5. Case Studies

5.1 Examples and Case Studies

The farm used in the case studies is located in Baltimore, Haldimand Township, Northumberland County, Ontario. It houses a small beef feedlot, which is partially covered. Figure 5.1 shows an aerial photograph of the farm.

Two alternative designs will be considered: Design #1—integrated, curb-wall, storage on the feedlot; and Design #2—external, dedicated storage basin with pumped flow of runoff to infiltration area.

The numbering of design steps is from the Design Guide found in Section 4.

5.1.1 Design #1—Design of VFS System (Integrated Storage)

The first step in the design of a VFS system for this farm was to confirm that the site conditions are suitable, with reference to Table 2.1 and Figure 2.2. A brief summary of the work required was prepared, to ensure that the project met the farmer's expectations.

For this case study, the following modifications are required to the existing systems:

- resurfacing the feedlot using an epoxy coating or concrete resurfacing to provide a watertight surface with a slope draining toward the collection point
- constructing curb walls around the feedlot, epoxy grouted to the surface, for a watertight seal
- constructing a road culvert for the conveyance piping

In addition, the following components of the system must to be installed:

- collection and conveyance pipes
- VFS with distribution pipe and perimeter berms

Following is the decision process used to complete the system design.

Step	Description	Method	Calculation
Runoff Collection Area			
1.1	Establish extent of area contributing runoff	Identify on a map the drainage patterns around proposed collection area; define all areas contributing surface runoff to the collection area; eliminate all clean water sources, diverting clean flow and other waste flow.	
1.2	Define and measure extent of runoff collection area	Measure the area contributing water to the runoff collection area.	Width = 20 m Length = 50 m Area = 20 m × 50 m = 1,000 m^2
1.3	Select runoff coefficient (see Section 3.1.3)	The surface of the runoff collection area must be non-porous. A concrete surface with a runoff coefficient of 0.95 is assumed.	C = 0.95
1.4	Determine storage/settling volume and peak discharge rate		
1.4.1 (a)	**Option 1**—Select design-storm event (see Section 3.1.4 for calculating the runoff storage/settling maximum basin volume requirements using the conservative method)	Establish closest centre to farmstead from Table 6.1 and determine storage/settling volume required.	Oshawa WPCP For an area of runoff collection A = 1,000 m^2, the maximum storage/settling volume V_{max} = 69.1 m^3
	or		
1.4.1 (b)	**Option 2**—Use IDF Tables and Equation 3.1 to determine the maximum storage/settling volume	From IDF Tables determine 25-year/24-hour storm event for closest centre (Oshawa WPCP) and calculate maximum storage/settling volume.	For an area of runoff collection A = 1,000 m^2, the 25-year/24-hour storm event for Oshawa WPCP = 72.7 mm, the maximum storage/settling volume $V_{max} = (0.95)(72.7 \times 10^{-3}$ m$)(1000$ m$^2) = 69.1$m^3
1.4.2 (a)	**Option 1**—Select minimum storage volume based on peak discharge rate of a 25-year/5-minute storm event over a 15-minute holding time	Establish closest centre to farmstead from Table 6.2 and determine volume of runoff generated.	Oshawa WPCP For an area of runoff collection A = 1,000 m^2, the minimum storage settling volume V_{min} = 31.7 m^3
	Calculate peak discharge rate	Calculate peak discharge by dividing volume obtained in Table 6.2 by holding time of 900 seconds.	$q_p = V_{min}/900 = 31.7/900$ $= 35 \times 10^{-3}$ m^3/s
	or		

