HQ Design Office of Storm Water Management

Intranet website: http://projdel/design/storm/treatment.asp

Overview Concepts

Water Quality Volume (PPDG Section 2, pages 2-18 and 19)

For Treatment BMPs designed using volume (e.g. Infiltration Basins and Trenches) the Water Quality Volume (WQV) is calculated by: Water Quality event depth x Tributary Area x C_{ave}

Note: a) Use consistent units (i.e. Water Quality event depth in ft, Area in ft^2).
b) *This is not the Rational Formula!*

The Water Quality event depth may be calculated using the ***Basin Sizer*** program, available at: http://stormwater.water-programs.com

Water Quality Flow (PPDG Section 2, pages 2-19 and 20)

For Treatment BMPs designed using flow (at this time, only the Biofiltration Systems) the Water Quality Flow (WQF) is defined as that flow rate reaching the device under a defined intensity from the tributary area, calculated using the Rational Formula.

Freeboard is that distance between the overflow event water surface elevation and the confinement (usually a berm or top of a vault); minimum freeboard = 0.3m

1. Biofiltration Systems — Biostrips and Bioswales

Design Criteria for BioSwales:

BioSwales	Vegetated open channel used to convey an HDM design event
Q calculated for WQF event	$Q = Q_{25}$ (typical)
0.20 < n < 0.24 (Manning's Coefficient)	n = 0.05
V < 0.3 m/s (1.0 fps)	Velocity $\leq$ 1.2 m/s intermittent
1.2 m (3.95 ft) < b < 4.0 m (13.1 ft) (typical)	1.2 m < b < 4.0 m
Z = 4 (minimum side slope)	Z = 4 (minimum side slope)
Depth of flow (as y or D): y $\leq$ 150 mm (0.5 ft)	NA
Hydraulic Resident Time (HRT) > 5 min	NA
HRT / (Depth * Velocity) $\geq$ C a constant: C = 0.22 for metric; 20 for US Customary units	NA

Flow through a Biofiltration System is calculated using the Rational Formula.

Rational Equation: $Q_{WQF} = 0.28 \times C \times i \times A$ (HDM Topic 819, page 810-15)

Q_{WQF} = Peak flow in m^3 / sec under WQF conditions — 0.28 = Conversion Factor
C = Runoff Coefficient — i = Rainfall Intensity (mm / hr)
A = Tributary Area (km^2)

Note: If also being used to convey the HDM design event: HDM Topic 819 requires a C(f) term for Q_{25} and larger events, giving the equation as: $Q_{HDM} = 0.28 \times C(f) \times C \times i \times A$

Notes: 1) $C(f)_{Q25} = 1.1$; $C(f)_{Q50} = 1.2$; $C(f)_{Q100} = 1.25$ 2) $C(f) \times C \leq 1.0$.
3) $C(f) = 1.0$ for WQF events.

Treatment Criteria using the relationship for HRT, Depth, and Velocity:
HRT / (Depth **x** Velocity) ≥ C (Eq. 1, PPDG Appendix B, page B-3)

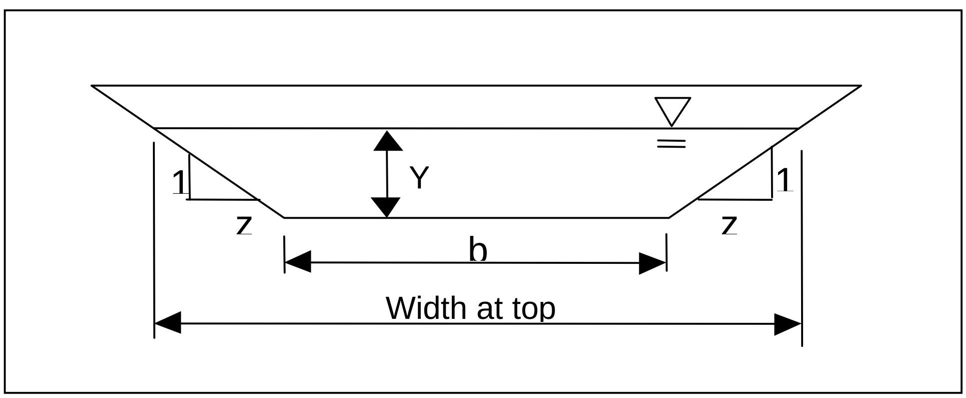
HRT (Hydraulic Residence Time during WQF) ≥ 5 minutes
Depth (depth of flow at WQF) ≤ 150 mm (0.5 ft)
velocity (velocity of flow at WQF) ≤ 0.3 m/s (1 fps)
C = a constant: 0.22 for metric; 20 for US Customary units

Note: This equation **must** use the units as shown.

For Eqn. 1: HRT (minutes) = (Bioswale Length / Velocity) and convert into minutes
Depth and velocity are solved using Manning's Equation, which is presented below.

