APPENDIX H
SWMP SAMPLE CALCULATIONS*

* *Note: The examples are based on the Stormwater Management Practices Planning and Design Manual (1994).*

H.1 Case I: Development is Governed by a Subwatershed Plan

The proposed development is within an area which has a subwatershed plan with the following stormwater management criteria:

- quantity control to reduce the 1 in 5 year post-development peak flow to pre-development levels;
- quality control to detain the runoff volume from a 25 mm rainfall event for 24 hours;
- erosion control equivalent to 100 m^3/ha to be detained for 24 hours; and
- baseflow maintenance of 10 mm/ha based on soils with a percolation rate of 70 mm/h.

The proposed development site is 4.5 ha and will consist of 100 townhouses with a total imperviousness of 63%. The soils in the area have an average percolation rate of 50 mm/h.

Based on the subwatershed plan, the total developed area will require 450 m^3 for erosion control (storage for 24 hours). Using OTTHYMO (Wisner and P'ng, 1983), the runoff volume was modelled for the total site for a 4 hour 25 mm rainfall event, and it was determined that approximately 566 m^3 is required for water quality control. Therefore, stormwater management controls are required to detain 566 m^3 for 24 hours to address water quality and erosion control criteria.

H.1.1 Lot Level Controls

Reduced Lot Grading

Based on the soils and the type of development, the lot grades will be reduced from 2% to 0.5%. Since the land is naturally flat, reduced lot grading will be feasible. The lots will be graded at 2% within 4 m of the building and at 0.5% for the remainder of the lot.

$$DSP = 4.67 + (2 - G)\,f$$

Equation 4.13: Adjusted Pervious Depression Storage

where G = 0.5% (lot grading)
f = 0.75 (longevity factor)

Using Equation 4.13, the pervious depression storage (DSP) was adjusted based on the longevity factor. The adjusted DSP used in the model was 5.8 mm to account for the reduced lot grading.

Roof Leader Discharge to Soakaway Pits

Since residential rooftop drainage is considered "clean water," the roof leaders from the buildings will be discharged to rear yard soakaway pits. The trenches will be located approximately 4 m away from the buildings and approximately 1.5 m above the seasonally high water table. They will be filled with 50 mm diameter clear stone and each trench will be lined with non-woven filter cloth to prevent clogging of the stone. The appropriate bottom area of each trench was calculated using Equation 4.3. Each soakaway pit will serve four townhouse units; therefore, each trench will need to be able to store a maximum volume of 20 mm over the rooftop area of four units (approximately 400 m²). For the 100 units, there will be a total of 25 trenches.

$$A = \frac{1{,}000V}{Pn\Delta t}$$

Equation 4.3: Infiltration Trench Bottom Area

where
- V = 8 m³ (runoff volume to be infiltrated: 20 mm × 400 m² rooftop area for four units)
- P = 50 mm/h (percolation rate of surrounding native soil)
- n = 0.4 (porosity for clear stone)
- Δt = 24 h (retention time)

In order to infiltrate this amount of water, the trench bottom area (A) needs to be at least 16.7 m². Based on the lot configuration and open space areas, soakaway pits which are 2 m wide and 8.5 m long can be constructed. For the storage volume of 8 m³, the pit needs to be 1.2 m deep.

Based on Equation 4.2, the maximum allowable soakaway pit depth is 1.2 m deep.

$$\text{Maximum Allowable Soakaway Pit depth} = P\,T$$

Equation 4.2

where
- P = 50 mm/h (minimum percolation rate)
- T = 24 h (drawdown time)

The required pit depth of 1.2 m (for 8 m³ storage volume) is within the range of maximum allowable soakaway pit depth (Equation 4.2).