Step	Description	Method	Calculation
1.4.2 (b)	**Option 2**—Use IDF Tables and Equations 3.2 and 3.3 to determine peak discharge rate and minimum storage/settling volume	Use Equation 3.2. Rainfall intensity of 25-year/5-minute storm event for Oshawa WPCP is 137.4 mm/hr.	For an area of runoff collection A = 1,000 m^2, the peak discharge rate qp = (0.0027)(0.95)(137.4 mm/hr)(0.1 ha) = 35 × 10^{-3} m^3/s
	Calculate minimum storage/settling volume	Use Equation 3.3 to determine storage/settling volume.	The minimum storage/settling volume $V_{min} = h_{tm}\, q_p$ = (900 sec)(35 × 10^{-3} m^3/s) = 31.5 m^3 for a 15-minute settling time
Storage Settling Basin			
2.1	Select storage type (see Section 3.2)	Determine preferred storage method: integrated using containment wall or dedicated external basin.	Integrated storage/settling basin
2.2	Develop integrated storage/settling basin	Use if runoff collection is suitable for development of a watertight basin and sufficient volume can be generated using a containment wall around all or portion of runoff containment area.	
2.2.1(a)	Determine maximum storage volume required	Volume required is equal to the conservative maximum storage volume calculated in Step 1.4.1 (a) or Step 1.4.1 (b).	V_{max} = 69.1 m^3
	or		
2.2.1(b)	Determine minimum storage volume required	Minimum storage volume required is equal to volume calculated in Step 1.4.2 (a) or Step 1.4.2 (b).	V_{min} = 31.7 m^3
2.2.2	Establish containment wall height		
2.2.2.1	Establish containment wall height assuming consistent slope perpendicular to runoff flow to a low side	Calculate containment wall height using Equation 3.4, assuming a consistent slope of 0.01 m/m and runoff collection area length perpendicular to the runoff flow.	V = 69.1 m^3 L = 50 m S = 0.01 m/m h = SQRT[(2 × 69.1 × 0.01/50)] = 0.17 m

Step	Description	Method	Calculation
2.2.3	Establish final height required to accommodate runoff storage + freeboard + emergency spillway	$h_t = h + 0.3 + 0.15$	$h_t = 0.17 + 0.3 + 0.15 = 0.62$ m (An integrated storage/settling basin can be accommodated for this application)
2.2.4	Calculate discharge rate from the runoff collection area for integrated storage/settling basin with a holding time ranging from 4 to 10 hours (see Section 3.3.1)	Establish closest centre to farmstead from Table 6.1 and determine discharge rate for a retention period of 4 hours.	Oshawa WPCP For an area of runoff collection A = 1,000 m^2 and a retention period of 4 hours, $q_{max} = 4.8 \times 10^{-3}$ m^3/s
		or	
		Using storage volume (V_{max}) from Step 2.2.1(a), determine discharge rate for a retention period of 4 hours.	Discharge rate $q_{max} = 69.1 m^3/4 \times 60 \times 60$ sec $= 4.8 \times 10^{-3}$ m^3/s
Runoff Discharge System Collection Bay			
3.1	Establish collection/discharge bay dimension to accommodate screens and drainpipe area for integrated storage basin	Minimum width of collection bay opening = 1.0 m Length of collection/discharge bay opening Collection/discharge bay height equal to containment wall height	Width = 1.0 m Length = 2.0 m Height = 0.62 m
3.2	Establish screen configuration	Determine size of screens based on the opening measurements of the collection/discharge bay.	Width = 1.0 m Height = 0.62 m
3.3	Select screen material	Material: galvanized metal screen Coarse—vertical spacing of approx. 25 mm Medium—vertical spacing of approx. 10 mm Fine—vertical spacing of approx. 3 mm	Installation of screens to be vertical or at an angle of 60 degrees above the horizontal and sloped away from the flow
3.4	Calculate orifice plate opening based on orifice discharge capacity	Size the orifice opening to accommodate the maximum discharge rate of the integrated storage/settling basin (empty time of 4 hours) for the head equal to final containment wall height (use Table 3.1 or Equation 3.7). Use discharge rate from Step 2.2.4.	$Q = 4.8 \times 10^{-3}$ m^3/s h = 0.62 m Using Table 3.1 the orifice opening diameter D = 0.05 m Using Equation 3.7 orifice area $A = 4.8 \times 10^{-3}$ $m^3/s/[(0.61)(2 \times 9.8 \times 0.62)^{05}]$ $= 2.25 \times 10^{-3}$ m^2 and orifice diameter $D = (4 \times 2.25 \times 10^{-3}/\pi)^{0.5} = 0.054$ m