Manning's Equation: $Q = (1/n) \times A \times R^{2/3} \times S^{1/2}$

Q = flow at defined storm event (e.g., Q_{WQF}, Q_{25}, or Q_{100})
n = Manning's Coefficient (Note: 'n' changes dramatically with depth of water)
Recommend using "n" = 0.05 for Q_{25} or Q_{100}, and use $0.20 \leq n \leq 0.24$ for Q_{WQF}
A = Cross-sectional area of channel in flow
R = Hydraulic Radius = Area / Wetted Perimeter
S = longitudinal slope, length per unit length



Workshop Exercise - Proposed Bioswale
Project Site Information

a) Treatment BMPs required for consideration
b) Tributary area = 0.016 km^2
c) Manning's n = 0.24 for WQF event
d) Runoff coefficient at C = 0.90
e) Proposed Bioswale: L = 100 m, as a trapezoidal channel with an invert width: 1.25 m; side slope ratio at 4H:1V; and 2% slope

Water Quality information

a) WQF intensity *i* = site in RWQCB Region 9: per PPDG Page 2-20 = 0.51 cm/hr (0.20 in/hr)

Assumptions and Requirements for Design of the Treatment BMP

a) Must meet Biofiltration Swales requirements.
b) No restrictions on depth of channel for HDM event.

To Do

1. Determine WQF event intensity *i* for the project for use in the Rational Formula.
   WQF event intensity for Region 9 = _________cm/hr = _________ mm/hr

2. Calculate Q_{WQF} (Rational Formula) = 0.28 C *x I [intensity,* mm/hr] x A [tributary area, km^2]
   0.28 x 0.90 x _____ x ______, then Q_{WQF} = __________________m^3/s

3. Calculate the depth and velocity in the trapezoidal channel *at Q_{WQF}*
Use Manning's formula: $Q = (1/n)\,(A)\,R^{2/3}\,S^{1/2}$ Solve using the table below.

Manning's Equation for WQF - Trapezoidal Channel $Q = (1/n) \times A \times R^{2/3} \times S^{1/2}$				
Assumed Conditions	**b, trapezoidal channel width**	**z (side slope [run:rise]))**	**Manning's n (use 0.20 to 0.24)**	**S (length / unit length)**
	1.25 (4.1 ft)	**4**	**0.24**	**0.02**

Q = WQF	**Y = D, Depth**	**Velocity of flow**	A, area of flow	P, wetted perimeter	R, Hyd. Rad.
(m^3 / sec)	(millimeters)	(m/s)	(meters ^ 2)	(meters)	(meters)
0.007	**60**	**0.08**	0.0894	1.74	0.051
0.008	**65**	**0.09**	0.0982	1.79	0.055
0.010	**70**	**0.09**	0.1071	1.83	0.059
0.011	**75**	**0.09**	0.1163	1.87	0.062
0.012	**80**	**0.10**	0.1256	1.91	0.066
0.013	**85**	**0.10**	0.1352	1.95	0.069
0.015	**90**	**0.10**	0.1449	1.99	0.073
0.016	**95**	**0.11**	0.1549	2.03	0.076
0.018	**100**	**0.11**	0.1650	2.07	0.080
0.020	**105**	**0.11**	0.1754	2.12	0.083
0.021	**110**	**0.11**	0.1859	2.16	0.086

Depth ("Y") of flow in trap channel = ___________________ mm
Velocity of flow in trap channel = ___________________ m/s

4a. Calculate the travel time (Hydraulic Residence Time) through the proposed Bioswale
   HRT = Length / Velocity / (60 sec/min) = 100 / ________ m/s /60 = _____________ min

4b. Calculate the HRT/D*V ratio and compare to BioSwale requirements
   HRT/(D x V) = _________ *Note units: HRT in minutes, D in mm [ft], velocity in m/s [ft/s]*

5. Compare calculated values to Bioswale requirements

Bioswale Requirements	**BioSwale as proposed**
Depth <= 150 mm	_________ OK?
Velocity <= 0.30 m/s	_________ OK?
HRT > 5 minutes	_________ OK?
HRT/(D x V) > 0.22 (metric units) [HRT/(D x V) > 20 (US Customary units)]	_________ OK?

2a. Infiltration Basin — Invert Area

Size to hold WQV. Minimum volume = 123 cubic meters. No restriction on shape of basin; trench usually rectangular.
$A_{est} = (C_1 \times SF \times WQV) / (k_{est} \times t)$ (Eq. 2, PPDG Appendix B, page B-18)

A_{est} = estimated area of invert of basin, m^2 or ft^2
C_1 = conversion factor (100 for cm to m; 12 for inch to ft)
SF = safety factor of 2.0
WQV = water quality volume calculated from the design storm, m^3
k_{est} = estimated infiltration rate, cm/hr or inches/hr
t = drawdown time, 40 to 48 hours

Large Event Release From Infiltration Basins Using Overflow Spillways

Controlled release of events larger than the WQ event is usually provided through the confining berm using a weir placed below the berm crest elevation. Height of the water in the Infiltration Basin needs to computed in order to later calculate available Freeboard below the crest of the conferment. The equation related to flow through Broad Crested Weirs is presented below.

Overflow Spillway using a Broad Crested Weir

$Q = C_{BCW} \times L \times H^{1.5}$ and rearranging depending upon the unknown to be solved: $L = [Q/(C_{scw} \times H^{1.5})]$ or $H = [Q/(C_{scw} \times L)]^{2/3}$ Q = Design Storm (may be Q_{100} or Q_{25} depending upon risk assessment and likelihood)
C_{BCW} = Weir coefficient
L = Length of weir (perpendicular to flow) Note: Minimum 'L' is 1.0 meter
H = difference in elevations between the water at overflow event and the weir elevation.

