June 25, 2015

Errata Sheet for 2015 Portneuf Valley Stormwater Manual

Note that Pocatello and Chubbuck have updated the online pdf versions with this erratum. Copies of the 2015 PVSDM downloaded after June 25th, 2015 will have all of these changes incorporated for the reader's benefit.

1. Page 2-9; "Table 2-1": Change to "Table 2-2".

2. Page 4-11; Under "Sheet Flow". Replace "Table 4-3" with "Table 4-4"

3. Page 4-12; Under "Shallow Concentrated Flow", change the second sentence to:

   The average velocity is calculated using velocity equation below.

4. Page 4-12; Under "Velocity Equation", change "Table 4-3" to "Table 4-4" (occurs twice).

5. Page 4-12; Under "Open Channel Flow", replace the sentence that begins "the kc values.." to:

   The k_c values from Table 4-4 (used in the above velocity equation) or water surface profile information can be used to estimate average flow velocity.

6. Page 4-12; Gray and bold the following lines:

SHALLOW, CONCENTRATED FLOW	**k_s**
CHANNEL FLOW (INTERMITTENT, R = 0.2)	**kc**
CHANNEL FLOW (CONTINUOUS STREAM, R =0.4)	**kc**

7. Page 4-15; "Runoff coefficients"; Replace 0.94 with 0.95.

8. Page 4-16; Table 4-6: Footnote #1 – Replace text with the following:

   [1]*When designing for the 25-, 50-, or 100-year storm events, increase the runoff coefficients by 10%, 20%, and 25% respectively. Coefficients should not exceed 0.95.*

9. Page 4-17; Last Paragraph; Replace second sentence with the following:

   An example time of concentration calculation is provided in Step 2 of Appendix 4C.

10. Page 4-20; Item #2; Replace "Table 4-6" with "Table 4-2"

11. Pages 4-27 to 4-29; Appendix 4C: Replace entire Appendix to accommodate changing the Runoff Coefficient values per Table 4-6 (to 0.22 and 0.95 for lawns and impervious areas, respectively),

and to accommodate a formula error in the worksheet example spreadsheet. The minimum storage required should be changed to 4,447 cu ft.

12. Page 6-7; Section 6.4.6 – 5th bullet down: Replace "see Chapter 4" with "see Chapter 3"

13. Page 6-9 under "Access". Replace "slide" with "side" at the end of the 2nd paragraph.

14. Page 6-18; Section 6.6.2 Construction Monitoring: Under the heading "Construction Inspections", add bullets following the second paragraph.

15. Page 6-20; Figure 6-5. Add the text "See Section 6.4.6 for design criteria" to the note.

16. Page 6-31; Table 6-3:

    Left hand column: replace "Soil Depth" with "Required soil depth to groundwater/bedrock"; replace "Soil Type: with "Required Soil Type";

    Bottom column: Add a new bottom column:

Treatment Required?[2,3,4]	All sites: Pretreatment.	Low – moderate use sites: Pretreatment; High use sites: Primary Treatment.	Low use sites: Pretreatment; High- moderate use sites: Primary Treatment.	All sites: Primary Treatment.

    Footnotes: Add the following text to footnote #4: "See Table 6-1 for appropriate pre-treatment and treatment facilities";

17. Page 6-32; Section 6.7.1. Delete 6.7.1.2 from the 3rd paragraph under Design Criteria. The paragraph should now read:

    > *Figures 6-13 and 6-14 are illustrations of this BMP. Control structures and overflows shall be designed following the guidelines in Section 6.7.1.1.*

18. Page 6-33; Figure 6-14. Replace text after NOTE with the following:

    > *This detail is a schematic representation only. Actual configuration will vary depending on specific site constraints and applicable design criteria. See Section 6.7.1 for design criteria.*

19. Page 6-33; Figure 6-14. Add "per 7.4.1" to text describing the emerging overflow spillway rock lining.

20. Page 6-34; Figure 6-15. Add NOTE with the following:

    NOTE: This detail is a schematic representation only. Actual configuration will vary depending on specific site constraints and applicable design criteria. See Section 6.7.1 for design criteria.

21. Page 6-34; Figure 6-15. Add “and fabric per 7.4.1” following call outs for rock lining.

22. Page 6-34; Figure 6-15. Replace asphalt call out in Section B-B with the following text:

    Asphalt/concrete and fabric per 7.4.1 (for spillway on access roads)

23. Page 6-35; “Control Structures” Replace “Table 6-2” with “Table 6-4”

24. Page 6-36; Figure 6-15. Remove callout for manhole ladders.

25. Page 7-3; 1st bullet; Replace “Table 7-4” with “Table 4-5”

26. Page 7-5; “Depth” Replace paragraph with the following:

    See Table 2-2 for minimum depth requirements for constructed channels.

27. Page 9-9: At top of page change “Contact Phone No: ____/____/____” to:

    “Contact Phone No: _____-_______-_______“

The source control measures applicable to various site uses are outlined in Chapter 5.

2.3.4 CORE ELEMENT #4
NATURAL AND CONSTRUCTED CONVEYANCE SYSTEMS

Conveyance systems are natural or constructed components that collect stormwater runoff in a manner that adequately drains structures, sites and roadways, minimizing the potential for flooding and erosion. Engineered conveyance systems are designed and constructed to provide protection against damage to property and improvements from uncontrolled or diverted flows, flooding and erosion.

Natural drainage features, including floodplains, drainage ways, and natural depressions, store water or allow it to infiltrate into the ground. These features are referred to as the "natural location of drainage systems" (NLDS). Projects shall be designed to protect the NLDS to ensure that stormwater runoff can continue to be conveyed and disposed of at its natural location.

Stormwater runoff shall be discharged in the same manner and at the same location as in the pre-developed condition, unless otherwise specifically accepted by the local jurisdiction. Stormwater runoff may not be concentrated onto down-gradient properties where sheet flow previously existed nor diverted to points not receiving stormwater runoff prior to development.

Applicability
All projects, regardless of whether they meet the regulatory threshold, shall comply with the Core Element for conveyance systems (unless they are exempt as per Sections 2.3.5).

NLDS
Project proponents are required preserve natural drainage systems as specified in Section 7.4.2 of this Manual.

Conveyance Sizing
Constructed channels shall be designed with sufficient capacity to convey and contain, at a minimum, the 50-year peak flow, plus the following freeboard requirements, assuming developed conditions for onsite tributary areas and existing conditions for any offsite tributary areas. Refer to Chapter 7 for additional criteria.

Table 2-1: Freeboard for Constructed Channels

Water depth	Freeboard
<12″	4″
12-24″	8″
>24“	12″

The design shall bypass storm events that exceed the above criteria and shall provide an overflow path, with the capacity to convey the 100-year storm event. The overflow path should drain toward the natural discharge point of the contributing basin, away from adjacent buildings, residences, etc.

New culverts shall be designed with sufficient capacity to convey the 50-year design storm assuming developed conditions for the onsite basin and existing conditions for the offsite basin. Increase culvert size to pass the 100-year event if a safe overflow for flows above the 50-year event cannot be provided.

Additionally, new culverts shall be designed with sufficient capacity to meet the headwater and tailwater requirements in Chapter 7.

New enclosed systems and inlets shall be designed with sufficient capacity to convey peak flow rate for the 50-year design storm event with at least six-inches of freeboard between the water surface and the proposed ground surface. Enclosed systems may surcharge or overtop drainage structures for storm events that exceed the 50-year event, so long as an overflow path is provided. The overflow path must be capable of conveying the 100-year storm event and should drain toward the natural discharge point of the contributing basin, away from adjacent buildings, residences, etc.

Drainage inlets shall be designed with sufficient capacity to convey the 50-year design storm assuming developed conditions for the contributing area.

2.3.5 CORE ELEMENT #5
ONSITE STORMWATER MANAGEMENT

Projects shall employ On-site Stormwater Management BMPs to infiltrate, disperse, and retain stormwater runoff on-site to the extent feasible without causing flooding or erosion impacts. The objective of onsite stormwater managment BMPs is to use practices distributed across a development that reduce the amount of disruption of the natural hydrologic characteristics of the site.

Applicability

All projects that meet the regulatory threshold (see Section. 2.2.1) and discharge to surface water (or a stormwater system that connects to surface water) must meet the requirements listed in Table 2-1 for onsite retention of stormwater.

Design Criteria

Project proponents are encouraged to utilize Low Impact Development site design (see Chapter 10) to achieve this objective. Flow Control and Treatment BMPs (Chapter 6) may also be used.

Travel time (T_t) is the time it takes stormwater runoff to travel from one location to another in a watershed. Time of concentration (T_c) is the time for stormwater runoff to travel from the hydraulically most distant point to the point of discharge of a watershed. T_c is computed by adding all the travel times for consecutive components of the drainage conveyance system as given by the following equation:

$$T_c = T_{t1} + T_{t2} + \dots T_{tn}$$

Where:
- T_c = time of concentration (min); (minimum 5 minutes)
- n = number of flow segments; and
- T_t = travel time (min) is the ratio of flow length to flow velocity given by:

$$T_t = \frac{L}{60V}$$

Where:
- L = flow length (ft);
- V = average velocity (ft/s); and,
- 60 = conversion factor (seconds to minutes).

T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c, thereby increasing the peak discharge. But T_c can be increased as a result of ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts, or reduction of land slope through grading. Note: the minimum T_c for any runoff calculations should be 5 minutes.

Sheet Flow

Sheet flow is flow over plane surfaces and shall not be used over distances exceeding 300 feet. Use Manning's kinematic solution to directly compute T_t:

$$T_t = \frac{0.42(n_s L)^{0.8}}{(P_2)^{0.5}(S_0)^{0.4}}$$

Where:
- T_t = travel time (min);
- n_s = Manning's effective roughness coefficient for sheet flow (use Table 4-4);
- L = flow length (ft);
- P_2 = 2-year, 24-hour rainfall (in); and,
- S_0 = slope of hydraulic grade line (land slope, ft/ft).

