0.0**
Number of stubouts	N_S =	**0**
Number of perforations per ft	*n* =	**4**
Diameter of perforations (in)	*d* =	**1.0**

Quick Check: Are requirements met?
Yes

Stresses and Loads

Radially Directed Pressure



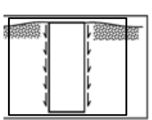
$P_R = (1.21 * K_A * y * H)/(1 - n * d/12)$ [Chevron Phillips, 2004] and [Qian et.al, 2002]

P_R = Radially directed pressure (psf)
K_A = Active earth pressure coefficient = $\tan^2(45-\varphi/2)$
φ = Soil angle of internal friction (°)
γ = Soil unit weight (pcf)
H = Height of fill above base (ft)
n = Number of perforations per ft = 4
d = Diameter of perforations (in) = 1

Layer No.	γ (pcf)	H (ft)	φ (°)	K_A	P_{R1} (psf)	P_{R2} (psf)	T_A (psf)
1 (Gravel above GWT)	**120**	**24.5**	**38.0**	0.238	**0.00**	1269.37	253.87
2 (Gravel below GWT)	**120**	**15.5**	**38.0**	0.238	1269.37	2072.44	668.36
3							
4							
5							
6							

Max. Radially Directed Earth Pressure P_R = **2072.44** psf
Dry soil weight (weighted average) = **120.00** psf

Downdrag Load



$T_A = \mu_F * (P_{R1} + P_{R2})/2$ [Chevron Phillips, 2004]

T_A = Average shear stress (psf)
P_{R1} = Radial pressure at top of riser (psf)
P_{R2} = Radial pressure at base of riser (psf)
μ_F = Friction coefficient between riser and soil = **0.4**
μ_F = 0.4 for HDPE risers with granular or granular-cohesive soils [Chevron Phillips, 2004]

Sample Calculation for Layer 1:
T_{A1} = 0.4 * (0 + 1269.37) / 2
T_{A1} = 253.87 psf

Average shear stress of riser = $(T_{A1}*H_1 + T_{A2}*H_2 + \ldots)/H$
T_A = **414.48** psf

Compressive Stress caused by Downdrag Load [Chevron Phillips, 2004]

$P_D = T_A * \pi * (D_O/12) * H$

P_D = Downdrag load (lbs)
D_O = Outside riser diameter (in) = 62.99
T_A = Average shear stress (psf) = 414.48
H = Height of manhole fill (ft) = 40.0

P_D = 414.48 * π * (62.99/12) * 40
P_D = **273403.3** lb

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Ring Strains

Ring Compressive Thrust

$N_T = P_R * R_M / 144$ [Chevron Phillips, 2004]

N_T = Ring compressive thrust (lb/in)

P_R = Radial pressure (psf)

R_M = Mean radius of riser (in) = (ID + t) /2 = (57.854 + 2.423)/2 = 30.1385

N_T = 2072.44 * 30.1385 / 144

N_T = **433.75** lb/in

Ring Compressive Strain

$\varepsilon_T = N_T / (E * t)$ [Chevron Phillips, 2004]

ε_T = Ring compressive strain (in/in)

N_T = Ring compressive thrust (lb/in)

E = Modulus of elasticity (psi)

t = Minimum wall thickness (in)

ε_T = 433.75 / (21432 * 2.423) Should be less than 3.5% at 73°F [Chevron Phillips, 2004]

ε_T = **0.84%** **<3.5% , OK!**

Ring Bending

$M_E = 0.25 * C_O * D_M * N_T$ [Chevron Phillips, 2004]

M_E = Ring bending moment (in-lb/in)

C_O = 0.02 (correction for 2% deflection), [Chevron Phillips, 2004]

D_M = Mean riser diameter (in)

N_T = Ring compressive thrust (lb/in)

M_E = 0.25 * 0.02 * 60.422 * 433.75

M_E = **131.04** in-lb/in

Ring Bending Strain

$\varepsilon_B = 6 * M_E / (E * t^2)$ [Chevron Phillips, 2004]