Broad-crested weir coefficient C_{BCW} values as a function of weir crest breadth and head*

Head	Breadth of weir having vertical sides (m)									
(m)	**0.15**	**0.20**	**0.30**	**0.40**	**0.50**	**0.60**	**0.70**	**0.80**	**0.90**	**1.00**
0.10	1.59	1.56	1.50	1.47	1.45	1.43	1.42	1.41	1.40	1.39
0.15	1.65	1.60	1.51	1.48	1.45	1.44	1.44	1.41	1.45	1.45
0.20	1.73	1.66	1.54	1.49	1.46	1.44	1.44	1.45	1.47	1.48
0.30	1.83	1.77	1.64	1.56	1.50	1.47	1.46	1.46	1.46	1.47
0.40	1.83	1.80	1.74	1.65	1.57	1.52	1.49	1.47	1.46	1.46
0.50	1.83	1.82	1.81	1.74	1.67	1.60	1.55	1.51	1.48	1.48
0.60	1.83	1.83	1.82	1.73	1.65	1.58	1.54	1.46	1.31	1.34
0.70	1.83	1.83	1.83	1.78	1.72	1.65	1.60	1.53	1.44	1.45
0.80	1.83	1.83	1.83	1.82	1.79	1.72	1.66	1.60	1.57	1.55
0.90	1.83	1.83	1.83	1.83	1.81	1.76	1.71	1.66	1.61	1.58
1.00	1.83	1.83	1.83	1.83	1.82	1.81	1.76	1.70	1.64	1.60

Breadth: measured longitudinally with the flow. Length: measured perpendicularly to flow. * For conversion into US Imperial units: at the same Head and Breadth, multiply the C_{BCW} by 1.81

Note: if an assumed value of C_{BCW}, is used to calculate either the H or L, verify that assumed C_{BCW} is applicable for the design inputs at the end of the calculation.

Workshop Exercise - Proposed Infiltration Basin

Assumptions and Requirements for Design of the Treatment BMP

a) BMP Drawdown times – given in To Do section
b) Permeability ("k") estimated as 2.5 cm/hr, Type A HSG:
c) HDM overflow event, at 0.26 m^3/sec

To Do

1. For an Infiltration Basin: Determine invert area

A = (C x SF x WQV) / (k x t) Eqn. 2, PPDG B-18
Where:
A = est. invert area, m^2 (or ft^2) C = conversion factor (*100* for cm to meters)
SF = safety factor of *2.0* WQV = water quality volume = *123* m^3
k = estimated infiltration rate = *2.5 cm/hr* t = drawdown time, *40 hours*

A = (___ x ____ x ____) / (___ x _____) Then, A_f= ______ m^2

2. For the Infiltration Basin[1]: Calculate the height of water, H, above the WQV elevation during the large event overflow.
Use a broad-crested weir of length L = *1 m*, C_{BCW} = *1.55,* HDM overflow event = *0.26* m^3/sec

$Q = (C_{BCW}) \times L \times H^{1.5}$ re-arrange terms, then take each side to the 2/3 power to solve for H:

$H = [Q / (C_{BCW} \times L)]^{2/3}$ [___ / (____ x ____)]$^{2/3}$ = _______ m

3. For the H calculated, determine from the Table, using Breadth = 0.40 m, if the assumed value of C_{BCW} used to calculate the H was sufficiently close, or if a revised calculation is needed.

From Table for H from Step 2: H = _______ has an associated C_{BCW} of_______
Compare to assumed C_{BCW} as 1.55 OK?

[1] Overflow devices from Detention Basins and Wet Basins would be sized using this procedure if a **broad crested weir** were used for overflow release.

2b. Infiltration Trench — Invert Area *(revised equations)**

a) Depth D = (k x t) / (C x SF x porosity)   Use consistent units as noted below.

where D = depth, m or ft
k = known or estimated infiltration rate, cm/hr or inches/hr
C = conversion factor (100 for cm to m; 12 for inch to ft)
SF = safety factor of 2.0
t = drawdown time, 72 hours
porosity = 0.35 for rock

Note: the elevation of the trench invert controlled by highest elevation:
i) elevation calculated using the relationship of permeability, drain time, and porosity in the formula shown above;
ii) separation of the invert from seasonally high groundwater ($\geq$3 m);
iii) maximum distance from ground surface (4 m).

b) Required total volume of the trench, V_T:
V_T = WQV/porosity of filler material.   Use 0.35 as the porosity for rock.
Where V_T = total volume

c) Calculate invert area, A (m^2 or ft^2), using values from above: $A = V_T / D$

d) Calculate surface dimensions Length L and width W using A from above:
A = L x W   Enter with the calculated A and the known dimension of L or W, and solve for unknown dimension.

Note: the Infiltration Trench is usually configured with vertical sides, so the invert dimensions will be the same as the surface dimensions.