The friction value (n_s) is used to calculate sheet flow. The friction value is Manning's effective roughness coefficient modified to take into consideration the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. The n_s values are for very shallow flow depths of about 0.1 foot and are only used for travel lengths up to 300 feet. Table 4-4 gives Manning's n_s values for sheet flow for various surface conditions.

Shallow Concentrated Flow

After 300 feet, sheet flow is assumed to have developed into shallow concentrated flow. The average velocity is calculated using velocity equation below.

Velocity Equation

A commonly used method of computing average velocity of flow, once it has measurable depth, is the following equation:

$$V = k\sqrt{S_0}$$

Where: V = velocity (ft/s);
k = k_s or k_c, time of concentration velocity factor (ft/s); and,
S_o = slope of flow path (ft/ft).

Table 4-4 provides "*k*" for various land covers and channel characteristics with assumptions made for hydraulic radius. For flow situations not addressed in Table 4-4, calculate "*k*" using the following equation:

$$k = \frac{1.49R^{2/3}}{n}$$

Where: R = hydraulic radius; and,
n = Manning's roughness coefficient for open channel flow (Table 4-4 or 4-5).

Open Channel Flow

Open channels are assumed to exist where channels are visible on aerial photographs, where streams appear on United States Geological Survey (USGS) quadrangle sheets, or where topographic information indicates the presence of a channel. The k_c values from Table 4-4 (used in the above velocity equation) or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full conditions. After average velocity is computed the travel time (T_t) for the channel segment can be computed.

Table 4-4: Friction Values (n and k) for Computing Time of Concentration

SHEET FLOW [1]	n_s
Bare sand	0.010
Smooth surfaces (concrete, asphalt, gravel, or bare hard soil)	0.011
Asphalt and gravel	0.012
Fallow fields of loose soil surface (no vegetal residue)	0.05
Cultivated soil with crop residue (slope < 0.20 ft/ft)	0.06
Cultivated soil with crop residue (slope > 0.20 ft/ft)	0.17
Short prairie grass and lawns	0.15
Dense grass	0.24
Bermuda grass	0.41
Range, natural	0.13
Woods or forest, poor cover	0.40
Woods or forest, good cover	0.80
SHALLOW, CONCENTRATED FLOW	k_s
Forest with heavy ground litter and meadows (n = 0.10)	3
Brushy ground with some trees (n =0.06)	5
Fallow or minimum tillage cultivation (n = 0.04)	8
High grass (n = 0.035)	9
Short grass, pasture and lawns (n = 0.030)	11
Newly-bare ground (n = 0.025)	13
Paved and gravel areas (n = 0.012)	27
CHANNEL FLOW (INTERMITTENT, R = 0.2)	k_c
Forested swale with heavy ground litter (n=0.10)	5
Forested drainage course/ravine with defined channel bed (n=0.050)	10
Rock-lined waterway (n=0.035)	15
Grassed waterway (n=0.030)	17
Earth-lined waterway (n=0.025)	20
CMP pipe (n=0.024)	21
Concrete pipe (n=0.012)	42
Other waterways and pipes	0.508/n
CHANNEL FLOW (CONTINUOUS STREAM, R =0.4)	k_c
Meandering stream with some pools (n=0.040)	20
Rock-lined stream (n=0.035)	23
Grassed stream (n=0.030)	27
Other streams, man-made channels and pipe	0.807/n

[1] These values were determined specifically for overland (sheet) flow conditions and are not appropriate for conventional open channel flow calculations.

Source: WSDOT Hydraulics Manual, March 2004; Engman (1983) and the Florida Department of Transportation Drainage Manual (1986).

TABLE 4-5: SUGGESTED VALUES OF THE MANNING'S ROUGHNESS COEFFICIENT "n" FOR CHANNEL FLOW

TYPE OF CHANNEL AND DESCRIPTION	*"n"*[1]
A. CONSTRUCTED CHANNELS	
a. Earth, straight and uniform	
1. Clean, recently completed	0.018
2. Gravel, uniform selection, clean	0.025
3. With short grass, few weeds	0.027
b. Earth, winding and sluggish	
1. No vegetation	0.025
2. Grass, some weeds	0.030
3. Dense weeds or aquatic plants in deep channels	0.035
4. Earth bottom and rubble sides	0.030
5. Stony bottom and weedy banks	0.035
6. Cobble bottom and clean sides	0.040
c. Rock lined	
1. Smooth and uniform	0.035
2. Jagged and irregular	0.040
d. Channels not maintained, weeds and brush uncut	
1. Dense weeds, high as flow depth	0.080
2. Clean bottom, brush on sides	0.050
3. Same, highest stage of flow	0.070
4. Dense brush, high stage	0.100
B. NATURAL STREAMS	
B-1 Minor Streams (top width at flood stage < 100ft.)	
a. Streams on plain	
1. Clean, straight, full stage, no rifts or deep pools	0.030
2. Same as above, but more stones and weeds	0.035
3. Clean, winding, some pools and shoals	0.040
4. Same as above, but some weeds	0.045
5. Same as 4, but more stones	0.050
6. Sluggish reaches, weedy deep pools	0.070
7. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.100
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages	
1. Bottom: gravel, cobbles and few boulders	0.040
2. Bottom: cobbles with large boulders	0.050
B-2 Flood Plains	
a. Pasture, no brush	
1. Short grass	0.030
2. High grass	0.035
b. Cultivated areas	
1. No crop	0.030
2. Mature row crops	0.035
3. Mature field crops	0.040
c. Brush	
1. Scattered brush, heavy weeds	0.050
2. Light brush and trees	0.060
3. Medium to dense brush	0.070
4. Heavy, dense brush	0.100
d. Trees	
1. Dense willows, straight	0.150
2. Cleared land with tree stumps, no sprouts	0.040
3. Same as above, but with heavy growth of sprouts	0.060
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.100
5. Same as above, but with flood stage reaching branches	0.120

[1]The "n" values presented in this table are the "Normal" values as presented in Chow (1959). For an extensive range and for additional values refer to Chow (1959).

4.4 RATIONAL METHOD

The rational method is used to predict peak flows for small undeveloped or developed drainage areas. The rational method can be used for the design of conveyance, flow control, and subsurface infiltration facilities. The greatest accuracy is obtained for areas smaller than 10 acres and for developed conditions with large impervious areas. In the Portneuf Valley, the Rational Method is recommended for small projects (less than 25 acres) in urbanized areas. For larger basins, the Curve Number Method in Section 4.3 should be used whenever possible. The rational method peak flow rate is calculated using the following equation:

$$Q_P = CIA$$

Where:
- $Q_P =$ peak flow rate (cfs);
- $C =$ runoff coefficient (dimensionless units);
- $I =$ rainfall intensity (in/hr); and,
- $A =$ drainage area (acres).

Calculation details are provided in the Section 4.4.1 through 4.4.3. An example runoff calculation using the Rational Method is included as steps 1-4 in the Bowstring Method example in Appendix 4C.

4.4.1 RUNOFF COEFFICIENTS

Table 4-6 provides runoff coefficients for the 10-year storm frequency. Less frequent, higher intensity storms require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. When designing for a 24-, 50-, or 100-year frequency, runoff coefficients should be increased by 10 percent, 20 percent, and 25 percent respectively. Runoff coefficients shall not be increased above 0.95. Higher values may be appropriate for steeply sloped areas and/or longer return periods, because in these cases infiltration and other losses have a proportionally smaller effect on runoff.

Table 4-6: Runoff Coefficients for the Rational Method - 10-Year Return Frequency[1]

TYPE OF COVER	FLAT (<2%)	ROLLING (2% - 10%)	HILLY (>10%)
Pavement and Roofs	0.90	0.90	0.90
Earth Shoulders	0.50	0.50	0.50
Drives and Walks	0.75	0.80	0.85
Gravel Pavement	0.85	0.85	0.85
City Business Areas	0.80	0.85	0.85
Apartment Dwelling Areas	0.50	0.60	0.70
Light Residential: 1 to 3 units/acre	0.35	0.40	0.45
Normal Residential: 3 to 6 units/acre	0.50	0.55	0.60
Dense Residential: 6 to 15 units/acre	0.70	0.75	0.80
Lawns	0.17	0.22	0.35
Grass Shoulders	0.25	0.25	0.25
Side Slopes, Earth	0.60	0.60	0.60
Side Slopes, Turf	0.30	0.30	0.30
Median Areas, Turf	0.25	0.30	0.30
Cultivated Land, Clay and Loam	0.50	0.55	0.60
Cultivated Land, Sand and Gravel	0.25	0.30	0.35
Industrial Areas, Light	0.50	0.70	0.80
Industrial Areas, Heavy	0.60	0.80	0.90
Parks and Cemeteries	0.10	0.15	0.25
Playgrounds	0.20	0.25	0.30
Woodland and Forest	0.10	0.15	0.20
Meadow and Pasture Land	0.25	0.30	0.35
Unimproved Areas	0.10	0.20	0.30

Source: ODOT Hydraulics Manual, June 2006

[1]When designing for the 25-, 50-, or 100-year storm events, increase the runoff coefficients by 10%, 20%, and 25% respectively. Coefficients should not exceed 0.95.

4.4.2 TIME OF CONCENTRATION

The travel time, the time required for flow to move through a flow segment, shall be computed for each flow segment. The flow path for each basin should be divided into segments representing different land cover and flow types (i.e. overland flow through grass vs. shallow gutter flow). The time of concentration is equal to the sum of the travel times for all flow segments. As with the Curve Number method, any overland flow segments should be limited to 300 feet in length.

The procedure described below was developed by the NRCS. It is sensitive to slope, type of ground cover, and the size of channel. The time of concentration can be calculated as follows:

$$T_t = \frac{L}{K\sqrt{S}}$$

$$T_c = T_{t1} + T_{t2} + \ldots + T_{tn}$$

Where:
- T_t = travel time of flow segment (min);
- T_c = time of concentration (min);
- L = length of segment (ft);
- K = ground cover coefficient, Table 4-7 (ft/min);
- S = slope of segment (ft/ft); and,
- n = number of flow segments.