Equation 4.17 was used to calculate a rating curve for input to the model based on the storage and outflow for all the soakaway pits:

$$Q = f \times \left(\frac{P}{3{,}600{,}000}\right) \times (2LD + 2WD + LW) \times n$$

Equation 4.17: Soakaway Pit Rating Curve

$$V = LWD \times n \times f$$

where
- f = 0.75 (longevity factor)
- P = 50 mm/h (native soil percolation rate)
- L = 212.5 m (total length of the soakaway pits)
- D = 1.2 m (depth of water in the soakaway pit)
- W = 2 m (width of each soakaway pit)
- n = 0.4 (void space in the soakaway pit clear stone)

Therefore, for a volume of 153 m^3, the discharge will be 0.004 m^3/s. This rating curve was modelled using OTTHYMO and the ROUTE RESERVOIR command for a 4 hour 25 mm storm to assess the contribution of the soakaway pit storage in the determination of end-of-pipe water quality storage requirements. Overflows from the trench storage were added to the runoff from the rest of the site.

H.1.2 Conveyance Controls

Pervious Pipe Systems

The townhouse development will be serviced with traditional curb and gutters. Groundwater contamination is not an issue for this development since a shallow aquifer feeds the stream and the road is local and will not be salted or sanded. Therefore, pervious pipes will be used with regular storm sewers for overflows. The municipality's standards allow pervious storm sewer systems. Grassed boulevards will be used as pre-treatment for the stormwater runoff. A total length of 260 m (130 m on each side of the roads) of perforated pipe with fifty 12.7 mm diameter perforations per metre will be used. The 200 mm diameter perforated pipe will be set at 0.5% slope to promote exfiltration. Clear stone (50 mm) will be used for pipe bedding. The bedding will be surrounded with non-woven filter fabric to prevent the native soil from clogging the voids. The maximum depth will be 1.2 m as calculated previously using Equation 4.1. A typical pervious pipe design is shown in Figure 4.11 (Chapter 4).

Based on the following equation, a rating curve was estimated for the perforated pipe exfiltration flow as a percentage of the pipe flow.

$$Q_{exf} = (15A - 0.06S + 0.33)\, Q_{inf}$$

Equation 4.18: Exfiltration Discharge

where Q_{exf} = exfiltration flow through pipe perforations (see Table H.1)
A = 0.006 m^2/m (area of perforations/m length of pipe)
S = 0.5% (slope of pipe)
Q_{inf} = flow in pipe (see Table H.1)
f = 1.0 (longevity factor)

Table H.1: Head Versus Exfiltration Flow for Perforated Pipe

Depth of water in pipe (m)	Flow in Pipe (m^3/s)	Exfiltration Flow (m^3/s)
0	0	0
0.025	0.001	0.0004
0.05	0.003	0.001
0.075	0.0065	0.003
0.1	0.012	0.005
0.125	0.0165	0.007
0.15	0.021	0.008
0.175	0.022	0.009
0.2	0.023	0.009

The following equation was used to determine the amount of storage volume available within the clear stone pipe bedding.

$$V = LWD \times n \times f$$

where
L = 260 m (length of pervious pipe and stone)
W = 3.0 m (width of stone)
D = 1.2 m (depth of stone)
n = 0.4 (void space for clear stone)
f = 0.75 (longevity factor based on native soil)

Therefore, the actual available volume (V) within the storage media is 281 m^3. The COMPUTE DUHYD command in OTTHYMO was used to divert the peak exfiltration flow to the pipe bedding. The exfiltrated flow was routed through the storage volume using the ROUTE RESERVOIR command.

The outflow from the pipe bedding (soakaway pit rating curve) was calculated based on Equation 4.17.

$$Q = f \times \left(\frac{P}{3{,}600{,}000} \right) \times (2LD + 2WD + LW) \times n$$

Equation 4.17: Soakaway Pit Rating Curve

where
f = 0.75 (longevity factor)
P = 50 mm/h (native soil percolation rate)
L = 260 m (total length of the soakaway pits)
D = 1.2 m (depth of water in the soakaway pit)
W = 3.0 m (width of each soakaway pit)
n = 0.4 (void space in the soakaway pit clear stone)

The outflow from the pipe bedding is 0.006 m^3/s. All overflows were separated from the exfiltrated flows once the pipe bedding storage was exceeded. The overflows were conveyed to the regular storm sewer and used to determine end-of-pipe stormwater management requirements.