Step	Description	Method	Calculation
3.5	Calculate diameter of drainpipe	Select drainpipe to accommodate two times the maximum flow rate through the orifice plate opening of the integrated storage/settling basin (empty time of 4 hours) for the head equal to final containment wall height (use Table 3.1 or Equation 3.7). Use discharge rate from Step 2.3.4.	$Q = 4.8 \times 10^{-3}$ m^3/s $\times 2 = 9.6 \times 10^{-3}$ m^3/s h = 0.62 m Using Table 3.1 the orifice opening diameter D = 0.08 m Using Equation 3.7 $A = 9.6 \times 10^{-3}/[(0.61)(2 \times 9.8 \times 0.62)^{0.5}]$ $= 4.5 \times 10^{-3}$m^2 and orifice diameter $D = (4 \times 4.5 \times 10^{-3}/\pi)^{0.5} = 0.076$ m Since the orifice diameter required is 0.08 m (approx), the drainpipe diameter will have to be 0.10 m diameter
3.6	Determine minimum open area in the perforated riser pipe	Use Equation 3.7 to determine the size of the open slot area. Increase the open area by 25%.	$A = 4.8 \times 10^{-3}/[(0.61)(2 \times 9.8 \times 0.62)^{0.5}]$ $= 2.25 \times 10^{-3} \times 1.25 = 2.8 \times 10^{-3}$ m^2 = 28 cm^2 To get 28 cm^2 with 2 × 2 cm slots, it will be necessary to insert 7 slots
3.7	Select sump	Select appropriate size and type of sump. Maximum water depth in sump should be below frost line. Total sump depth = frost line depth + depth to accommodate volume required by pump. Contact sump and pump manufacturer for sump volume recommendations.	
Conveyance Pipe			
4.1	Define target flow rate	Target conveyance pipe flow rate is 10% greater than storage/settling basin discharge rate from Step 2.2.4.	$Q = 4.8 \times 10^{-3} \times 1.10 = 5.3 \times 10^{-3}$ m^3/s
4.2	Establish minimum pipe slope	Use Manning's Equation (Equation 3.9), assuming minimum velocity of 0.6 m/s and Manning's n coefficient = 0.009.	$A = Q/V = 5.3 \times 10^{-3}/0.6 = 8.8 \times 10^{-3}$ m^2 $D = (4A/\pi)^{0.5} = (4 \times 8.8 \times 10^{-3}/\pi)^{0.5} =$ 0.11 m; Select 150 mm pipe R = D/4 = 0.15/4 = 0.0375 m $S = (Vn/R^{2/3})^2 = [0.6 \times 0.009/(0.0375)^{2/3}]^2 = 0.0023$ m/m
4.3	Establish design variables for evaluation of conveyance system	Establish inlet elevation of the conveyance pipe from the sump.	Inlet elevation = 253 m
		Establish conveyance pipe run length from sump to top end of infiltration area.	Conveyance pipe run length = 260 m
		Establish the elevation of the existing grade at top end of the candidate infiltration area.	Elevation at top end of infiltration area = 270 m

Step	Description	Method	Calculation
		Calculate outlet elevation of conveyance pipe.	Conveyance pipe outlet elevation = 253 m – (0.0023 m/m × 260 m) = 252.4 m
		Compare conveyance pipe outlet elevation with elevation of existing grade at top end of infiltration area. See Step 4.4.	Conveyance pipe outlet elevation of 252.4 m vs. elevation of existing grade at top end of infiltration area of 270 m (see Step 4.4)
		Calculate elevation change between inlet elevation of conveyance pipe from sump/drainpipe and elevation of existing grade at top end of infiltration area.	Elevation change = 253 m – 270 m = –17 m, thus a rise in elevation of 17 m
4.4	Determine if gravity or pump system	If the existing grade at the top end of the infiltration area is higher than the conveyance pipe outlet elevation calculated, then gravity flow to the top end of the infiltration area is not possible.	See Step 4.3; a pump system must be used
4.5	Define target flow rate	See Step 4.1.	$Q = 4.8 \times 10^{-3} \times 1.10 = 5.3 \times 10^{-3}\ m^3/s$
4.6	Determine total head losses between pump inlet and distribution pipe discharge	Use Darcy-Weisbach Equation (Equation 3.8) with f = 0.020. For this case study it was assumed localized friction losses were negligible. Add the pressure head of 0.9 m at the distribution pipe.	Head differential (difference in elevation) = 17 m Friction losses = $f(L/D)(V^2/2g)$ $= 0.020\ (260/0.15)\ (0.6)^2/2 \times 9.81) = 0.64$ m Distribution pipe pressure head = 0.9 m Total head losses = 17 + 0.64 + 0.9 = 18.54 m
4.7	Select type and size of pump	Submersible sewage pump (screw-induced flow preferred).	Obtain pump curves from manufacturer and select pump to accommodate target conveyance pipe flow rate and total head losses, i.e., $5.3 \times 10^{-3}\ m^3/s$ @ total head losses of 18.54 m
4.8	Determine pipe size required	Recalculate pipe size based on pump selection and check that velocity does not exceed 1.5 m/s and friction losses are acceptable.	
4.9	Determine requirements for power and controls	Estimate distance from power source and effort required to power and automatically control pump.	Contact consultant or electrical contractor for recommendations.
Distribution Pipe			