The time of concentration for any one basin shall not be less than 5 minutes. An example time of concentration calculation is provided in Step 2 of Appendix 4C. For a few drainage areas, the time of concentration that produces the largest amount of runoff is less than the time of concentration for the entire basin. This can occur when two or more basins have dramatically different types of cover. The most common case would be a large paved area together with a long narrow strip of natural area. In this case, the Engineer shall check the runoff produced by the paved area alone to determine if this scenario would cause a greater peak runoff rate than the peak runoff rate produced when both land segments are contributing flow. The scenario that produces the greatest runoff shall be used, even if the entire basin is not contributing flow to this runoff.

Table 4-7: Ground Cover Coefficients

TYPE OF COVER	K (ft/min)
Forest With Heavy Ground Cover	150
Minimum Tillage Cultivation	280
Short Pasture Grass Or Lawn	420
Nearly Bare Ground	600
Small Roadside Ditch W/Grass	900
Paved Area	1,200
Gutter Flow:	
4 inches deep	1,500
6 inches deep	2,400
8 inches deep	3,100
Storm Sewers:	
12 inch diameter	3,000
18 inch diameter	3,900
24 inch diameter	4,700
Open Channel Flow (n = .040):	
12 inches deep	1,100
Narrow Channel (w/d =1):	
2 feet deep	1,800
4 feet deep	2,800
Open Channel Flow (n = .040):	
1 foot deep	2,000
Wide Channel (w/d =9):	
2 feet deep	3,100
4 feet deep	5,000

Source: WSDOT Hydraulics Manual, March 2004;

4.4.3 INTENSITY

Rainfall intensity is related to rainfall duration and the recurrence interval (or frequency) of the design storm. Rainfall Intensity-Duration-Frequency Interval (IDF) curves for the Portneuf Valley are as provided in Appendix 4A.

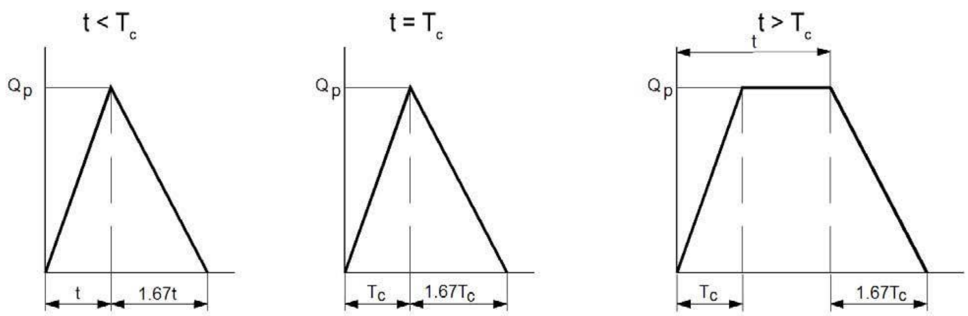
For each project, first determine the appropriate curve to use based on the project location and the IDF Curve Map. Then calculate the appropriate rainfall intensity for each basin based on the time of concentration (T_c) calculated in Section 4.4.2 and the desired storm recurrence interval. Curves are provided for the 2, 5, 10, 25, 50, and 100-year storm events.

4.4.4 BOWSTRING METHOD

This method is used to estimate storage requirements for a given design storm using a series of hydrographs for different storm durations (*t*). It is recommended for small projects (under 25 acres) in urbanized areas. Larger projects should use the NRCS Curve Number Method.

Depending on the relative magnitude of the time of concentration (T_c) and the storm duration, the shape of the hydrograph generated with this method varies from triangular to trapezoidal (see Figure 4.2).

FIGURE 4-2 BOWSTRING HYDROGRAPH



The recession period (T_R) of the hydrograph is given by the following equation:

$$T_R = 1.67T_P$$

Where: $T_P = T_c$, when $t \geq T_c$; or
$T_P = t$, when $t < T_c$.

The volume (*V*) under the hydrograph at a given time (t) is given by:

$$V(t) = 1.34Q_P\, t \text{ for } t \leq T_c \text{ (triangular hydrograph)}$$

$$V(t) = Q_P t + 0.34Q_P T_c \text{ for } t > T_c \text{ (trapezoidal hydrograph)}$$

With these equations, the base of the triangular hydrograph is equal to 2.67*t*. For the trapezoidal hydrograph, the time base is $t + 1.67T_c$. The top width of the trapezoid is equal to $t - T_c$. With this method, the hydrograph for each storm duration is overlaid with the outflow hydrograph. The outflow hydrograph is given by the following equation:

$$V_{OUT}(t) = Q_{OUT}\, t$$

The critical storm duration is the storm duration that results in the maximum required flow control storage.

DESIGN STEPS

The following steps outline how to use the spreadsheet referenced in Appendix 4C. The spreadsheet was created specifically for projects in the Portneuf Valley. Users of spreadsheet must understand input data, output results and must certify that results are accurate regardless of spreadsheet output. The highlighted fields in the spreadsheet require input or consideration of the designer. An example of the spreadsheet input and results for a sample project site is shown in Appendix 4C.

For detention and retention pond design using the Bowstring Method, the following procedure can be used:

1. Determine the weighted Runoff Coefficient (C) for the post-developed condition. Refer to Table 4-2.
2. Calculate Time of Concentration (T_c) using Rational Method. Refer to Section 4.3.6. T_c shall not be less than 5 minutes.
3. Calculate Intensity (I) for T_c. Refer Section 4.4.3 and Appendix 4A.
4. Compute flow rate Q_p for $t = T_c$ for the post-developed condition using Rational Method equation in Section 4.4.
5. Calculate allowable release rate (Q_{OUT}). For infiltration facilities, Q_{OUT} is the calculated rate being discharge into subsurface soils based upon the design infiltration rate as determined in the GSR. For detention facilities Q_{OUT} is limited to either the pre-developed peak flow rate or 0.1 cfs/acre (which ever is less). The Bowstring Method is not intended to size evaporation ponds. Refer to the Water Budget Method if using 0.00 cfs as a release rate.
6. Calculate the outflow volume $V_{OUT} = Q_{OUT} * t$.
7. Compute intensities (I), peak flow rates (Q_p), and inflow and outflow volumes (V, V_{OUT}) for various times (i.e. t = 5, 10, 25… minutes).
8. The required storage is obtained as the maximum difference between inflow and outflow volumes by the tabular methods as shown in the sample spreadsheet.

APPENDIX 4C – BOWSTRING METHOD EXAMPLE

GIVEN:
The existing site is approximately 5-acres, consisting of silty soils. Existing surface vegetative conditions include short grass and weeds.

Post-developed site conditions are as follows:

- 20-10,000 square foot (s.f.) residential lots;
- 1,500 s.f. homes with 750 s.f. concrete driveways;
- 0.50 acres of street impervious surface; and,
- Topography 2%-5%.

Post-developed time of concentration

- 100-ft of overland flow @ 3.0%;
- 300 feet of gutter flow @ 3.0%; and,
- 300 feet of pipe flow @ 2.0%.

Project proponent proposes a bio-infiltration swale with overflow to a subsurface infiltration gallery.

CALCULATIONS:
1. Determine the weighted Runoff Coefficient (C) for the post-developed condition:

From Table 4-6:

Lawns (silty soils, rolling 2%-10%): C= 0.22 * 1.2 (50 year storm conversion) = 0.264.

Streets, driveways, roofs and sidewalks: C = 0.90 * 1.2 (50 year storm conversion) = 1.08. Reduce to C= 0.95 per Table 4-6.

Total Area Breakdown

Roof Area	= 20 homes *1,500 s.f./home	= 30,000 s.f.
Square Feet to Acres	= 30,000 s.f./43,560 s.f. per acre	= 0.689 acres
Driveway Area	= 20 homes * 750 s.f./home = 15,000 s.f.	= 0.344 acres
Streets		= 0.500 acres
Lawn/Landscape	= 5.0 ac – 0.689ac – 0.344ac – 0.500ac	= 3.467 acres
Weighted C	= ((3.467*0.264) + (1.533*0.94))/1	= 0.47

2. Determine Time of Concentration (T_c)

Ground Cover Coefficient (K): Refer to Table 4-7

Flow Segment of Travel Time (T_t): $T_t = \frac{L}{K\sqrt{S}}$

Table 4C-1

Flow Segment	Length (feet)	Slope (feet/foot)	K (feet/minute)	T_t (minutes)
Overland Flow	100	0.03	420	1.375
Gutter Flow	300	0.03	1500	1.155
Pipe Flow	300	0.02	3000	0.707
Total Time of Concentration (T_c)				3.24

3. Determine storm intensity using Appendix 4A.

   For a 50-year storm (design event for many areas in the Portneuf Valley) and a T_c of 3.24 minutes: I = 4.19 in/hr. Since $T_c < 5$ min, use $T_c = 5$ min = 3.88 in/hour

4. Determine the peak flow rate for $t = T_c$ using equation $Q_P = CIA$

   Q_P = 0.47 * 3.88 inches/hour * 5.0 acres = 9.13 cfs

5. Compute the volume for t = Tc using equation $V(t) = 1.34Q_P\ t$

   V(t) = 1.34 * 9.13 cfs * 5.0 min * 60 sec/min

   = 3,669 cubic feet

6. Determine the allowable release rate (Q_{OUT})

   For this example, Q_{OUT} is determined to be 2.0 cfs.

7. Compute the outflow volume (V_{OUT}) for $t = T_c$

   $V_{OUT}(t) = Q_{OUT} * t$

   = 2 cfs * 5.0 min * 60 sec/min = 600 c.f.

8. Compute intensities (I), peak flow rates (Q_p), and inflow and outflow volumes (V, V_{OUT}) for various times (i.e. t = 5, 10, 25… minutes). A spreadsheet can be created to perform this task. Refer to the following sample spreadsheet.

9. The required storage volume is obtained as the maximum difference between inflow and outflow volumes. Refer to the following sample spreadsheet. Minimum required storage volume is 4,618 cubic feet.