** NOTE: These revised equations will replace Eqs. 3 through 5 in PPDG Appendix B, page B-19.*

Workshop Exercise - Proposed Infiltration Trench

Project Site Information

a) WQV: 123 m^3
b) No disqualifying physical siting criteria present
c) Influent to the proposed Infiltration Trench conveyed via a bioswale, and overflow events continue downstream in the bioswale

Water Quality information

a) No RWQCB restrictions on water that can be infiltrated
b) No pollutant plumes below proposed site

Assumptions and Requirements for Design of the Treatment BMP

a) Depth: as required for the invert area – assume that depth to groundwater does not control
b) Permeability ("k") estimated as 5.0 cm/hr, Type A HSG
c) 72 hour drawdown time ("t")
d) Assume a 2:1 L to W ratio for the invert area.

To Do

Determine depth and footprint for the infiltration trench, using the steps below.

1. Calculate maximum depth D (Note: D <= 4.0 m)

$D = (k_{est} \times t) / (C \times SF \times porosity)$ C = conversion for permeability, 100 cm/m

D = (______ x _____) / (_______ x ______ x ______) = ________m by equation

Use depth D = ________m

2. Calculate the required total volume of the trench, V_T:

V_T = WQV/porosity of filler material. Use 0.35 as the porosity for rock.

total volume of the trench, V_T = _______/______ = ______

3. Calculate invert area A

$A = V_T / D$ = _______/______ = ______

4. Calculate Length and Width, with 2W = L

Area $A = L \times W = 2W^2$ $W = (A/2)^{1/2}$ = ______ and L = ________

3. Detention Basins

Size to hold WQV; Minimum volume = 123 cubic meters.

Length/Width (L / W) ratio should be greater than or equal to 2.0 <u>at WQV surface elevation</u>.

Equations to size Water Quality Outlet orifices are shown below.

3a. WQV Release Using Outlets: Orifice Devices on a CMP Riser

a) Single row of orifice(s) (i.e., all orifices at same elevation):

$a = [(2 \times 10^6) \times A \times (H\text{-}H_o)^{0.5}] / [3600 \times C \times T \times (2g)^{0.5}]$ (Eq. 6, PPDG Appendix B, page B-30)

a = area of drainage orifices (square mm); typically use 3 orifices at 60 to 120 degrees separation around invert of riser.
A = <u>Surface area of the water in the basin</u> <u>at mid WQV elevation</u> (m^2)
C = orifice coefficient (commonly 0.4 to 0.6)
T = Basin Drawdown Time (normally 40 to 48 hours)
g = Acceleration due to gravity (9.81 m/sec^2)
H = Elevation of water surface at WQV (m)
H_o = Elevation of basin invert (m) Note: *H-Ho equals the depth in the basin at WQV*

b) Multiple rows of orifices (i.e., several orifices arranged vertically):

$a_t = [2 \times 10^6 \times A \times h_{max}] / [3600 \times C \times T \times (2g \times [h_{max} - h_{centroid}])^{0.5}]$
metric version, Eq. 8, PPDG Appendix B, page B-30

a_t = total area of drainage orifices (mm^2); typically use 3 orifices per row, at 60 to 120 degree separation circumferentially
A = <u>Surface area of the water in the basin</u> <u>at mid WQV elevation</u> (m^2)
C = orifice coefficient (commonly 0.4 to 0.6)
T = Basin drawdown time (normally 40 to 48 hours)
g = Acceleration due to gravity (9.81 m/sec^2)
h_{max} = distance from lowest orifice to WQV elevation (m)
$h_{centroid}$ = distance from lowest orifice to centroid of orifice configuration (m)

3b. Large Event Release From The Top Of A CMP Riser (Sharp Crested Weir)

Often used in a Detention Basin, an opening at the top of a Water Quality Outlet Riser (usually as a CMP [corrugated metal pipe]) could serve as overflow relief; it would be analyzed as a sharp-crested weir using the formula below. There is a check that must be performed to ensure that the flow through the riser does not operate under orifice conditions (which releases less water).

$Q = C_{scw} \times L \times H^{1.5}$ rearranging depending upon the unknown to be solved:
$L = [Q/(C_{scw} \times H^{1.5})]$ or $H = [Q/(C_{scw} \times L)]^{2/3}$ Q = HDM Design Storm (Q_{100} or Q_{25} as directed by Hydraulics)
C_{scw} = Sharp Crested Weir coefficient (see next page for discussion)
L = Length of weir, which, <u>if using a CMP riser will be the perimeter of the riser</u>
H = difference in elevations between the water at overflow event and the weir elevation.

Sharp-crested weir coefficient C_{scw} values
The ratio is used to calculate the sharp crested weir coefficient as given by the equation:

C_{scw} = 1.81 + 0.22 (H/H_c), metric units
= 3.27 + 0.4 (H/Hc), US Customary units

H = height of water above the weir, and Hc = height of water between the weir and basin invert

For values of the H/Hc ratio less than 0.3, a constant coefficient of 1.84 (metric) is often used (3.33 in US Customary units).

NOTE: At some height of water above a riser operating as a sharp-crested weir, the flow regime can transition to orifice flow, therefore two calculations must be made, and the lower flow (larger H) will control. To perform this check, use this simplified orifice formula, using the given inflow rate (formula assumes a circular orifice coefficient of 0.6). Note: D = diameter of the riser at its crest.