Step	Description	Method	Calculation
5.1	Determine distribution pipe length	Determine length of distribution pipe based on infiltration area width (calculated in Step 6.7).	Pipe length = 35 m (see Step 6.7)
5.2	Select diameter of distribution pipe required	Use Manning's Equation (Equation 3.9) with the assumption that the pipe slope is between 0.1% and 0.3% and the pipe will run full with minimum velocity of 0.6 m/s and n = 0.009.	Assume slope S = 0.3% = 0.3 × 0.01 = 0.003 m/m $D = 4[((Vn)/S^{0.5})^{1.5}] = 4[((0.6 \times 0.009)/0.003^{0.5})^{1.5}] = 0.124$ m Select 150 mm pipe
5.3	Determine capacity of orifice opening in distribution pipe	Use Equation 3.7 and assume the following: h = 0.9 m C = 0.61 Orifice diameter = 10 mm	$Q_{orifice} = CA(2gh)^{0.5}$ $= 0.61 \times \pi(0.01)^2/4 \times (2 \times 9.81 \times 0.9)^{0.5}$ $= 0.2 \times 10^{-3}$ m^3/s
5.4	Determine number of orifices to meet flow capacity being delivered	Use a 1.25 target distribution volume. Divide the target discharge rate for the distribution pipe by the orifice discharge rate for each orifice opening as determined in Step 5.3.	$Q = 4.8 \times 10^{-3} \times 1.10 \times 1.25$ $= 6.6 \times 10^{-3}$ m^3/s No. of orifices = $6.6 \times 10^{-3}/0.2 \times 10^{-3}$ = 33 orifices Select 34 orifices Orifice spacing = distribution pipe length (Step 5.1)/no of orifices = 35 m/34 = 1.03 m Space orifices on 1.03 m centres
Infiltration Area			
6.1	Determine minimum infiltration area required based on saturated hydraulic conductivity (Stage 1)	An *in-situ* determination of saturated hydraulic conductivity was performed and a value of 0.3168 m/day was obtained. Table 6.3 can also be used after soil texture has been determined.	$Q = 4.8 \times 10^{-3} \times 1.10 = 5.3 \times 10^{-3}$ m^3/s = 458 m^3/day A = 458 m^3/day/0.3168 m/day = 1,445 m^2
6.2	Determine minimum infiltration area required based on liquid loading (Stage 2)	Use Table 3.3 to obtain the highest normal monthly precipitation for the closest station.	Highest normal monthly precipitation for closest centre (Trenton Airport) = 91.2 mm = 0.0912 m/month
		Calculate highest normal weekly precipitation (highest normal monthly precipitation/4).	Highest normal weekly precipitation = $0.0912/4 = 22.8 \times 10^{-3}$ m/week
		Calculate outside paved yard weekly runoff volume. Paved yard has area of 1,000 m^2.	Paved yard weekly runoff volume = 1,000 m^2 × 22.8×10^{-3} = 22.8 m^3

Step	Description	Method	Calculation
		Calculate limiting precipitation amount per week on VFS. Maximum allowable liquid loading is 0.05 m.	Limiting precipitation amount/week on VFS = 0.05 m – (22.8×10^{-3}) = 27.2×10^{-3}m
		Calculate minimum VFS area required based on paved yard weekly runoff and limiting amount of precipitation/week on VFS area.	Minimum VFS area required = 22.8 m^3/27.2×10^{-3}m = 838 m^2
6.3	Determine minimum infiltration area from Stage 1 (Step 6.1) and Stage 2 (Step 6.2)	See Step 6.1 and Step 6.2. Select larger infiltration area from Step 6.1 and Step 6.2.	Minimum infiltration area from Stage 1 (Step 6.1) is 1,445 m^2
6.4	Determine minimum length of infiltration area	VFS slope is 4%. See Table 3.4	For slope of 4% the VFS length is 41 m
6.5	Determine minimum width of infiltration area	Use Equation 3.10. See Table 3.4.	Flow depth 1.27×10^{-2}m Velocity 4.54×10^{-2} m/s W = 5.3×10^{-3} m^3/s/(1.27×10^{-2}m × 4.54×10^{-2}m/s) = 9.2 m
6.6	Calculate actual infiltration area dimensions	Select largest infiltration area determined from Stage 1 (Step 6.1), hydraulic conductivity measurement or Stage 2 (Step 6.2), liquid loading limit. Use minimum length (Step 6.4) in determining final dimensions of infiltration area. Ensure that final width exceeds minimum width (see Step 6.5).	Largest area results from Step 6.1 of 1,445 m^2, therefore, A = 1,445 m^2 Step 6.1 L = 41 m Step 6.4 W = A/L = 1,445 m^2/41 m = 35.2 m > 9.2 m
6.7	Determine final dimensions of infiltration area	Final infiltration area dimensions.	Length = 41 m Width = 35.2 m Area = 1,445 m^2