Example Spreadsheet
Areas colored blue must be entered by the project designer. Local jurisdictions may provide excel versions of this spreadsheet to aid calculations.

sample BOWSTRING (MODIFIED RATIONAL) WORKSHEET

Design Storm	50	year	**PROJECT :**	**Flows Downhill Estates**
Area	5.00	acres	**BASIN:**	**#1 of 3**
Time of Concentration	3.24	min	**DESIGNER:**	**Drip Drop**
Weighted Cpost	0.47		**DATE:**	**6/1/2015**
Allowable Release Rate	2	cfs		

TIME t(min)	TIME t(sec)	Intensity (in/hr)	Qp(cfs)	Vin (cu.ft)	Vout (cu.ft)	Storage cu.ft
5.00	300	3.88	9.13	3669	600	3069
5	300	3.88	9.13	3669	600	3069
10	600	3.01	7.08	4970	1200	3770
15	900	2.54	5.98	5991	1800	4191
20	1200	2.28	5.37	6988	2400	4588
25	1500	2.02	4.76	7618	3000	4618
30	1800	1.76	4.14	7881	3600	4281
35	2100	1.66	3.89	8565	4200	4365
40	2400	1.55	3.64	9098	4800	4298
45	2700	1.44	3.38	9479	5400	4079
50	3000	1.33	3.13	9707	6000	3707
55	3300	1.22	2.88	9784	6600	3184
60	3600	1.12	2.62	9708	7200	2508
65	3900	1.07	2.52	10091	7800	2291
70	4200	1.03	2.42	10413	8400	2013
80	4800	0.94	2.22	10875	9600	1275
90	5400	0.86	2.02	11096	10800	296
100	6000	0.77	1.81	11074	12000	-926
110	6600	0.69	1.61	10810	13200	-2390
120	7200	0.60	1.41	10303	14400	-4097
					Minimum Storage required	**4618**

INPUT NEEDED
OUTPUTS

Define Cpost

Area	Type	Cpost	sq.ft	acres
1	**Driveways/Roofs/Roads**	0.95	66780	1.533
2	**Lawns**	0.264	151000	3.466
				0.000
				0.000
			total acres	**5.00**
			Weighted C	**0.47**

Define Time of Concentration

Flow Segment	Length(ft)	Slope(ft/ft)	K(ft/min)	Tt(min)	
Overland flow	100	0.03	420	1.37	*Length (L): Measured from site plan*
Gutter flow	300	0.03	1500	1.15	*Ground Cover Coefficient (K): Table 4-7*
Pipe flow	300	0.02	3000	0.71	*Flow Segment Travel Time(Tt): Tt/(K*S^.5)*
Total L = hydraulic length.			**Tc(min)**	**3.24**	

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located as close to the source of oil-generating activity as possible. They should be located upstream of flow control facilities wherever possible. All infiltration facilities shall have pretreatment facilities installed upstream of the infiltration facility in order to collect TSS before contact with the permeable soil layer.

6.4.5 BYPASSES

See Table 2-1 and Section 2.3.6 for basic requirements. For flow based treatment facilities, a flow splitter manhole or vault is typically located upstream of the treatment facility with an orifice or weir designed to divert only the desired flow into the facility. The splitter should be designed such that the maximum flow through the treatment facility is equal to or lower than the flow rate of the Water Quality Design storm. The bypass conveyance system is sized to handle the peak expected bypass flows.

For a volume based treatment facility, the bypass is typically an elevated outlet or other overflow structure located above the water quality design volume. The bypassed water may flow to another treatment facility or directly into a conveyance or infiltration facility.

6.4.6 SETBACKS, SLOPES, EMBANKMENTS & SPILLWAYS

Setbacks

Adequate room for maintenance equipment should be considered during site design. Pond overflow structures shall be located a minimum of 10 feet from any structure or property line. The toe of the berm shall be a minimum of 5 feet from any structure or property line. Evaluate all proposed overflow paths to ensure that runoff will not pose a danger to the public or cause any adverse impact to downstream properties or structures.

The following setbacks apply for stormwater infiltration and treatment facilities:

- Located at least 200' from water supply wells;
- Located at least 100' from streams and other open water features.
- Setback from septic and drain fields per state standards;
- Located at least 20' from building foundations (the local jurisdiction's engineer may approve facilities with smaller setbacks);
- Located at least 50' (or the height of the slope – whichever is greater) away from slopes over 15%. The GSC (see Chapter 3) must address the potential impact of any facilities sited on or near a steep slope;
- Shall not be located where they can impact a soil or groundwater contamination site as identified by DEQ; and
- Additional setback criteria for specific UIC facilities (e.g. dry wells and infiltration trenches/galleries) is provided in Section 6.6.

Side Slopes and Embankments

- Side slopes should not be steeper than a slope of 3H:1V. Recommend using 4H:1V if side slopes are to be grassed and mowed. Interior side slopes may be increased to a maximum of 2H:1V if the surrounding grade creates a cut or fill with no greater depth than 1.0 foot;
- City maintained ponds must have interior and exterior side slopes of 4:1 if they are to be mowed.
- Moderately undulating slopes are acceptable and can provide a more natural setting for the facility. In general, gentle side slopes improve the aesthetic attributes of the facility and enhance safety.
- Interior side slopes may be vertical retaining walls, with the following conditions:
    - Walls 2.5 feet or taller shall have a fence along the top of the wall;
    - The retaining wall design shall be prepared and sealed by a licensed civil engineer, when required by code; and
    - An access ramp (with slopes less than 4H:1V) to the bottom shall be provided.
- Exterior side slopes that are steeper than 2H:1V shall be analyzed for stability by a geotechnical engineer.
- The height of an embankment is measured from the top of the bank to the catch point of the native soil at the lowest elevation. Embankments shall meet the following minimum requirements:
    - Embankments 4 feet in height or more shall be constructed as recommended by a Geotechnical Engineer. Depending upon the site, geotechnical recommendations may be necessary for lesser embankment heights;
    - Embankments that impound over 50 acre feet of water or are 10 feet in height or more may need a Dam Construction permit from the Idaho Department of Water Resources.
    - Embankments constructed on native soil shall be consolidated, free of loose surface soil materials, roots, and other organic debris or as recommended by the Geotechnical Engineer;
    - Erosion control shall be provided to stabilize the bank and its overflow during construction;
    - Energy dissipation measures (i.e., riprap pads) shall be installed where pipes discharge onto embankments;
    - Embankment compaction shall produce a dense, low permeability engineered fill that can tolerate post-construction settlements with a minimum of cracking. The embankment fill shall be placed on a stable subgrade and compacted to a minimum of 95 percent of the Modified Proctor Density, ASTM Procedure D1557. Placement moisture content should lie within 1 percent dry to 3 percent wet of the optimum moisture content;

- Seepage protection (e.g. collars) are required on all penetrations within the wetted perimeter;
- Embankment must be constructed by excavating a key. The key width shall equal 50 percent of the berm embankment cross-sectional width, and the key depth shall equal 50 percent of the berm height; and,
- The berm top width shall be a minimum of 4 feet.

Emergency Overflow Spillway

Emergency overflow spillways are used to direct overflows into the downstream conveyance system in the event of total failure or extreme inflows.

Emergency overflow spillways shall be provided for ponds with water depths over 2 feet in height or for ponds located on grades in excess of 5 percent. See Section 6.7.1.1 for design requirements.

Access

Maintenance access roads shall be provided to control and other drainage structures associated with the stormwater facility (i.e. inlet or bypass structures).

In ponds and swales, an access ramp is needed for the removal of sediment with a trackhoe and truck. The ramp should extend to the facility bottom if the bottom area is greater than 1,500 square feet. If the bottom area is less than 1,500 square feet, the ramp may end at an elevation 4 feet above the facility bottom. On large, deep ponds, truck access to the pond bottom via an access ramp is necessary so loading can be done in the pond bottom. On smaller facilities, the truck can remain on the ramp, facility edge, or internal berm for loading. (In most cases, trackhoes will be able to negotiate pond side slopes.)