ε_B = Ring bending strain (in/in)

M_E = Ring bending moment (in-lb/in)

E = Modulus of elasticity (psi)

t = Minimum wall thickness (in)

ε_B = 6 * 131.04 / (21432 * 2.423^2)

ε_B = **0.62%**

Combined Ring Strain

$\varepsilon_T + \varepsilon_B$ should be less than 5% [Chevron Phillips, 2004]

$\varepsilon_T + \varepsilon_B$ = **1.46%** **<5% , OK!**

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Ring Buckling

Ring Buckling (manhole above GWT)

$N_{CR} = 0.3 * R_H * t * E^{1/3} * E_S^{2/3}$ [Chevron Phillips, 2004]

N_{CR} = Critical ring compressive thrust (lb/in)
R_H = Geometry factor
t = Minimum wall thickness (in)
E = Modulus of elasticity (psi)
E_S = Elastic Modulus of soil (tsf) = **250** = 3472.22 (psi)

Calculate Relative Stiffness RS; if RS<0.005, R_H=1.0 [Chevron Phillips, 2004]

$RS = (0.22*E*t^3) / (E_S * R_M^3)$
RS = (0.22* 21432 * 2.423^3)/ (3472.22 * 30.1385^3)
RS = 0.0007 Geometry Factor=1.0

N_{CR} = 0.3 * 1.0 * 2.423 * 21432^1/3 * 3472.22 ^2/3
N_{CR} = 4629.85

Ring Buckling (manhole below GWT)

$N_{CRW} = 0.82 * ((R * B' * E_S * E * t^3)/D_M)^{0.5}$ [Chevron Phillips, 2004]

N_{CRW} = Critical ring compressive thrust for manhole below GWT (lb/in)
R = Buoyancy reduction = 1 - 0.33 * H_w /H = 0.872
$B' = 1/(1+4*e^{(-0.065H)})$ = 1 / (1+4*exp(-0.065*40))= 0.771
t = Minimum wall thickness (in)
E = Modulus of elasticity (psi)
E_S = Elastic Modulus of soil (psi) = 3472.22

N_{CRW} = 0.82 * (0.771*0.872*3472.22*21432*(2.423)^3) / (60.422)
N_{CRW} = 2814.28 lb/in

N_T should not be greater than 50% of the more critical of N_{CR} and N_{CRW} [Chevron Phillips, 2004]

N_T = 433.75 lb/in
N_{CR} = 4629.85 lb/in
N_{CRW} = 2814.28 lb/in more critical
N_T/N_{CR} = **15.41%** **Less than 50%, OK!**

Longitudinal Compressive Strain

$\varepsilon_A = (P_D + P_L + P_W) / (E * \pi * D_M * t)$ [Chevron Phillips, 2004]

ε_A = Longitudinal compressive strain (in/in)
P_D = Downdrag load (lbs)
P_L = Live load (lbs) = **1000**
P_W = Dead load, including riser weight (lbs)
E = Modulus of elasticity (psi)
D_M = Mean riser diameter (in)
t = Minimum wall thickness (in)

ε_A = (273403.3 + 1000 + 8080.4) / (21432 * Pi * 60.422 * 2.423)
ε_A = 2.87%

$\varepsilon_{CR} = (2 * t) / (D_M * (3*(1-\mu^2))^{0.5})$ [Chevron Phillips, 2004]

ε_{CR} = Critical longitudinal compressive strain (in/in)
D_M = Mean riser diameter (in)
μ = Poisson's ratio for HDPE = 0.45 for long-term [Chevron Phillips, 2004]
t = Minimum wall thickness (in)

ε_{CR} = (2 * 2.423) / (60.422* (3*(1 - 0.45^2))^0.5)
ε_{CR} = 5.19%

ε_A must be < ε_{CR} AND ε_A must be < 3.5% [Chevron Phillips, 2004]