5.1.2 Design #2—Design of VFS System (External Storage)

For Design #2, utilizing an existing external storage basin, the following modifications are required for the existing system:

- resurfacing the feedlot to provide a watertight surface with a slope draining toward the collection point
- constructing a road culvert for the conveyance piping

In addition, the following components of the system must be installed:

- external settling/storage basin (existing) and sump with pump
- collection and conveyance pipes
- VFS with distribution pipe and perimeter berms

Following is an outline of the decision process used to complete the system.

5.1.2 Design #2—Design of VFS System (External Storage)

Step	Description	Method	Calculation
Runoff Collection Area			
1.1	Establish extent of area contributing runoff	Identify on a map the drainage patterns around proposed collection area; define all areas contributing surface runoff to the collection area; eliminate all clean water sources, diverting clean flow and other waste flow.	
1.2	Define and measure extent of runoff collection area	Measure the area contributing water to the runoff collection area.	Width = 20 m Length = 50 m Area = 20 m × 50 m = 1,000 m^2
1.3	Select runoff coefficient (see Section 3.1.3)	The surface of the runoff collection area must be non-porous. A concrete surface with a runoff coefficient of 0.95 is assumed.	C = 0.95
1.4	Determine storage/settling volume and peak discharge rate		
1.4.1 (a)	**Option 1**—Select design-storm event (see Section 3.1.4) for calculating the runoff storage/settling basin volume requirements using the conservative method)	Establish closest centre to farmstead from Table 6.1 and determine storage/settling volume required.	Oshawa WPCP For an area of runoff collection A = 1,000 m^2, the storage/settling volume V_{max} = 69.1 m^3
	or		
1.4.1 (b)	**Option 2**—Use IDF Tables and Equation 3.1 to determine the storage/settling volume	From IDF Tables determine 25-year/24-hour storm event for closest centre (Oshawa WPCP) and calculate storage/settling volume.	For an area of runoff collection A = 1,000 m^2, the 25-yr/24-hr storm event for Oshawa WPCP = 72.7 mm, the storage/settling volume V_{max} = $(0.95)(72.7 \times 10^{-3})(1{,}000\ m^2) = 69.1\ m^3$
1.4.2 (a)	**Option 1**—Select minimum storage volume based on peak discharge rate of a 25-year storm with 5-minute duration and 15-minute holding time (see Section 3.1.4 for calculating the storage/settling basin volume requirements using the rational method)	Establish closest centre to farmstead from Table 6.2 and determine volume of runoff generated.	Oshawa WPCP For an area of runoff collection A = 1,000 m^2, the minimum storage settling volume V_{min} = 31.7 m^3
		Calculate the peak discharge by dividing the volume obtained in Table 6.2 by the holding time of 900 seconds.	$q_p = V_{min}/900 = 31.7/900 = 35 \times 10^{-3}\ m^3/s$
	or		

Step	Description	Method	Calculation
1.4.2 (b)	**Option 2**—Use IDF Tables and Equations 3.2 and 3.3 to determine peak discharge rate and minimum storage/settling volume Calculate minimum storage/settling volume	Use Equation 3.2 Rainfall intensity of 25-year/5-minute storm event for Oshawa WPCP is 137.4 mm/hr.	For an area of runoff collection A= 1,000 m^2, the peak discharge q_p = (0.0027)(0.95)(137.4 mm/hr)(0.1 ha) = 35 × 10^{-3} m^3/s
		Use Equation 3.3 to determine minimum storage volume.	The minimum storage/settling volume $V_{min} = h_{tm}q_p$ = (900 sec)(35 × 10^{-3} m^3/s) = 31.5 m^3 for a 15-minute settling time
Storage Settling Basin			
2.1	Select storage type (see Section 3.2)	Determine preferred storage method: integrated using containment wall or dedicated external basin.	Dedicated external basin
2.3	Dedicated external storage/settling basin development		
2.3.1(a)	Determine maximum storage volume required	Volume is equal to the maximum storage volume calculated in Step 1.4.1 (a) or Step 1.4.1 (b).	V_{max} = 69.1 m^3
	or		
2.3.1(b)	Determine minimum storage volume required	Minimum storage volume required is equal to volume calculated in Step 1.4.2(a) or Step 1.4.2(b).	V_{min} = 31.7 m^3
2.3.2	Establish storage volume of existing facility		
2.3.2.1	Establish volume capacity of existing storage facility	Subtract freeboard height and emergency overflow height before calculating maximum storage volume.	Existing storage basin dimensions: depth = 1.5 m, diameter = 7 m $V = [\pi(7)^2/4] \times (1.5 - 0.3 - 0.15) = 40.4$ m^3 Since volume capacity is greater than minimum storage volume (31.7 m^3) required, the existing basin can be used to accommodate runoff water; however, existing storage does not have capacity to hold 25-yr/24-hr storm (69.1 m^3).
2.3.2.2	Establish operating parameters for existing storage facility	Establish elevation of maximum storage volume.	Elev = 1.50 m
		Establish elevation of maximum storage volume minus freeboard and emergency overflow height.	Elev = 1.05 m