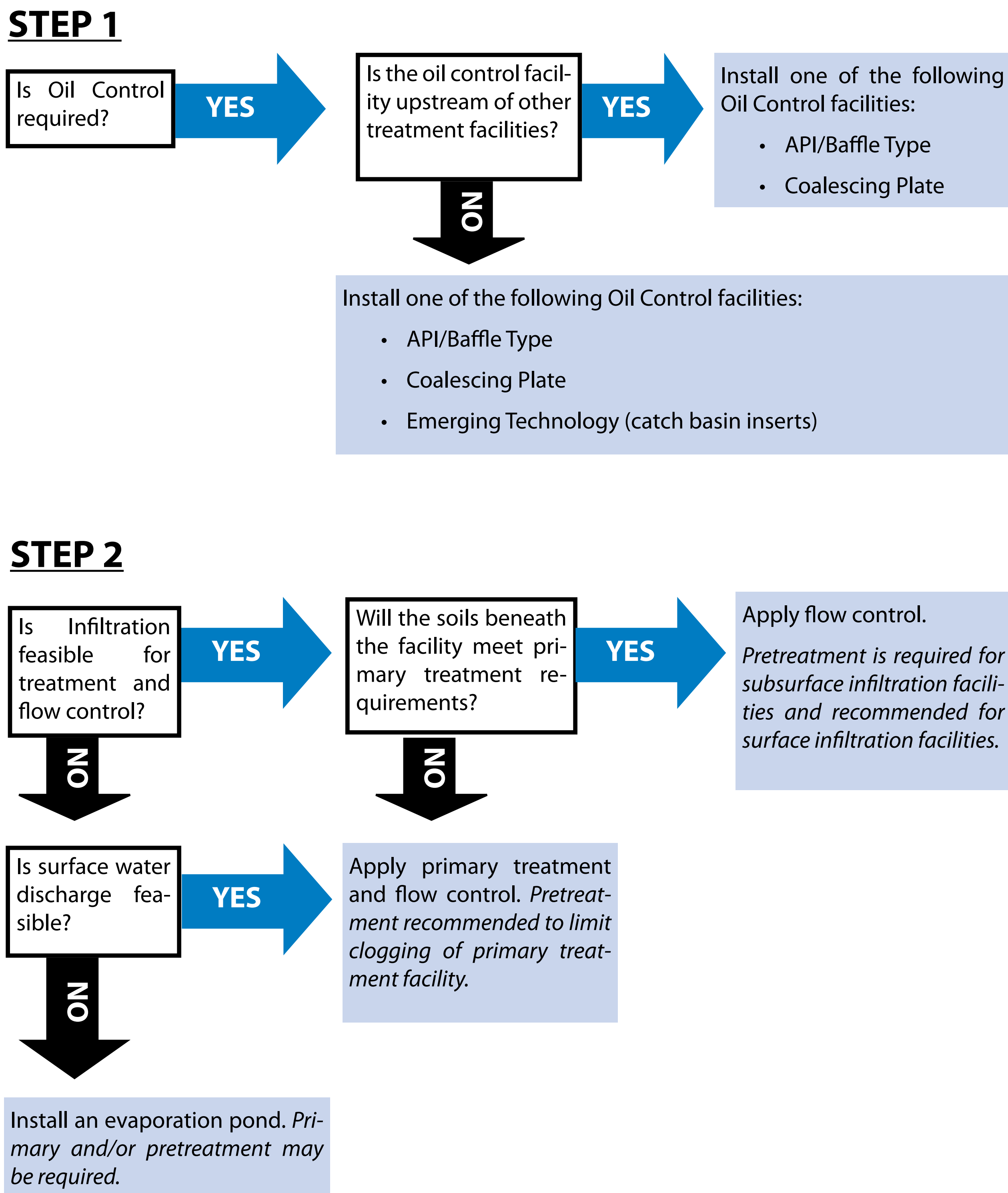
If a fence is required, access should be limited by a double posted gate or by bollards – that is, two fixed bollards on each side of the access road and two removable bollards equally located between the fixed bollards. Access ramps and roads must meet the following design and construction requirements:

- Maximum grade shall be 15 percent;
- Outside turning radius should be a minimum of 50 feet;
- Minimum width should be 15 feet on curves and 12 feet on straight sections; and
- A paved apron must be provided where access roads connect to paved public roadways;

Planting Requirements

Exposed earth on the pond bottom and side slopes shall be vegetated or otherwise stabilized in a timely manner, taking into account the current climate/season. Unless a dryland grass is proposed, irrigation shall be provided. When possible, select native/adapted, drought-tolerant, pest-resistant species to reduce the need for irrigation, fertilizers, and pesticides. Refer to the Portneuf Valley Revegetation Guide for additional design criteria.

FIGURE 6-1: Treatment & Flow Control Facility Flow Chart



depending on soil type (e.g. the expected long-term infiltration rate of local silt soils is ~30% of their demonstrated steady state infiltration rate). The safety factor should be noted in the GSR.

Sizing Criteria

Infiltration facilities shall be sized to fully infiltrate the post-development design storm per Table 2-1. The infiltration rate, safety factors, and size of the infiltrating area are used in conjunction with the storage volume to design the facility.

For facilities in Chubbuck that are not in Special Drainage Areas, no allowance for infiltration is permitted. The ponded depth (or void space for subsurface facilities) must accommodate the design storm.

Depth to Bedrock, Water Table, or Impermeable Layer:

The base of all infiltration basins, trench systems and swales should be ≥ 5 feet above the seasonal highwater mark, bedrock (or hardpan) or other low permeability layer. A minimum separation of 3 feet may be considered if the groundwater mounding analysis, volumetric receptor capacity, and the design of the overflow and/or bypass structures are judged by the professional engineer to be adequate to prevent overtopping and to meet the site suitability criteria specified in this section.

Previously Contaminated or Unstable Soils:

Infiltration facilities shall not be located where they can impact a soil or groundwater contamination site. The design professional should investigate whether the soil under the proposed infiltration facility has contaminants that could be transported from the facility. If so, measures should be taken to remediate the site prior to construction of the facility, or an alternative location should be chosen.

The designer should also determine if the soil beneath the proposed infiltration facility is unstable, due to improper placement of fill, subsurface geologic features, etc. If so, further investigation and planning should be undertaken before siting the facility.

Physical and Chemical Suitability for Treatment:

Surface infiltration facilities and infiltration trenches combine microbial action and soil properties to remove and degrade stormwater pollutants by biological, mechanical filtration, and chemical processes. Soil biological processes include the biodegradation of filtered and adsorbed organic pollutants by micro-organisms and invertebrates. Filtration is the removal of coarse and fine solids by the straining action of soils. The dominant chemical processes include adsorption and cation exchange.

The soil texture and design infiltration rates should be considered along with the physical and chemical characteristics specified below to determine if the soil is adequate for removing the target pollutants. The following soil properties should be carefully considered in making such a determination:

- Cation exchange capacity (CEC) of the treatment soil must be ≥ 5 milliequivalents CEC/100 g dry soil (USEPA Method 9081). Consider empirical testing of soil sorption capacity, if practicable. Ensure that soil CEC is sufficient for expected pollutant loadings, particularly heavy metals. CEC values of >5 meq/100g are expected in northern Bannock County's silty soils according to NRCS soils maps.
- Organic content of the treatment soil must be over 1%. Organic matter can increase the sorptive capacity of the soil for some pollutants.
- Waste fill materials should not be used as infiltration soil media nor should such media be placed over uncontrolled or non-engineered fill soils.

6.6.2 CONSTRUCTION MONITORING

During construction, infiltration ponds and swales also used for sediment control (See Chapter 8) should be excavated to within 1 foot of the final elevation of the pond bottom. Prevent compaction of soils in and around infiltration facilities by limiting heavy equipment operation in the area. While the construction site is still active, limit sediment entering the facility by first conveying runoff through a pre-settling basin, filter bag, or other sediment collection device.

Any accumulation of silt in the infiltration facility must be removed during final stabilization of the site. Excavate infiltration trenches and ponds to final grade only after construction has been completed and all upgradient soil has been stabilized.

Construction Inspections

In order to reduce the potential for compaction, construction equipment and vehicles shall be kept off the pond/swale bottom.

An infiltration test (per the requirements of Chapter 3) demonstrating the facility's conformance to the infiltrative rate criteria is required prior to construction certification if any of the following conditions apply:

- Construction equipment/vehicles were placed on/drove on the swale/pond area.
- During construction the swale/pond area was not protected from sediment runoff.
- When the material used to backfill the swale does not match the design infiltration rate.
- The facility must have vegetation established prior to passing final inspection. In addition, if during final inspection, it is found that the constructed infiltration facility does not conform to the accepted design, the system shall be reconstructed so that it does comply or another facility may need to be added.

6.6.3 INFILTRATION BASINS

Infiltration basins are earthen impoundments used for the collection, temporary storage, and infiltration of incoming stormwater runoff. With appropriate soil, infiltration basins can provide primary treatment. Infiltration basins must meet the requirements of Section 6.4.1 Site Suitability for Infiltration Facilities, as well as the requirements listed below.

Design Criteria

When not preceded by a primary treatment facility, infiltration swales/ponds shall be divided into two cells (a fore-bay and an infiltration cell), separated by a baffle or berm. The fore-bay shall be designed to contain the water quality design storm plus 15% for sediment storage. It is recommended that the forebay be preceded by a control structure to divert the water quality design storm flow to the forebay, with a high flow bypass going directly to the main pond. Refer to Figure 6.5 for a typical infiltration basin configuration.

The slope of the bottom of an infiltration pond shall be flat.

A minimum of one foot of freeboard is recommended when establishing the design ponded water depth. Special attention to freeboard should be taken into consideration for basins built with embankment to help protect downstream properties from flooding and to provide public safety for emergency situations. Freeboard is measured from the rim (top of basin) of the infiltration facility to the maximum ponding level or from the rim (top of basin) down to the overflow point if overflow or a spillway is included. The engineer of record shall use their professional judgment when determining an appropriate freeboard.

When overflow is provided by a drywell, the pond berm elevation shall be located a minimum of 6 inches above the drywell rim. Overflow drywells must be located adjacent to the pond berm (not at the low point of the facility) to reduce the likelihood of short circuiting, leakage, or erosion around the drywell barrel.

These facilities are best used for smaller drainage basins 10 acres or less.