$Q_{\text{as orifice flow over the riser}} = K_{or} \times D^2 \times H^{0.5}$ with K_{or} = 2.09 (metric) or 3.78 (US Customary units) where D = diameter of overflow outlet riser, and H = elevation of flow over the riser during orifice flow

References: FHWA Hydraulic Engineering Circular No. 22 URBAN DRAINAGE DESIGN MANUAL pages 8-22ff; Second Edition, August 2001

Workshop Exercise - Detention Basin

<u>To Do</u>

1. Calculate the WQ orifice size, assuming single orifice at the base of a riser, and the other information supplied below for the terms described above.

$a = [(2 \times 10^6) \times A \times (H-Ho)^{0.5}] / [3600 \times C \times T \times (2 \times g)^{0.5}]$

where a = orifice area (mm^2) = TBD A = *<u>655.8 m^2</u>* C = *<u>0.60</u>*
T = <u>*48*</u> hours g = *<u>9.81 m/s^2</u>* H = *<u>101.20 m</u>* H_O = *<u>100.00 m</u>*

$$a = \frac{2\times10^6(655.8)(101.2-100.00)^{0.5}}{3600(0.60)(48)(2\times9.81)^{0.5}}$$ Then, a = __________ mm^2

2a. For the large event overflow from the top of the riser, calculate the height of water, H, above the WQV elevation while the riser functions as a sharp-crested weir.[2]
Use a sharp crested weir L = *1.92 m*, C_{SCW} = *1.84*, and HDM $Q_{overflow}$ = *0.26 m^3/sec*

$Q_{overflow} = (C_{SCW}) \times L \times H^{1.5}$ re-arrange terms, then take each side to the $^2/_3$ power to solve for H:

$H_{\text{weir flow}} = [Q_{overflow} / (C_{SCW} \times L)]^{2/3}$ $= [0.26 / (1.84 \times 1.92)]^{2/3}$ = ___ m

2b. Recheck the flow regime through the riser, assuming flow functions as an orifice:[3]
Diameter = _______ if the perimeter is *1.92 m*, and HDM $Q_{overflow}$ = *0.26 m^3/sec*
Then solve for $H_{\text{orifice flow}} = [Q_{overflow} / (K_{or} \times D^2)]^2$ = [0.26 / (2.09 x ___^{2})]^2 = ___ m

3. Use the **larger** H from 2a. or 2b. when setting top of berm elevation:
H = ________m with flow operating under ____________ conditions.

[2 and 3] Overflow devices from Infiltration Basins and Wet Basins would be sized using this procedure if a sharp crested weir were used for overflow release.

4. If the WQV depth is 1.2 m, what is the depth measured from the top of the berm.

$1.2 + H_{overflow}$ event + freeboard = 1.2 + _______ + 0.3 = ______ m

4. Dry Weather Flow Diversion

Refer to PPDG Appendix B pages B-39 through B-40 for Dry Weather Flow Diversion.

There is no Workshop Exercise for Dry Weather Flow Diversion

5. Gross Solids Removal Devices (PPDG, pages B-25 through B-29)

Refer to PPDG Appendix B pages B-41 through B-46 for Dry Weather Flow Diversion.

Assume volume of litter, V_L, equal to 10 ft^3/ac/yr (0.7 m^3/ha/yr) unless site-specific quantities are available.

Style of device based upon the drop in hydraulic head available, then by expected litter volume, then by Q.

Workshop Exercise - GSRDs

From plan sheet 1 of the GSRD detail plans, determine the screen length for a Linear Radial GSRD for a 'debris area' of 0.35 Ha and a flow rate of 0.09 m^3/sec. ____________ m

6. Traction Sand Traps (PPDG Appendix B, pages B-21 through B-23)

V = (S x R x L x E)/(F)

Where:

V = The total volume of traction sand that must be stored (m^3).
S = The estimated volume of sand applied (m^3/yr). Typical sand application rates range from 47 m^3/lane/km/yr for areas with average application rates to 95 m^3/lane/km/yr for areas with high application rates.
R = A reduction factor to account for sand recovered by roadway sweeping. Assume R = 1.0 (if no roadway sweeping) and 0.6 (if aggressive winter sweeping)
L = A factor to account for other miscellaneous losses/accumulations. Assume that L = 0.8 (high losses from known sources such as snow blowers) to 1.2 (high accumulation from known sources).
E = An estimated recovery efficiency. Assume E = 1.0
F = The number of times the trap will be cleaned (times/yr). Typically F = 1.0

Application rates in July 2005 PPDG, Appendix B, in US Customary units and metric units:

- 2,670 ft^3/lane/mi/year (47 m^3/lane/km/year) is estimated as an "average" application rate
- 5,400 ft^3/lane/km/yr (95 m^3/lane/km/year) is estimated as a "high" application rate

Workshop Exercise - Traction Sand Trap

Project Site Information

a) Mountainous region, traction sand applied at least twice yearly
b) A ¼ mile section of two-lane roadway proposed to capture of traction sand
c) No disqualifying physical siting criteria present

Water Quality information

a) Siting of a Detention Basin cannot be made (first choice for a Traction Sand Device)
b) Depth to groundwater not a design issue

Assumptions and Requirements for Design of the Treatment BMP

a) CMP Riser style devices can be placed, diameter of riser = 3 ft, maximum placement depth at 10 ft below ground surface; outflow pipe at 1.5 ft below ground surface, and 6 inches of clearance maintained between the top of stored sand and the outflow pipe:
b) With the above dimensions: the available volume for storage of traction sand is 57 ft^3 for each riser
c) Other design information given under the To Do section
d) Traction sand application rate: 2,670 ft^3/lane/mi/year.
e) One clean out per year.