ε_A = 2.87% **OK!**

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Flotation (anti-flotation slab)

$FS = F_{DOWN} / F_{UP}$ [Chevron Phillips, 2004]

FS = Factor of safety

F_{DOWN} = Downward forces (resisting flotation)

F_{UP} = Uplift forces (causing flotation)

$F_{DOWN} = W_{MH} + W_{CC} + (W_{AF})/FS_{AF} + (W_{AFS})/FS_{AFS}$ [Chevron Phillips, 2004]

W_{MH} = Manhole weight (lb)= **0.0**

W_{CC} = Concrete cap weight (lb) = **0**

W_{AF} = Anti-flotation slab weight (lb)

W_{AFS} = Soil weight above slab (lb)

Collar Dimensions and Safety Factors

Width of slab (ft)	w_{AF} =	**7.25**
Length of slab (ft)	L_{AF} =	**7.25**
Thickness of slab (in)	t_{AF} =	**6.0**
Plan area of riser directly on top of slab (ft^2)	A_{OVER} =	**21.6**
Unit weight of concrete (pcf)	γ_C =	**150**
Height of soil above slab(ft)	H_{AF} =	**39.5**
Height of soil above slab below GWT(ft)	H_{WAF} =	15.0
Safety Factor for slab weight	FS_{AF} =	**1.5**
Safety Factor for soil weight	FS_{AFS} =	**1.5**

Weight of Slab

$W_{AF} = L_{AF} * w_{AF} * (t_{AF} / 12) * (\gamma_c - \gamma_w)$ [Chevron Phillips, 2004]

W_{AF} = 7.25 * 7.25 * (6 /12) * (150 - 62.4)

W_{AF} = 2302.2 lb

Weight of Soil

$W_{AFS} = (L_{AF} * w_{AF} - A_{OVER}) *(\gamma*(H_{AF} - H_{WAF}) + H_{WAF} * (\gamma - \gamma_w))$ [Chevron Phillips, 2004]

W_{AFS} = Soil weight above slab (lb)

γ = Soil unit weight (pcf)

γ_w = Unit weight of water = 62.4 pcf

W_{AFS} = (7.25 * 7.25 - 21.6) * (120*(39.5 - 15) + 15*(120 - 62.4))

117781.4 lb

Total Downward Force

$F_{DOWN} = W_{MH} + W_{CC} + (W_{AF})/FS_{AF} + (W_{AFS})/FS_{AFS}$ [Chevron Phillips, 2004]

80055.7 lb

Total Upward Force

$F_{UP} = \pi * (D_o^2/576) * \gamma_w * H_w$ [Chevron Phillips, 2004]

F_{UP} = π * (62.99^2 / 576) * 62.4 * 15.5

F_{UP} = **20930.9** lb

Factor of Safety

Note that Safety Factors have already been applied individually to resisting factors, target FS=1.0

$FS = F_{DOWN} / F_{UP}$

FS = 80055.73 / 20930.9

FS = **3.82** **>1.0, OK!**

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Sump Volume Calculations: Design Flow Rate (15 gal/min)

Design Parameters[1]

	Sump 1	Sump 2
Gravel Porosity n =	0.4	0.4
Perforated Length L (ft) =	2.08	2.08
Target Inflow Rate Q_{IN} (gal/min) =	10	5
Pumping rate Q_{PUMP} (gal/min) =	30	20
Riser Nominal Diameter D_R (ft)=	5	5
Length at Maximum Liquid Level a (ft) =	80	80
Length at Minimum Liquid Level b (ft) =	70	70
Width at Maximum Liquid Level c (ft) =	65	65
Width at Minimum Liquid Level d (ft) =	55	55

Volume Calculations[2]