Step	Description	Method	Calculation
		Establish elevation of minimum storage volume.	Elev = 0.82 m (assumed)
2.3.4	Calculate discharge rate from external storage/settling basin	Use routing equation (Equation 3.6) since storage volume available is less than storage volume calculated in Step 2.3.1(a) but more than minimum storage volume from 25-year/5-minute storm event over a 15-minute period from Step 2.3.1(b).	$q_p = 35 \times 10^{-3}$ m³/s V = 40.4 m³ = 4.04×10^{-3} ha-m R = 72.7 mm (IDF Table – Oshawa) A = 1,000 m² = 0.1 ha $q_o = 35 \times 10^{-3} [1.25 - (1500 \times 4.04 \times 10^{-3}/72.7 \times 0.1 + 0.06)^{0.5}] = 10.7 \times 10^{-3}$ m³/s
Runoff Discharge System Collection Bay			
3.1	Establish collection point where runoff exits collection area to external basin	Select screening device to prevent coarse materials from entering external basin.	Wood picket fence screening device to have ¾″ spacing between 2″ × 6″ vertical pickets
3.4	Calculate orifice plate opening based on orifice discharge capacity	Size the orifice opening to accommodate the maximum discharge rate of the external storage/settling basin for head equal to depth of liquid on orifice plate (Use Table 3.1 or Equation 3.7). Use discharge rate from Step 2.3.4.	$Q = 10.7 \times 10^{-3}$ m³/s h = 1.05 – 0.82 = 0.23 m Using Table 3.1 the orifice opening diameter D = 0.11 m Using Equation 3.7 Orifice area $A = 10.7 \times 10^{-3}$ m³/s/$[(0.61)(2 \times 9.8 \times 0.23)^{0.5}]$ $= 8.26 \times 10^{-3}$ m² and orifice diameter D $= (4 \times 8.26 \times 10^{-3}/\pi)^{0.5}$ = 0.10 m
3.5	Calculate diameter of drainpipe	Select drainpipe to accommodate two times the discharge rate of the external storage/settling basin (Use Table 3.1 or Equation 3.7). Use discharge rate from Step 2.3.4.	$Q = 10.7 \times 10^{-3}$ m³/s × 2 $= 21.4 \times 10^{-3}$ m³/s h = 1.05 – 0.82 = 0.23 m Using Table 3.1 the orifice opening diameter D = 0.14 m Using Equation 3.7 $A = 21.4 \times 10^{-3}$ m³/s $/[(0.61)(2 \times 9.8 \times 0.23)^{0.5}]$ $= 16.5 \times 10^{-3}$ m² and orifice diameter $D = (4 \times 16.5 \times 10^{-3}/\pi)^{0.5}$ = 0.14 m
3.6	Determine minimum open area in perforated riser pipe	Use Equation 3.7 to determine the size of the open slot area. Increase the open area by 25%.	$A = 10.7 \times 10^{-3}/[(0.61)(2 \times 9.8 \times 0.23)^{0.5}]$ $= 8.26 \times 10^{-3} \times 1.25 = 10.3 \times 10^{-3}$ m² = 103 cm² To get 103 cm² with 2 × 2 cm slots insert 26 slots