Based on the OTTHYMO output, the entire pipe bedding storage is not required. Therefore, as a cost-saving measure, the storage volume was reduced to 140 m^3 (width was reduced to 1.5 m and the corresponding outflow was 0.004 m^3/s). **Note:** An alternative approach would have been to increase the number of perforations and hence, the exfiltration in the perforated pipe.

H.1.3 End-of-Pipe SWMPs

Quality Control

According to the runoff volume reported in the OTTHYMO modelling, the required end-of-pipe storage is 275 m^3. The contributing drainage area and runoff volume are too small for the design of a wet pond or wetland. Therefore, a sand filter is recommended to provide the remaining water quality control. Based on the area available for the sand filter, Equation 4.20 was used to calculate the outflow from the sand filter.

$$Q = f \times \left(\frac{P}{3{,}600{,}000}\right) \times (LW \times n)$$

Equation 4.20: Sand Filter Discharge

where f = 1.0 (longevity factor based on the percolation rate for sand)
P = 210 mm/h (percolation rate for sand)
L = 32 m (length of the filter)
W = 8 m (width of the filter)
n = 0.25 (void space in the sand filter)

Therefore, the outflow from the filter will be 0.004 m^3/s. The storage available within the sand filter is 32 m^3. Storage to a depth of 1.0 m above the sand filter will be used to provide 256 m^3 of active storage. The ROUTE RESERVOIR command was used to model the storage and outflow rating curve.

To provide control of the 1 in 5 year post-development peak flow, a dry pond is recommended which will receive 1 in 5 year flows from the storm sewers. The pond will provide 520 m^3 of storage at approximately 1.0 m depth. The outlet was sized to control the 1 in 5 year post-development peak flow to the pre-development flow.

H.1.4 Baseflow

The reported percolation rate of the soil is actually 50 mm/ha. Therefore, using Equation H.1, the actual infiltration target is 7 mm/ha.

$$I = V\left(\frac{P_{site}}{P_{SWP}}\right)$$

Equation H.1: Site-Specific Infiltration Adjustment

where V = 10 mm/ha (target volume of infiltration from subwatershed plan based on a specific storm event)
P_{site} = 50 mm/h (percolation rate of site-specific soils)
P_{SWP} = 70 mm/h (percolation rate of soils used in subwatershed plan)

Based on the infiltration measures recommended for this site, the total amount of recharge is 14.73 mm/ha which is greater than the required 7 mm/ha to meet the adjusted infiltration target.

H.1.5 Summary of Case I

Based on the stormwater management criteria outlined in the subwatershed plan for this site, quantity control, quality control, erosion control and baseflow maintenance are required. The following stormwater management design will meet each of these criteria.

i) the 1 in 5 year post-development peak flow will be controlled with a dry pond approximately 520 m^3 in volume;

ii) the reduced lot level grading and soakaway pits will reduce the required water quality storage by storing 15 mm (based on the longevity factor) of runoff from the roof area (approximately 150 m^3);

iii) the pervious pipe system will further reduce the water quality storage by providing storage in the pipe bedding (approximately 140 m^3);

iv) the sand filter will provide the remaining water quality storage (approximately 275 m^3);

v) the stormwater management controls will double the required baseflow contribution (approximately 14 mm/ha); and

vi) the measures designed for water quality control will also provide erosion control benefits.

H.2 Case II: No Subwatershed Plan Governs Development

In the absence of watershed/subwatershed planning, Chapter 3 of the SWMP manual was used to provide guidance on the design of stormwater management controls for a 50 ha subdivision. The proposed level of imperviousness for the site is 55%. The entire development will consist of 950 single detached housing units on typical 12 m × 30 m lots. Since there are no flood damage sites downstream of the site, and the site is located at the downstream end of the watershed, the site does not require flood control. The level of protection for aquatic habitat for the receiving water course is normal protection.

H.2.1 Lot Level Controls

Based on the soils, the potential for use of lot level controls is low. The soils have a percolation rate of 20 mm/h, and within this municipality, flat lot grading (< 2%) is not permitted. Also, the potential for contamination of the groundwater is a concern. Therefore, the only lot level control recommended for this site is soakaway pits for rooftop drainage.