To Do

1. For CMP: Riser style Traction Sand Trap: Determine the total volume of traction sand applied to the ¼ mile roadway section. Calculate S, the estimated volume of sand applied (ft^3 or m^3) using the application rate, the number of lanes, and the length of the roadway.

S = application rate x number of lanes x length (miles)
= 2,670 ft^3/lane/mi/year x 2 lanes x ¼ mile = _____ft^3

2. For CMP: Riser style Traction Sand Trap: Determine the total volume of traction sand estimated to be captured by the Traction Sand Trap, using Eqn. 9 of the PPDG and the inputs provided.

V = [S x R x L x E)]/F Eqn. 9, PPDG Pg. B-35

Where:
V = Volume of traction sand that must be stored (ft^3 or m^3): TBD
S = Estimated volume of sand applied (ft^3 or m^3): *Calculated above*
R = Factor for sand recovered by sweeping: *use 0.6*
L = Factor for other miscellaneous losses or accumulations: *use 0.8, high losses*
E = Estimated recovery efficiency: *use 1.0*
F = Number of times the trap will be cleaned (times/yr): *once annually*

V = [S x R x L x E)]/F = ______ x ___ x ___ x ___ x = _____ ft^3

3. For CMP: Riser style Traction Sand Trap: Determine the number of TST riser devices needed to capture the volume V.

Number of risers = V / storage capacity of each riser = _______ / _______ = _______ CMP risers

7. Media Filter — Austin Sand Filter

Refer to PPDG Appendix B pages B-47 through B-54 for Media Filters

Austin Sand Filters

Area of 2nd chamber (filtration chamber) for the Austin Sand Filter

A_{ff} = [FS x C x WQV x d)] / [k x T x (h + d)] (Eq. 10, PPDG Appendix B, page B-51)

where

A_{ff} = area of 2nd chamber filter bed, full sedimentation basin; m^2 or ft^2

FS = factor of safety; 2

C = conversion factor for units of permeability (100 for cm to m; 12 for inches to ft)

WQV = Water Quality Volume; m3

d = depth of sand layer in the Austin-style filter bed, typically 0.46 m (1.5 ft)

k = coefficient of permeability of the filtering medium; cm/hr. metric: 10 cm/hr; US Customary units: 4 inches/hr

T = design drain time for WQV, typically 24 hours

h = average water height above the surface of the media bed, taken as ½ the maximum head of the second chamber (distance to any overflow device to the surface of the media bed); typically. 0.46 m (1.5 ft)

Note: For partial Austin Sand Filter, multiply A_{ff} by 1.8 to calculate A_{fp}.

Delaware Sand Filters

Please refer to PPDG Appendix B pages B-47 through B-54 for Media Filters, but updated design equations are expected in the next printing of the PPDG. Contact HQ Design OSWM if use of Delaware Sand Filters is contemplated.

The table below provides approximate sizes for the DSF at various WQVs.

WQV ft^3	D ft	Width ft	L ft
3500	6	12	88
	6	20	53
	7	12	75
	7	20	45
	8	12	66
	8	20	39
5000	6	16	94
	6	20	75
	7	16	80
	7	20	64
	8	16	70
	8	20	56

WQV ft^3	D ft	Width ft	L ft
10000	6	24	125
	6	30	100
	7	24	107
	7	30	86
	8	24	94
	8	30	75
15000	6	36	125
	7	30	129
	7	36	107
	8	30	113
	8	36	94

Workshop Exercise - AVSFs

The Workshop Exercise for the partial AVSF (Austin Vault Sand Filters) consists of: reviewing the detail sheets provided during this training, and understanding the interrelated design constraints.

8. MCTT

Refer to PPDG Appendix B pages B-55 through B-59 for Media Filters

Chamber volumes: 1st sized typically at 25% of WQV; 2nd at 75% of WQV; 3rd at 75% of WQV.

Calculation of the MCTT Filter Chamber area

Area of 3rd chamber (filtration chamber) for the MCTT:

$A_{ff} = [FS \times C \times WQV \times d)] / [k \times T \times (h + d)]$ (Eq. 19, PPDG Appendix B, page B-58)

Where:

A_{ff} = area of 3rd chamber filter bed, m^2 or ft^2
FS = factor of safety; 2
C = conversion factor for units of permeability (100 for cm to m; 12 for inches to ft)
WQV = Water Quality Volume; m^3
d = depth of sand layer in the Austin-style filter bed, typically 0.46 m (1.5 ft)
k = coefficient of permeability of the filtering medium; cm/hr. metric: 10 cm/hr; (US Customary units: 4 inches/hr)
T = design drain time for WQV, typically 40 to 48 hours
h = average water height above the surface of the media bed, taken as ½ the maximum head of the second chamber (distance to any overflow device to the surface of the media bed); typically. 0.46 m (1.5 ft)

The table below provides approximate sizes for the MCTTs at various WQVs.