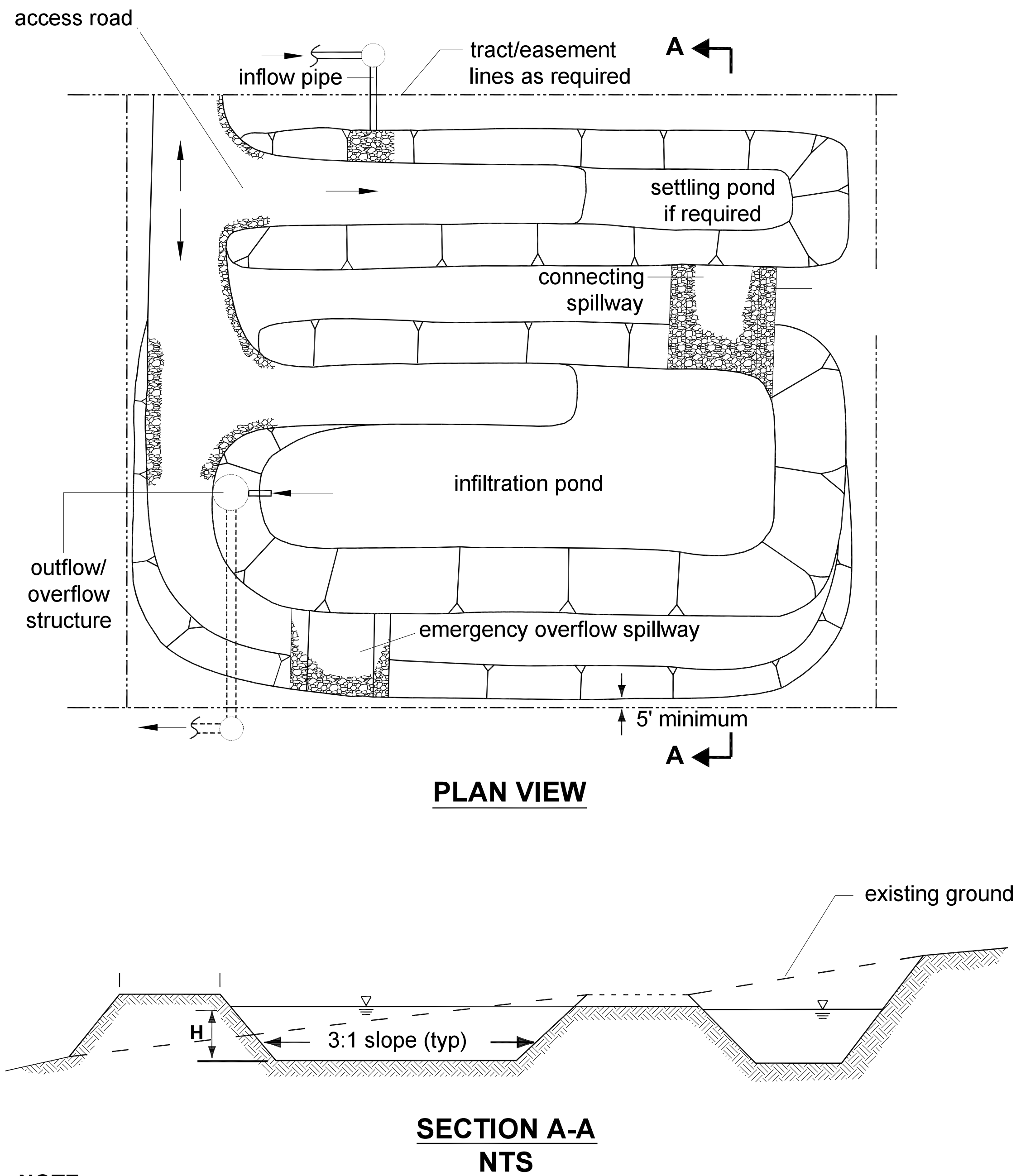
Location

If approved by the local jurisdiction, the distance between the bottom of the basin and the seasonally high groundwater level may be reduced to 3 feet if the infiltration rate safety factor is increased by 0.2 for each foot of separation less than 5 feet. In no case should the pond bottom be less than 3 feet from the seasonal high groundwater level or impermeable soil layer.

Pretreatment & Soil Criteria

Pretreatment prior to the forebay to remove additional sediment and floatables may be desired. Primary treatment is required prior to the facility if the forebay does not meet the soil criteria in Table 6.2.

Figure 6-5: Typical Infiltration Basin



NOTE:
Detail is a schematic representation only. Actual configuration will vary depending on specific site constraints and applicable design criteria.
See Section 6.4.6 for design criteria.

Souce: King County Surface Water Manual (2009)

Table 6-3: Design Criteria for Drywells, Infiltration Galleries/Chambers and other Subsurface Infiltration Facilities (UICs).[1]

Requirements				
Treatment Capacity of Soil beneath UIC facility.	HIGH TREATMENT CAPACITY	MODERATE TREATMENT CAPACITY	LOW TREATMENT CAPACITY	NO TREATMENT CAPACITY
Required soil depth to groundwater/ bedrock.	>5-ft;	>10-ft;	>25-ft;	N/A
Required Soil Type	Materials with median grain size < 0.125 mm. Having a sand to silt/clay ratio of less than 1:1 and sand plus gravel < 50%. Soil types include: Lean, fat, or elastic clay; Sandy or silty clay; Silt; Clayey or sandy silt; Sandy loam or loamy sand; Silt/clay with inter-bedded sand; Well-compacted, poorly-sorted materials. This category generally includes loess, till, hardpan, and caliche.	Materials with median grain size 0.125mm to 4mm. Sand to silt/clay ratio from 1:1 to 9:1 and % sand > % gravel. Soil types include: Fine, medium or coarse sand; Sand with interbedded clay and/or silt; and Poorly-compacted, poorly-sorted materials. This category includes some alluvium and outwash deposits.	Materials with median grain size > 4mm to 64mm. Having a sand to silt/clay ratio greater than 9:1 and % sand less than % gravel. Soil types include: Poorly-sorted, silty or muddy gravel; Sandy gravel, gravelly sand, or sand and gravel. This category includes some alluvium and outwash deposits.	Materials with median grain size > 4mm to 64mm. Having a sand to silt/clay ratio greater than 9:1 and % sand less than % gravel. Soil types include: Poorly-sorted, silty or muddy gravel; Sandy gravel, gravelly sand, or sand and gravel. This category includes some alluvium and outwash deposits.
Treatment Required?[2,3,4]	All sites: Pretreatment.	Low – moderate use sites: Pretreatment; High use sites: Primary Treatment.	Low use sites: Pretreatment; High- moderate use sites: Primary Treatment.	All sites: Primary Treatment.

[1] Not to be used for Infiltration Trenches. See Section 6.6.1 (Table 6-2).

[2] This table does not address oil control requirements. See Section 2.3.6.

[3] See Section 6.3 for definitions of high, moderate and low-use sites.

[4] See Table 6-1 for appropriate pre-treatment and treatment facilities; Primary treatment facilities that are an emerging technology and placed prior to UICs shall be approved by the Washington Department of Ecology TAPE program for 'Basic Treatment' or 'Pretreatment' at the Conditional or General Use Level.

6.7 DETENTION FACILITIES

A detention system is a low lying area that is designed to temporarily hold and slowly release stormwater. A detention facility is intended to control peak stormwater runoff rates, and as designed per the criteria in this Chapter, does not control volume (except to infiltrate the water quality design storm and the volume of the bottom 6" of the facility).

Stormwater runoff from a developed site shall leave the site in the same manner and location as it did in the pre-developed condition. Therefore, a detention system may be used only when a well-defined natural drainage course is present prior to development.

Detention facilities are designed to limit the release rate. The analysis of multiple design storms is needed to control and attenuate both low and high flow storm events. The total post-developed discharge rate (including bypass flow) shall be limited to the rates outlined in Table 2.1. The NRCS Type II 24-hour storm event is the design storm for all detention facilities.

6.7.1 DETENTION PONDS

Sizing

Detention ponds must be designed as flow-through systems. Developed flows must enter through a conveyance system separate from the control structure and outflow conveyance system. Maximizing the distance between the inlet and outlet is encouraged to promote plug flow and allow sediment to settle to the bottom of the pond.

Design Criteria

Detention ponds shall comply with the facility requirements in Section 6.3.

Pond bottoms should be level and located a minimum of 12" below the inlet and outlet to provide sediment storage.

Figures 6-13 and 6-14 are illustrations of this BMP. Control structures and overflows shall be designed following the guidelines in Section 6.7.1.1.

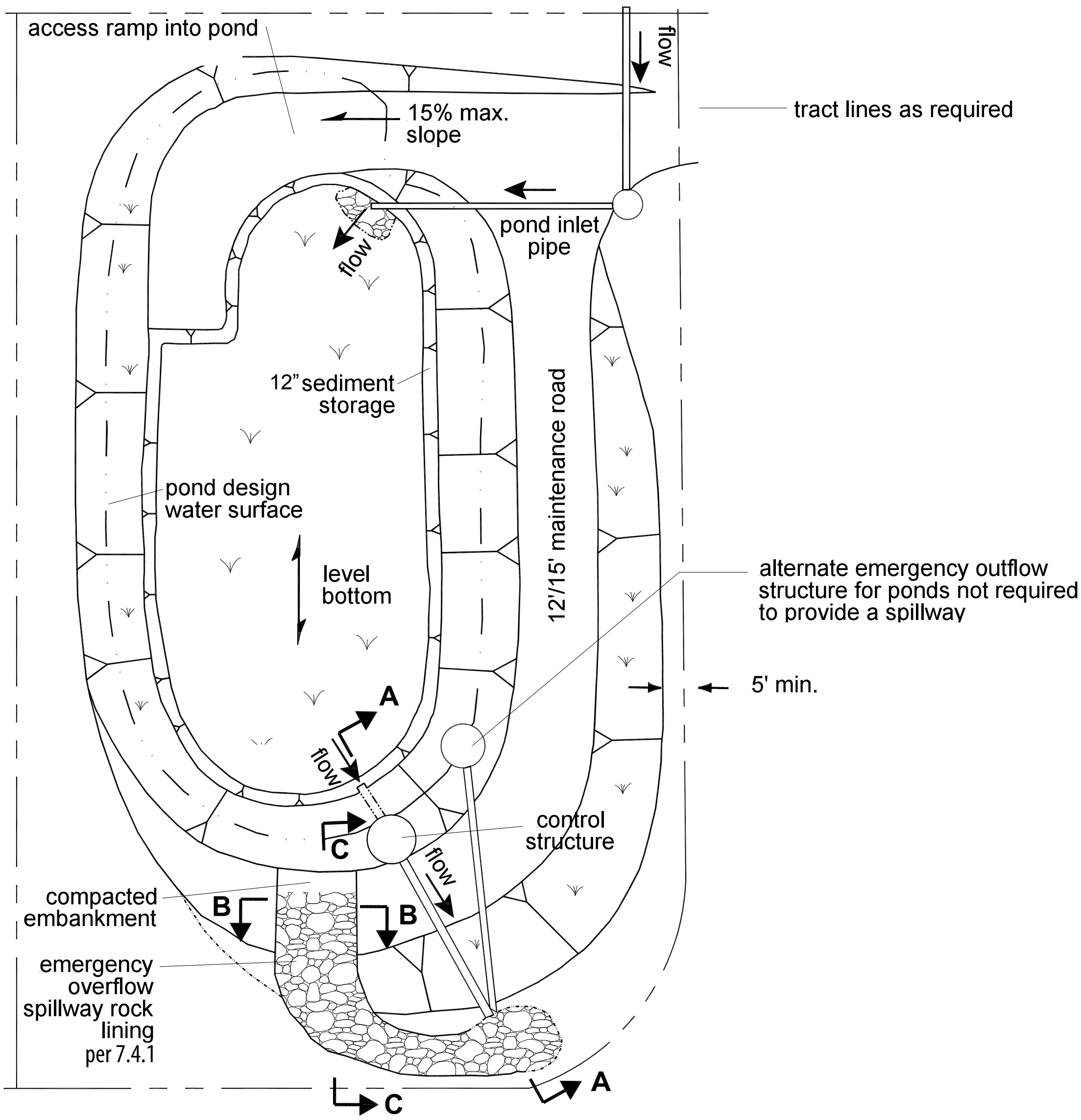
When not preceded by a primary treatment facility, detention ponds shall be divided into two cells (a forebay and detention cell), separated by a baffle or berm. The fore-bay shall be sized to contain the water quality treatment design storm plus 15% sediment storage. It is recommended that the forebay be preceded by a control structure to divert the water quality design storm flow to the forebay, with a high flow bypass going directly to the main pond.