Step	Description	Method	Calculation
3.7	Select sump	Select appropriate size and type of sump. Maximum water depth in sump should be below frost line. Total sump depth = frost line depth + depth to accommodate volume required by pump or siphon device. Contact sump and siphon device or pump manufacturer for sump volume recommendations.	
3.8	Select siphon device	Contact a siphon device manufacturer to select the most appropriate mechanism for gravity flow (e.g., Flout™)[1].	
Conveyance Pipe			
4.1	Define target flow rate	Target conveyance pipe flow rate is 10% greater than storage/settling basin discharge rate from Step 2.3.4.	$Q = 10.7 \times 10^{-3} \times 1.10 = 11.8 \times 10^{-3}$ m^3/s
4.2	Establish minimum pipe slope	Use Manning's Equation (Equation 3.9), assuming minimum velocity of 0.6 m/s and Manning coefficient = 0.009.	$A = Q/V = 11.8 \times 10^{-3}/0.6 = 0.020\ m^2$ $D = (4 \times 0.020/\pi)^{0.5} = 0.16$ m; select 200 mm pipe $R = D/4 = 0.2/4 = 0.05$ m $S = [0.6 \times 0.009/(0.05)^{2/3}]^2 = 0.0016$ m/m
4.3	Establish design variables for evaluation of conveyance system	Establish inlet elevation of the conveyance pipe from the sump.	Inlet elevation = 253 m
		Establish conveyance pipe run length from sump to top of infiltration area.	Conveyance pipe run length = 260 m
		Establish the elevation of the existing grade at top end of the candidate infiltration area.	Elevation at top end of infiltration area = 270 m
		Calculate outlet elevation of conveyance pipe.	Conveyance pipe outlet elevation = 253 m – (0.0016 m/m × 260 m) = 252.6 m
		Compare conveyance pipe outlet elevation with elevation of existing grade at top end of infiltration area (see Step 4.4).	Conveyance pipe outlet elevation of 252.6 m vs. elevation of existing grade at top end of infiltration area of 270 m (see Step 4.4)
		Calculate elevation change between inlet elevation of conveyance pipe from sump/drainpipe and elevation of existing grade at top end of infiltration area.	Elevation change = 253 m – 270 m = –17 m, thus a rise in elevation of 17 m for pump system

[1] The use of trademarks does not imply endorsement by OMAFRA, MOE and CH2M HILL Canada Limited.

Step	Description	Method	Calculation
4.4	Determine if gravity or pump system	If the existing grade at the top end of the infiltration area is higher than the conveyance pipe outlet elevation calculated, then gravity flow to the top end of the infiltration area is not possible.	See Step 4.3 (a pump system must be used)
4.5	Define target flow rate	See Step 4.1.	$Q = 10.7 \times 10^{-3} \times 1.10 = 11.8 \times 10^{-3}$ m^3/s
4.6	Determine total head losses between pump inlet and distribution pipe discharge	Use Darcy-Weisbach Equation (Equation 3.8) with f = 0.020. For this case study it was assumed localized friction losses were negligible. Add the pressure head of 0.9 m at the distribution pipe.	Head differential (difference in elevation) = 17 m Friction losses = $f\ (L/D)\ (V^2/2g)$ $= 0.020\ (260/0.15)\ (0.6)^2/2 \times 9.81) = 0.64$ m Distribution pipe pressure head = 0.9 m Total head losses = 17 + 0.64 + 0.9 = 18.54 m
4.7	Select type and size of pump	Select submersible sewage pump (screw-induced flow preferred).	Obtain pump curves from manufacturer and select pump to accommodate target conveyance pipe flow rate and total head losses, i.e., 11.8×10^{-3} m^3/s at total head losses of 18.54 m
4.8	Determine pipe size required	Recalculate pipe size based on pump selection and check that velocity does not exceed 1.5 m/s and friction losses are acceptable.	
4.9	Determine requirements for power and controls	Estimate distance from power source and effort required to power and automatically control pump.	Contact consultant or electrical contractor for recommendations
Distribution Pipe			
5.1	Determine distribution pipe length	Determine length of distribution pipe based on infiltration area width (calculated in Step 6.7).	Pipe length = 78.5 m (Step 6.7)
5.2	Select diameter of distribution pipe required	Use Manning's Equation (Equation 3.9) with the assumption that the pipe slope is between 0.1% and 0.3% and the pipe will run full with minimum velocity of 0.6 m/s and n = 0.009.	Assume slope S = 0.3% = 0.3 × 0.01 = 0.003 m/m $D = 4[((Vn)/S^{0.5})^{1.5}] = 4[((0.6 \times 0.009)/0.003^{0.5})^{1.5}] = 0.124$ m Select 150 mm pipe