MCTT Sizing Summary

WQV	Length	Width	Depth
ft3	ft	ft	ft
3500	90	15	6
	70	15	7
	49	15	9
5000	96	20	6
	75	20	7
	53	20	9
10000	137	28	6
	111	25	7
	79	25	9
15000	145	28	7.5
	105	28	9.5

Workshop Exercise - MCTT

Project Information

a) WQV at 123 m^3
b) No physical or institutional constraint to siting an MCTT
c) Area available for Treatment BMP about 30 m wide by 50 m long

Water Quality information

a) No physical or institutional constraint relating to water quality would prohibit placement

Assumptions and Requirements for Design of the Treatment BMP

a) 40 hour drawdown time
b) 1^{st} chamber: use 25% WQV; 2^{nd} chamber: use 75% WQV; 3^{rd} chamber: use 75% WQV
c) Average depth in all chambers at 1.8 m; assume width at 6 m.

To Do

Determine footprint of the MCTT using the design assumptions for each chamber.

A) 1st chamber at 25%, 2nd chamber at 75% WQV, depths as: 1^{st} chamber: 2.0 m; 2^{nd} chamber.

Volume 1st chamber = 0.25 x 123 = ____ m^3 = L x 6.0 m wide x 1.8 m deep
so L = __________ m

Volume 2nd chamber = 3x the 1^{st} chamber, at same depth and width
so L = __________ mB) Determine footprint of the 3^{rd} chamber for the MCTT, at 100% WQV, using the other terms as given below.

A_f = (FS x C x WQV x d) / (k x T x [h + d])
where:
A_f = TBD); FS = factor of safety, 2; C = 100 cm/m;
Capacity of 3^{rd} chamber = 100% WQV; d = 0.45 m; k = 10 cm/hr; T = 40 hours;
h =

A_f = (2 x 100 x [123 x 0.75] x 0.45) / (10 x 40 x [0.45 + 0.45])

Then, A_f = ______ and length = A_f / 6= _____ m in Length

Total length = _______ + _______ + ______ (ignores walls of the MCTT vault)

9. Wet Basin

Refer to PPDG Appendix B pages B-61 through B-67 for Wet Basins

Key parameters: a) permanent pool sized at 3x the WQV. b) Design is similar to the Detention Basin, with the WQV being treated (by detaining in the Wet Basin) typically 24 to 48 hours, with the Water Quality outlet design using weir or an orifice device. c) Passage of events greater than the WQV event: pass this water from the Wet Basin usually through an overflow weir; maintain 12 inches (300 mm) freeboard above the largest event that can enter the Wet Basin.

The table below provides approximate sizes for Wet Basins at various WQVs, with a 'shorter' and a 'longer' version shown.

Note: to minimize the area available for growth of emergent vegetation, the minimum depth that should be considered is 2.0 m.

WQV (m^3)	Length at WQV surface (m)	Width at WQV surface (m)	Depth (does not include freeboard) (m)
125	22	20	2.8
	28	18	1.9
250	33	23	2.9
	41	21	1.9
370	38	27	3.0
	45	25	2.1
490	44	28	3.0
	50	29	2.0

Workshop Exercise - Wet Basin

Project Site Information

a) WQV at 123 m^3
b) No physical or institutional constraint to siting a Wet Basin
c) Area available for Treatment BMP about 30 m wide by 50 m long

Water Quality information

a) No physical or institutional constraint relating to water quality would prohibit placement

Assumptions and Requirements for Design of the Treatment BMP

a) Trapezoidal shape
b) Depth in Wet Basin: total depth, up to 2 m deep, allowing ¾ the total depth to the Permanent Pool, and the remainder to the WQV pool and freeboard (0.3 m minimum)
c) Permanent Pool: use 3H:1V side slopes, and volume = 3x WQV
d) WQV pool: 4H:1V side slopes

To Do

Use the table elsewhere in this Handout to size this Wet Basin's Permanent Pool and WQV pool, and estimate the available remaining depth for freeboard.

A) Estimate the depth of the permanent pool at 75% of the allowed depth, equal to 1.5 m. From the handout, find two potential inverts that provide between 95% and 105% of the desired volume.

Desired volume = 3x WQV = ___ m^3
Inverts: _________ L_{invert} x ___________ W_{invert} @ volume = _______ %WQV
_________ L_{invert} x ___________ W_{invert} @ volume = _______ %WQV

B) Selected one of the above, and determine the L and W at the surface of the Permanent Pool, using the assigned side slope ratio.
_________ $L_{\text{surface of Permanent Pool}}$ x ____________ $W_{\text{surface of Permanent Pool}}$

C) Use the $L_{\text{surface of Permanent Pool}}$ and the $W_{\text{surface of Permanent Pool}}$ as the invert of the WQV pool, estimate the depth of the WQV pool needed to provide storage between 95% and 105% of the WQV (123 m^3) from the table provided. ___________ m

D) Compare remaining depth of the initial estimate of 2.0 m to the required depths, including freeboard.
$D_{\text{Permanent Pool}}$ + $D_{\text{WQV Pool}}$ + freeboard = ____ + ____ +____ = ________ OK?