Roof Leader Discharge to Soakaway Pits

Since rooftop drainage is considered "clean water," the roof leaders from the buildings will be discharged to rear yard soakaway pits. The trenches will be located approximately 4 m away from the buildings and approximately 1.5 m above the seasonally high water table. They will be filled with 50 mm diameter clear stone. Each trench will be lined with non-woven filter cloth to prevent clogging of the stone.

According to Table 4.11, the water quality storage requirements for the site should be reduced based on the use of soakaway pits. The appropriate bottom area of each trench was calculated using Equation 4.3. Each rooftop is approximately 102 m^2. Equation 4.3 was used to calculate the bottom area required to store the maximum volume of 20 mm over the rooftop area.

$$A = \frac{1{,}000\,V}{P\,n\,\Delta t}$$

Equation 4.3: Infiltration Trench Bottom Area

where V = 2.04 m^3 (runoff volume to be infiltrated for 1 lot)
P = 20 mm/h (percolation rate of surrounding native soil)
n = 0.4 (porosity for clear stone)
Δt = 24 h (retention time)

Therefore, the bottom area of each trench would have to be 10.6 m^2. An area of 5.4 m^2 can be accommodated on each lot (1.2 m wide and 4.5 m long). Based on Equation 4.2, the maximum allowable soakaway pit depth is as follows:

$$\text{Maximum Allowable Soakaway Pit depth} = P\,T$$

Equation 4.2

where P = 20 mm/h (minimum percolation rate)
T = 24 h (drawdown time)

The maximum soakaway pit depth is 0.5 m. Based on the maximum depth and bottom area which can be accommodated, 10 mm of roof drainage can be accommodated in the soakaway pits.

A total of 1,026 m^3 storage will be provided in soakaway pits for the subdivision (950 lots).

H.2.2 Conveyance Controls

Traditional curb and gutters will service this development. Based on the infiltration rates of the soils on this site and the potential for groundwater contamination, pervious pipes are not recommended.

H.2.3 End-of-Pipe SWMPs

A wet pond was chosen as the end-of-pipe stormwater management facility for this subdivision. According to Table 3.2, the design of a wet pond will require 110 m^3/ha of storage which corresponds to the following storage volumes for 50 ha: 3,500 m^3 for permanent pool and 2,000 m^3 for extended detention storage. The wet pond will be located outside of the floodplain and will have a length-to-width ratio of 4:1. The permanent pool will be 2 m deep, and the extended detention storage will be approximately 1.25 m deep.

Storage Requirements

Equation 4.16 determines the reduction in the required end-of-pipe water quality storage volume (active storage) as given by Table 3.2, based on the use of soakaway pits for rooftop drainage.

$$V = \left[(A - RS) \times S\right] + \left[(RS \times S) - (SPV \times f)\right]$$

Equation 4.16: Water Quality Storage Volume Required

where V = volume of water quality storage required (m^3)
A = 50 ha (total area of site)
RS = 9.69 ha (total roof area for all 950 lots)
S = 110 m^3/ha (water quality storage requirement from Table 3.2)
f = 0.5 (longevity factor)

and SPV = $LWD \times n$ (volume of soakaway pit storage)

where L = 4,275 m (length of all soakaway pits)
W = 1.2 m (width of each soakaway pit)
D = 0.5 m (depth of each soakaway pit)
n = 0.4 (void space in the soakaway pit clear stone)

Therefore, the required end-of-pipe active water quality storage volume is reduced from 2,000 m^3 to 1,487 m^3.

Temperature

Since the receiving water course is sensitive to temperature changes, Equation H.2 was used to calculate the temperature change in the stream. Equation H.3 was used to calculate the average urban runoff temperature.

$$\triangle T_{stream} = \left(\frac{QT + q(T_{urb} + \triangle T_{SWMP})}{(Q + q)}\right) - T$$

Equation H.2: Temperature Mass Balance

where Q = 0.233 m^3/s (average monthly summer daily maximum flow rate in the stream)
T = 20°C (average monthly summer temperature in the stream)
T_{urb} = 20.2 (average urban runoff summer temperature)
q = 0.03 m^3/s (average flow from SWMP during a 15 mm storm event)
$\triangle T_{SWMP}$ = 5.1°C (average increase in temperature by SWMP type (Table 4.3))

$$T_{urb} = 15.8 + 0.08(55)$$

Equation H.3: Urban Runoff Temperature

Therefore, the change in stream temperature ($\triangle T_{stream}$) is 0.60°C.