Soil Criteria & Pretreatment

Pretreatment prior to the forebay to remove sediment may be desired.

Since detention systems infiltrate the bottom 12" of water in their ponds, they shall adhere to the subgrade infiltrative criteria specified for infiltration basins. Primary treatment is required prior to the facility if the forebay does not meet the soil criteria in Table 6.2.

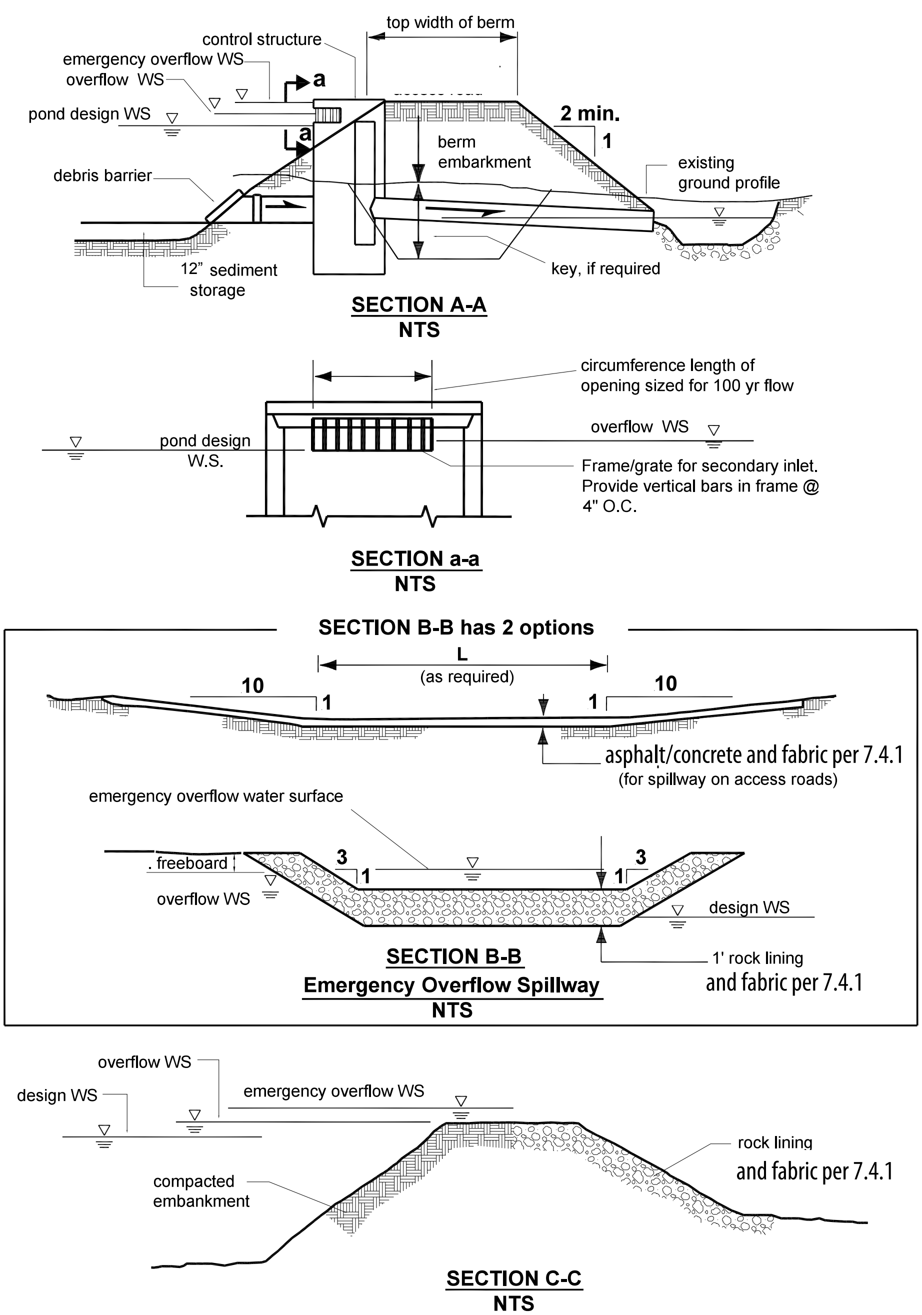
Figure 6-13 – Typical Detention Pond (*forebay not shown)*



NOTE:
This detail is a schematic representation only. Actual configuration will vary depending on specific site constraints and applicable design criteria. See Section 6.7.1 for design criteria.

Source: King County Surface Water Design Manual

Figure 6-14 – Typical Detention Pond Sections



NOTE: This detail is a schematic representation only. Actual configuration will vary depending on specific site constraints and applicable design criteria. See Section 6.7.1 for design criteria.

Source: King County Surface Water Design Manual

6.7.1.1 OUTFLOW STRUCTURE DESIGN

Outflow structures are designed to control the release rate of flow from a detention facility to meet the required performance standard. Outflow structures should include a control structure, primary overflow, and emergency overflow.

Control Structures

Control structures include weirs, culverts, or catch basins or manholes with a restrictor device for controlling outflow from a detention facility. Riser type restrictor devices ("tees") or flow restrictor oil pollution control tees ("FROPTs") also provide some incidental oil/water separation to temporarily detain oil or other floatable pollutants. The restrictor device usually consists of multiple orifices and/or weir sections sized to meet the performance standards.

Control structures shall be selected taking into consideration the expected hydraulic heads. Table 6-2 presents typical control structures and their applicability:

TABLE 6-4: OPTIMAL APPLICATION OF CONTROL STRUCTURES

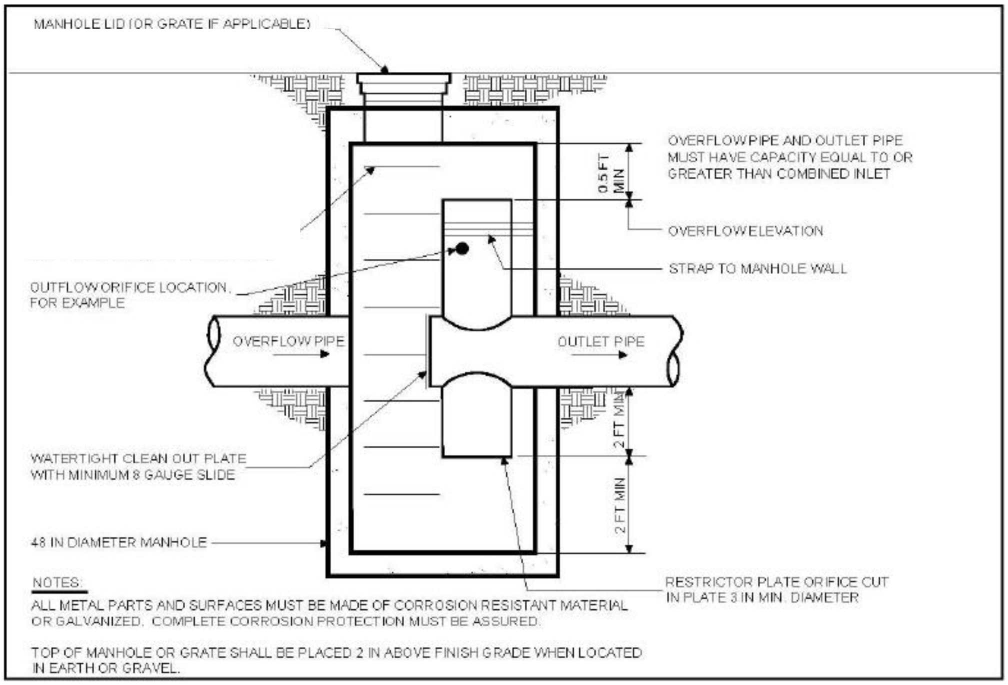
CONTROL STRUCTURE	POND HEAD
Outlet Pipe	Very Low
V-Notch Weir	Low
Slotted Weir	Moderate
Multi-Stage Orifice	High

Outflow control structures shall meet the following requirements:

- Circular orifices shall be at least 1.0 inch in diameter. Slotted weirs can be used in lieu of smaller orifices to reduce the occurrence of plugging. Note: The live storage depth need not be reduced to less than 3 feet to meet the performance standards;
- Orifices may be constructed on a tee section as shown in Figure 6-15, or on a baffle (internal wall) within the catch basin or manhole;
- In some cases, performance requirements may require the top orifice/elbow to be located too high on the riser to be physically constructed (e.g., a 13-inch diameter orifice positioned 0.5 feet from the top of the riser). In these cases, a notch weir in the riser pipe may be utilized to meet the performance requirements;
- A flow control structure utilizing a riser arrangement, shall conform to Figure 6-15 and shall include, as a minimum, a watertight cleanout, 200-lb test chains, an overflow riser with a 12-inch diameter or larger riser, and sump. At least three riser straps spaced no more than 2 feet on center shall be provided;

- Outlet structures shall be located where accessible for maintenance, preferably adjacent to access roads; and
- Manhole lids shall be locking/bolt down and the top of the manhole shall be placed 2 inches above the finished grade for detention ponds, unless the structure is placed in a parking lot, road, or shoulder.