Appendix A: Conversions

Length: 1 ft = 0.3048 m, 1 m = 3.28 ft
Area 1 hectare = 10,000 square meters = 0.01 square kilometers = 0.405 acre
Volume: 123 m^3 = 4,344 ft^3
Rates: Litter 10 ft^3/ac/yr = 0.7 m^3/ha/yr
Traction Sand 2,670 ft^3/lane/mi/year = 47 m^3/lane/km/year

Appendix B: Volume of a Trapezoid

$V = [L \times W \times D] + [(L+W) \times Z \times (D^2)] + [1.33 \times (Z^2) \times (D^3)]$

V = volume at a depth D, m^3 (ft^3)
D = depth of trapezoid, m (ft)
L = length of trapezoid at base, m (ft) [Note convention]
W = width of the trapezoid at base, m (ft) [Note convention]
Z = side slope factor, ratio of H:1V [Note convention]

Reference: FHWA Hydraulic Engineering Circular No. 22 URBAN DRAINAGE DESIGN MANUAL Page 8-15; Second Edition, August 2001

See the table that follows for solved volumes of select trapezoids.

Volume of Select Trapezoids

Permanent Pool Volume at Z = 3 (3H:1V)

For D = 1.50

	W =											
L =	***2.0***	***3.5***	***5.0***	***6.5***	***8.0***	***9.5***	***11.0***	***12.5***	***14.0***	***15.5***	***17.0***	***18.5***
10.0	151	184	217	249	282	315	347	380	412	445	478	510
12.0	171	208	245	282	319	357	394	431	468	505	542	579
14.0	190	232	274	315	357	399	440	482	523	565	607	648
16.0	210	256	302	348	394	441	487	533	579	625	671	717
18.0	229	280	331	381	432	483	533	584	634	685	736	786
20.0	249	304	359	414	469	525	580	635	690	745	800	855
22.0	268	328	388	447	507	567	626	686	745	805	865	924
24.0	288	352	416	480	544	609	673	737	801	865	929	993
26.0	307	376	445	513	582	651	719	788	856	925	994	1,062
28.0	327	400	473	546	619	693	766	839	912	985	1,058	1,131

WQ Volume at Z = 4 (4H:1V)

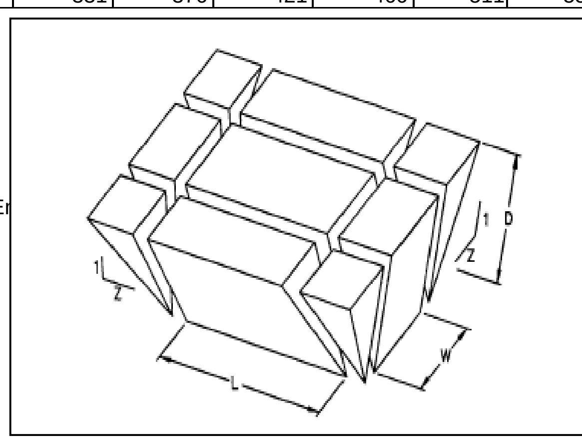
For D = 0.25

	W =											
L =	***2.0***	***5.0***	***8.0***	***11.0***	***14.0***	***17.0***	***20.0***	***23.0***	***26.0***	***29.0***	***32.0***	***35.0***
10.0	8	17	25	33	41	50	58	66	74	83	91	99
12.0	10	20	29	39	49	59	68	78	88	98	107	117
14.0	11	23	34	45	56	68	79	90	101	113	124	135
16.0	13	26	38	51	64	77	89	102	115	128	140	153
18.0	14	29	43	57	71	86	100	114	128	143	157	171
20.0	16	32	47	63	79	95	110	126	142	158	173	189
22.0	17	35	52	69	86	104	121	138	155	173	190	207
24.0	19	38	56	75	94	113	131	150	169	188	206	225
26.0	20	41	61	81	101	122	142	162	182	203	223	243
28.0	22	44	65	87	109	131	152	174	196	218	239	261

For D = 0.50

	W =											
L =	**2.0**	**5.0**	**8.0**	**11.0**	**14.0**	**17.0**	**20.0**	**23.0**	**26.0**	**29.0**	**32.0**	**35.0**
10.0	25	43	61	79	97	115	133	151	169	187	205	223
12.0	29	50	71	92	113	134	155	176	197	218	239	260
14.0	33	57	81	105	129	153	177	201	225	249	273	297
16.0	37	64	91	118	145	172	199	226	253	280	307	334
18.0	41	71	101	131	161	191	221	251	281	311	341	371
20.0	45	78	111	144	177	210	243	276	309	342	375	408
22.0	49	85	121	157	193	229	265	301	337	373	409	445
24.0	53	92	131	170	209	248	287	326	365	404	443	482
26.0	57	99	141	183	225	267	309	351	393	435	477	519
28.0	61	106	151	196	241	286	331	376	421	466	511	556

V = (L)(W)(D) + (L+W)(Z)(D^2) + 1.33(Z^2)(D^3)
V = volume at a depth D, m^3 (ft^3)
D = depth of trapezoid, m (ft)
L = length of trapezoid at base, m (ft) [Note convention]
W = width of the trapezoid at base, m (ft) [Note convention]
Z = side slope factor, ratio of H:V [Note convention]
From Page 8-15, FHWA "Urban Drainage Design Manual, Hydraulic E
Circular No. 22," 2nd edition, August 2001
Note: W and L are interchangeable in the above formulas.



<u>Notes</u>

Notes