Erosion

There are a variety of methods that designers can use to determine appropriate erosion control requirements including the Simplified Design Approach and the Detailed Design Approach (see Chapter 3 – Environmental Design Criteria and Appendices B, C and D).

A subwatershed study was not performed for this site. The following example outlines a method that has been used by the Toronto and Region Conservation Authority (TRCA) and assumes that required erosion control would be 24 hour detention for a 25 mm rainfall event.

The required volume is 6,875 m^3 which is greater than the 1,487 m^3 required for water quality control (Table 3.2). The required volume for the pond will be decreased by the soakaway pit volume for a total required volume of 6,362 m^3 (6,875 m^3 − 513 m^3 provided by the soakaway pits).

The soils in the area are clayey silts and silty clays. Therefore, according to Figure 4.6, the critical velocity for a 0.01 mm size of particle is approximately 45 cm/s or 0.45 m/s. OTTHYMO was then used to model the erosion control volume to determine if the critical velocity is surpassed in the downstream channel. The uncontrolled post-development flows exceed the critical velocity resulting in an index value of 625.25 based on Equation H.4.

$$E_i = \sum (V_t - V_c)\,\triangle t$$ **Equation H.4: Erosion Index**

where E_i = erosion index
V_t = 1.18 m/s (velocity in the channel at time t = 1.5 h (> V_c))
0.72 m/s (velocity in the channel at time t = 1.667 h (> V_c))
0.49 m/s (velocity in the channel at time t = 1.834 h (> V_c))
V_c = 0.45 m/s (critical velocity above which erosion will occur)
$\triangle t$ = 601.2 s (timestep (0.167 h))

Flows under pre-development and controlled post-development conditions do not exceed the critical velocity. Therefore, the 25 mm control is adequate for this site.

Drawdown Time

The drawdown time in the pond can be estimated using Equation 4.10.

$$t = \frac{2A_p}{CA_o(2g)^{0.5}}\left(h_1^{0.5} - h_2^{0.5}\right)$$ Equation 4.10: Drawdown Time

or if a relationship between A_p and h is known (i.e., $A = C_2 h + C_3$)

$$t = \frac{0.66C_2h^{1.5} + 2C_3h^{0.5}}{2.75A_o}$$ Equation 4.11

where A_p = varies (surface area of the pond)
C = 0.62 (discharge coefficient)
A_o = 0.04 m² (cross-sectional area of the orifice for 226 mm diameter)
g = 9.81 m/s² (gravitational acceleration constant)
h_1 = varies (starting water elevation above the orifice)
h_2 = varies (ending water elevation above the orifice)
C_2 = 4371 (slope coefficient from the area-depth linear regression)
h = 1.09 m (maximum water elevation above the centre-line of orifice)
C_3 = 3220 (intercept from the area-depth linear regression)

The linear regression was based on the area versus depth (y) listed.

for: A_p = 3,136 m² h_1 = 0 m
A_p = 3,969 m² h_1 = 0.14 m (0.25 - 0.113)
A_p = 4,900 m² h_1 = 0.39 m (0.5 - 0.11)
A_p = 5,929 m² h_1 = 0.64 m (0.75 - 0.11)
A_p = 7,056 m² h_1 = 0.89 m (1 - 0.11)
A_p = 8,036 m² h_1 = 1.09 m (1.2 - 0.11)

$$A_p = 4{,}371\ h + 3{,}220$$

$$A_o = \frac{3{,}282 + 6{,}724}{2.75t}$$ (from Equation 4.10)

$$t = \frac{3{,}639}{A_o}$$

Therefore, the drawdown time in the pond is equal to 89,752 s or 24.9 hours.