Vol. of Sump from W_{MIN} to W_{MAX} (gal) =	$V_{PYRAMID}$	
$V_{PYRAMID}$ (gal) =	$(L/6)*(ad+bc+2ac+2bd)/0.134$	
$V_{PYRAMID}$ (gal) =	69980.1	69980.1
Storage Vol. inside Riser V_{RISER} (gal) =	$(\pi/4*D_R^2*L)/.134$	
V_{RISER} (gal)=	304.8	304.8
Total Storage Vol. (gal) =	$(V_{PYRAMID}-V_{RISER})*n+V_{RISER}$	
Total Storage Vol. (gal) =	28174.9	28174.9

Pump On and Pump Off Time Calculations[3]

Pump off time (hr)=	$V_{STORAGE}/Q_{IN}$	
Pump off time (hr)=	47.0	93.9
Pump off time (days)=	2.0	3.9
Pump on time (hr)=	$V_{STORAGE}/(Q_{PUMP}-Q_{IN})$	
Pump on time (hr)=	23.5	31.3

Notes:

1. The lengths and widths at minimum and maximum liquid levels are calculated assuming a 2.5 H:1 V side slope and geometry as shown in Figure 2.
2. The sump volume is calculated based on a truncated rectangular pyramid. The riser pipe volume is calculated based on a cylindrical pipe. The volume occupied by the riser pipe wall thickness is small relative to the overall storage volume and is ignored.
3. Pump off time represents the amount of time necessary for the sump area to fill from the minimum water level to the maximum water level. The pump on time represents the amount of time that the pump runs to remove the liquid.
4. The backup riser is conservatively assumed to be filled with gravel in this calculation.

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Sump Volume Calculations: Post-Closure Condition

Design Parameters[1]

	Sump 1	Sump 2
Gravel Porosity n =	0.4	0.4
Perforated Length L (ft) =	0.5	0.5
Target Inflow Rate Q_{IN} (gal/min) =	0.27	0.13
Pumping rate Q_{PUMP} (gal/min) =	2	2
Riser Nominal Diameter D_R (ft)=	5	5
Length at Maximum Liquid Level a (ft) =	66.25	66.25
Length at Minimum Liquid Level b (ft) =	63.75	63.75
Width at Maximum Liquid Level c (ft) =	51.25	51.25
Width at Minimum Liquid Level d (ft) =	48.75	48.75

Volume Calculations[2]

	Sump 1	Sump 2
Vol. of Sump from W_{MIN} to W_{MAX} (gal) =	$V_{PYRAMID}$	
$V_{PYRAMID}$ (gal) =	$(L/6)*(ad+bc+2ac+2bd)/0.134$	
$V_{PYRAMID}$ (gal) =	12128.8	12128.8
Storage Vol. inside Riser V_{RISER} (gal) =	$(\pi/4*D_R^2*L)/.134$	
V_{RISER} (gal)=	73.3	73.3
Total Storage Vol. (gal) =	$(V_{PYRAMID}-V_{RISER})*n+V_{RISER}$	
Total Storage Vol. (gal) =	4895.5	4895.5

Pump On and Pump Off Time Calculations[3]

	Sump 1	Sump 2
Pump off time (hr)=	$V_{STORAGE}/Q_{IN}$	
Pump off time (hr)=	302.2	627.6
Pump off time (days)=	12.6	26.2
Pump on time (hr)=	$V_{STORAGE}/(Q_{PUMP}-Q_{IN})$	
Pump on time (hr)=	47.2	43.6

Notes:

1. The lengths and widths at minimum and maximum liquid levels are calculated assuming a 2.5 H:1 V side slope and geometry as shown in Figure 2.
2. The sump volume is calculated based on a truncated rectangular pyramid. The riser pipe volume is calculated based on a cylindrical pipe. The volume occupied by the riser pipe wall thickness is small relative to the overall storage volume and is ignored.
3. Pump off time represents the amount of time necessary for the sump area to fill from the minimum water level to the maximum water level. The pump on time represents the amount of time that the pump runs to remove the liquid.
4. The backup riser is conservatively assumed to be filled with gravel in this calculation.