Step	Description	Method	Calculation
5.3	Determine capacity of orifice opening in distribution pipe	Use Equation 3.7 and assume the following: h = 0.9 m C = 0.61 Orifice diameter = 20 mm	$Q_{orifice} = 0.61 \times \pi(0.02)^2/4 \times (2 \times 9.8 \times 0.9)^{0.5} = 0.8 \times 10^{-3}$ m³/s
5.4	Determine number of orifices to meet flow capacity being delivered	Use a 1.25 target distribution volume. Divide the target discharge rate for the distribution pipe by the orifice discharge rate for each orifice opening as determined in Step 5.3.	$Q = 11.8 \times 10^{-3} \times 1.25 = 0.015$ m^3/s No. of orifices = $0.015/0.8 \times 10^{-3}$ = 19 orifices Select 20 orifices Orifice spacing = distribution pipe length (Step 5.1)/no. of orifices = 78.5 m/20 = 3.93 m Space orifices 78.5 m/20 = 3.93 m on centres
Infiltration Area			
6.1	Determine minimum infiltration area required based on saturated hydraulic conductivity (Stage 1)	An *in-situ* determination of saturated hydraulic conductivity was performed and a value of 0.3168 m/day was obtained.	$Q = (10.7 \times 10^{-3}) \times 1.10 = 11.8 \times 10^{-3}$ m^3/s = 1,020 m^3/day A = 1,020/0.3168 = 3,220 m^2
6.2	Determine minimum infiltration area required based on liquid loading (Stage 2)	Use Table 3.3 to obtain the highest normal monthly precipitation for the closest station.	Highest normal monthly precipitation for closest centre (Trenton Airport) = 91.2 mm = 0.0912 m/month
		Calculate highest normal weekly precipitation (highest normal monthly precipitation divided by 4).	Highest normal weekly precipitation = 0.0912/4 = 22.8×10^{-3} m/week
		Calculate outside paved yard weekly runoff volume. Paved yard has area of 1,000 m^2.	Paved yard weekly runoff volume = 1,000 $m^2 \times (22.8 \times 10^{-3}) = 22.8$ m^3
		Calculating limiting precipitation amount per week on VFS. Maximum allowable liquid loading is 0.05 m per week.	Limiting precipitation amount/week on VFS = 0.05 m – $(22.8 \times 10^{-3}) = 27.2 \times 10^{-3}$ m
		Calculate minimum VFS area required, based on paved yard weekly runoff and limiting amount of precipitation per week on VFS area.	Minimum VFS area required = 22.8 $m^3/27.2 \times 10^{-3}$ m = 838 m^2
6.3	Determine minimum infiltration area from Stage 1 (Step 6.1) and Stage 2 (Step 6.2)	See Step 6.1 and Step 6.2. Select larger infiltration area from Step 6.1 and Step 6.2.	Minimum infiltration area from Stage 1 (Step 6.1) is 3,220 m^2

Step	Description	Method	Calculation
6.4	Determine minimum length of infiltration area	VFS slope is 4% (see Table 3.4).	For slope of 4% the length is 41 m
6.5	Determine minimum width of infiltration area	Use Equation 3.10 (see Table 3.4).	Flow depth 1.27×10^{-2} m Velocity 4.54×10^{-2} m/s W = 11.8×10^{-3} $m^3/s/(1.27 \times 10^{-2}$ m $\times$ 4.54×10^{-2} m/s) = 20.5 m
6.6	Calculate actual dimensions	Select largest infiltration area determined from Stage 1 (Step 6.1), hydraulic conductivity measurement or Stage 2 (Step 6.2), liquid loading limit. Use minimum length (Step 6.4) in determining final dimensions of infiltration area. Ensure that final width exceeds minimum width (Step 6.5).	Largest infiltration area results from Step 6.1 of 3,220 m^2; therefore, A = 3,220 m^2 (Step 6.1) L = 41 m (Step 6.4) W = A/L = 3,220 m^2/41 m = 78.5 m > 20.5 m
6.7	Determine final dimensions of infiltration area	Final infiltration area dimensions.	Length = 41 m Width = 78.5 m Area = 3,220 m^2

5.2 Alternative Runoff Treatment

Should the farmer decide that a VFS system is not suitable for his or her operation, alternatives for managing the runoff could include:

- extending roof to cover the feedlot and manure storage area completely
- collecting the runoff and storing it for subsequent land application (volume to provide 240 days storage required)

If land application of runoff is limited by the nutrient content of the runoff, then the following alternatives may be considered for treatment:

- constructed wetland
- vegetative infiltration basin (VIB)

These two alternatives rely on soil and plant properties for filtering nutrients and other contaminants from the runoff water. In the VIB system tile drains collect the infiltrated water and deliver the treated sub-surface discharge to a storage tank or to an additional treatment unit (e.g., constructed wetland or VFS system).