Forebay Length

The forebay size depends on several calculations.

1. Settling Calculations

The first step is to determine the distance to settle out a certain size of sediment in the forebay. The settling velocities for different sized particles can be estimated from the stormwater particle size distribution monitoring data by the U.S. EPA. Equation 4.5 defines the appropriate forebay length for a given settling velocity.

$$\text{Dist} = \sqrt{\frac{r\,Q_p}{V_s}} \qquad \text{Equation 4.5: Forebay Settling Length}$$

where r = 2:1 (length-to-width ratio of forebay)
Q_p = 0.1 m^3/s (peak flow rate from the pond during design quality storm)
V_s = 0.0003 m/s (settling velocity for 0.15 mm diameter particles)

Therefore, the forebay should be 26 m long to settle particles approximately 0.15 mm diameter in size.

2. Dispersion Length

Equation 4.6 provides a simple guideline for the length of dispersion required to dissipate flows from the inlet pipe. It is recommended that the forebay length is such that a fluid jet will disperse to a velocity ≤ 0.5 metre/second at the forebay berm. The fluid jet should be based on the capacity of the inflow pipe (if the pipe is ≤ 10 year pipe). In this subdivision, the pipe will be designed to convey the 5 year storm flows. A flow splitter will not be implemented.

$$\text{Dist} = \frac{8Q}{d\,V_f} \qquad \text{Equation 4.6: Dispersion Length}$$

where Q = 5.1 m^3/s (inlet flow rate)
d = 2 m (depth of the permanent pool in the forebay)
V_f = 0.5 m/s (desired velocity in the forebay)

Therefore, the forebay length should be 40.8 m for the peak flow during a 5 year storm.

A guideline for the minimum bottom width of this deep zone is given by:

$$\text{Width} = \frac{\text{Dist}}{8} \qquad \text{Equation 4.7: Minimum Forebay Bottom Width}$$

Therefore, the forebay deep zone should be at least 5.1 m wide.

Therefore, the forebay will be 45 m long and 20 m wide (based on an approximate 2:1 length-to-width ratio). The velocity of the flow as it moves through the forebay will be as follows:

$$\text{Velocity} = \frac{Q}{A}$$

where Q = 5.1 m^3/s
A = 22 m^2 (cross-sectional area)

Therefore, the average velocity through the forebay will be 0.23 m/s. This velocity is acceptable since it is less than the 0.45 m/s permissible velocity to prevent erosion, as noted previously.

Given the results of Equations 4.5 and 4.6, the forebay length will be 45 m long and 20 m wide. The permanent pool volume of the forebay will be approximately 900 m^3.

3. Clean-out Frequency

Based on Table 7.3, the annual sediment loading for this site will be approximately 2,300 kg/ha or 1.9 m^3/ha. Therefore, based on the volume of the forebay (900 m^3) and a pond removal efficiency of 70% (Level 2 protection **[Editor's Note: now referred to as normal level of protection]**), the forebay will be required to be cleaned out every 13.5 years. This is acceptable to the municipality since it is greater than its 0 year minimum cleanout frequency.

Forebay Berm

The forebay will be separated from the rest of the pond by an earthen berm. The berm will be submerged slightly below the permanent pool. Low flow pipes will be installed in the berm to convey low flows from the forebay to the pond. The conveyance pipes will be installed in the berm at 0.6 m above the bottom of the forebay. A maintenance pipe will also be installed in the berm to drawdown the forebay for maintenance purposes.

H.2.4 Summary of Case II

According to Table 3.1, a wet pond for this site will require 3,500 m^3 for a permanent pool and 2,000 m^3 for active storage to provide water quality control. For erosion control, the required volume is 6,875 m^3 based on the 25 mm rainfall event. The following SWMPs have been designed to meet these criteria:

i) Soakaway pits will accommodate 10 mm of runoff from the roof area which will reduce the required end-of-pipe active storage requirements by 513 m^3; and

ii) A wet pond will provide the end-of-pipe stormwater management (water quality and erosion) control. The pond will provide 3,500 m^3 of permanent pool storage and 6,362 m^3 of active storage.