Figure 6-15 – Typical Flow Control Structure



Primary Overflow

A primary overflow shall be provided to bypass the 100-year developed peak flow over or around the restrictor system. The primary overflow is intended to protect against breaching of a pond embankment in the event of plugged orifices or high flows.

An open topped riser can often serve as the primary overflow, provided the riser, inlet pipe, and outflow pipe both have capacity to carry the 100-year developed peak flow. The combined orifice and riser (or weir) overflow may be used to meet the performance requirements. However, the design must still provide primary overflow for the developed 100-year peak flow assuming all orifices are plugged.

In ponds, a secondary inlet to the control structure shall also be provided as additional protection against overtopping should the inlet pipe to the control structure become plugged. A grated opening (“jailhouse window”) in the control structure manhole functions as a weir (see Figure 6-14) when used as a secondary inlet. Note: the maximum circumferential length of this opening shall not exceed one-half of the control structure circumference. The “birdcage” overflow structure shown in Figure 6-16 may also be used as a secondary inlet.

- Complete channel calculations that state the design peak flow rates and design information, such as channel shape, slope, Manning's coefficient (Table 4-5).
- Calculations, including the velocity, capacity, and Froude number shall be provided for each distinct channel segment whenever the geometry of the channel changes (i.e. if the slope, shape or roughness change significantly);
- The centerline and direction of flow for all constructed drainage ditches or natural channels located within the project limits are to be clearly shown in the construction plans and basin map. For all proposed channels, locating information shall be provided at all angle points; and,
- Calculations shall support the riprap area, thickness, riprap size and gradation, filter blanket reinforcement for all channel protection, which shall be provided when permissible velocities are exceeded (see Table 7-1). This information shall be included in the plans;

TABLE 7-1 PERMISSIBLE CHANNEL VELOCITIES

Soil/Type of lining (Earth; No vegetation)	**Maximum Permissible Velocities (ft/sec)**		
	Clear Water	**Water Carrying Fine Silts**	**Water Carrying Sand & Gravel**
Fine Sand (non-colloidal)	1.5	2.5	1.5
Sandy Loam (non-colloidal)	1.7	2.5	2.0
Silt Loam (non-colloidal)	2.0	3.0	2.0
Ordinary Firm Loam	2.5	3.5	2.2
Volcanic Ash	2.5	3.5	2.0
Stiff Clay (very colloidal)	3.7	5.0	3.0
Graded, Loam to Cobbles (non-colloidal)	3.7	5.0	5.0
Graded, Silt to Cobbles (colloidal)	4.0	5.5	5.0
Alluvial Silts (non-colloidal)	2.0	3.5	2.0
Alluvial Silts (colloidal)	3.7	5.0	3.0
Course Gravel (non-colloidal)	4.0	6.0	6.5
Cobbles and Shingles	5.0	5.5	6.5
Shales and Hard Pans	6.0	6.0	5.0

Source: Special Committee on Irrigation Research, American Society of Civil Engineers, 1926. Based on uniform flow in continuously wet, aged channels with erodible linings.

The Froude number shall be checked near the beginning and near the end of a channel that has significantly different grade changes to determine if a hydraulic jump occurs (Froude number changes from <1 to >1, or vice versa). Since it is difficult to correlate the location of a hydraulic jump to the actual location in the field, the Engineer shall propose evenly spaced riprap berms, checkdams, or other protective measures to ensure that the jump does not erode the conveyance facility.

- When geosynthetics are used for channel stabilization, the plans shall clearly specify fabric type, placement, and anchoring requirements. Installation shall be per the manufacturer's recommendation; and,
- Plans for grass-lined channels shall specify seed mixture and irrigation, as applicable.

Slope

Minimum grades for constructed channels shall be the following:

- 1⁄2 percent (0.005 feet/feet) for cement concrete, graded earth or close-cropped grass; and
- 1 percent (0.010 feet/feet) for asphalt concrete or rip rap lined channels.

Note: Non-structured alternatives are preferred over asphalt and concrete channels whenever possible.

Side Slopes

Ditches may be "V" shaped or trapezoidal. However, V-ditches are not recommended in easily erodible soils or where problems establishing vegetation can be anticipated.

The side slope of roadside ditches shall conform to the requirements for clear zone of the local jurisdiction and/or ITD design standards.

Ditches or channels shall not have side slopes that exceed the natural angle of repose for a given material or per Table 7-2:

TABLE 7-2 MAXIMUM DITCH OR CHANNEL SIDE SLOPES

TYPE OF CHANNEL	SIDE SLOPE HORIZONTAL: VERTICAL
Firm Rock	Vertical to 1⁄4:1
Concrete-Lined Stiff Clay	1⁄2:1
Fissured Rock	1⁄2:1
Firm Earth With Stone Lining	1 1⁄2:1
Firm Earth, Large Channels	1 1⁄2:1
Firm Earth, Small Channels	2:1
Loose, Sandy Earth	2:1
Sandy, Porous Loam	3:1

Source: Civil Engineering Reference Manual, 8th Edition

Location

Constructed channels cannot be placed within or between residential lots. Ditches and channels shall be located within a drainage tract or within a border easement. Large lot subdivisions (lots ≥ 1 acre) may be allowed to have ditches or channels traverse through the lot(s) and consideration may be given as to placement within an easement versus a tract. The local jurisdiction will review these proposals on a case by case basis.

Depth

See Table 2-2 for minimum depth requirements for constructed channels.

Velocity

Table 7-1 lists the maximum permissible mean channel velocities for various types of soil and ground cover. If mean channel velocities exceed these values during the design flow, channel protection is required. In addition, the following criteria shall apply:

- Where only sparse vegetative cover can be established or maintained, velocities should not exceed 3 feet/second;
- Where medium density vegetation can be established by seeding, velocities in the range of 3 to 4 feet/second are permitted;
- Where dense sod can be developed quickly or where the normal flow in the channel can be diverted until a vegetative cover is established, velocities of 4 to 5 feet/second are permitted; and,
- On well-established sod of good quality, velocities in the range of 5 to 6 feet/second are permitted.

Channel Capacity

Open channels shall be sized using the following variation of Manning's formula:

$$Q = VA = \frac{1.486 A R^{2/3} S^{1/2}}{n}$$

Where:	Q=	rate of flow (cfs);
	V =	mean velocity in channel (ft/s);
	A =	cross-sectional area of flow in the channel (ft2);
	n =	Manning's coefficient (See Table 5-4);
	S =	channel slope (ft/ft); and,
	R =	hydraulic radius (ft) = Area/wetted perimeter

Note: The Manning equation will give a reliable estimate of velocity only if the discharge, channel cross section, roughness, and slope are constant over a sufficient distance to establish uniform flow conditions. Strictly speaking, uniform flow conditions seldom, if ever, occur in nature because channel sections change from point to point. For practical purposes, however, the Manning equation can be applied to most open channel flow problems by making judicious assumptions.

Energy Dissipation Design

An energy dissipater is useful in reducing excess velocity, as a means of preventing erosion below an outfall or spillway. Common types of energy dissipaters for small hydraulic works are: hydraulic jumps, stilling wells, riprap outfall pads, outlet aprons, and gabion weirs. Document that any hydraulic jump is located where it will not cause damage.

Channel Protection

Channel velocities shall be analyzed periodically throughout the channelized route, particularly at the following locations:

- At the first point that the stormwater runoff becomes concentrated into a natural or constructed channel;
- At all changes in channel configuration (grade, sideslopes, depth, shape, etc.); if an erosive velocity is determined at a change in channel configuration, the velocity shall be evaluated up the chanel until the point at which the velocity is determined not to be erosive; and,
- At periodic locations along the entire channelized route.

A material shall be selected that has revetment and armoring capabilities, and the channel shall be analyzed using the Manning's "n" value for that material to determine if the material will reduce the velocity in the channel. In some cases, vegetative cover (natural grasses, etc.) may provide excellent protection without changing the flow characteristics and should be evaluated. If the calculations reveal that common materials such as matting or riprap are not adequate, stronger protection such as gabions and/or stilling pools may be necessary.

Riprap Protection at Outlets

If the flow velocity at a channel or culvert outlet exceeds the maximum permissible velocity for the soil or channel lining, channel protection is required. The protection usually consists of a reach, such between the outlet and the stable downstream channel lined with an erosion-resistent material such as riprap.

The ability of riprap revetment to resist erosion is related to size, shape and weight of the stones. Most riprap lined channels require either a gravel filter blanket or filter fabric under the riprap.

Riprap material shall be blocky in shape rather than elongated. The riprap stone shall have sharp, angular, clean edges. Riprap stone shall be reasonably well-graded.

Apron Dimensions: The length of an apron (L_a) as shown in Figure 7-1, is determined using the following empirical relationships that were developed for the U.S. Environmental Protection Agency (1976):

Prepared By: ______________________________ Contact Phone No.: ______-______-_________

for official use only

Date: ______________

Expected Construction Completion Date: ______________

Permit No.: _________

BY SIGNING BELOW: *Applicant accepts and agrees to the terms and conditions contained in the O&M Manual and in any document executed by filer and recorded with it.* **TO BE SIGNED IN THE PRESENCE OF A NOTARY.**

Applicant Signature ____

Applicant Signature ____

INDIVIDUAL Acknowledgement

STATE of IDAHO, Bannock County;

This instrument was acknowledged before me on _____/_____/______

By: ____

Notary Signature: ____

My Commission Expires: ____________________ *for notary seal*

CORPORATE Acknowledgement:

STATE of IDAHO, Bannock County;

This instrument was acknowledged before me on _____/_____/______

By: ____

As (Title): ____

Of (Corporation): ____

Notary Signature: ____

My Commission Expires: ____________________ *for notary seal*

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