Appendix B – Equations and Examples

Purpose

The purpose of this appendix is to provide background equations and example problems for clarity of calculations used throughout the manual. This appendix is divided into sections by referenced chapters in the manual. Topics are differentiated by arrows.

Chapter 2 – Runoff

- **Depth-Duration-Frequency**

  The depth-duration-frequency (DDF) data from the NOAA Atlas Volume 3 is summarized in **Table 1**.

Table 1 – Point Rainfall Depth-Duration-Frequency *in* Aspen, Colorado

Period	*5-min*	*10-min*	*15-min*	*30-min*	***1-hr (P1)***	*2-hr*	*3-hr*	*6-hr*	*24-hr*
2-yr	0.18	0.29	0.36	0.50	**0.64**	0.75	0.83	0.98	1.40
5-yr	0.29	0.45	0.57	0.79	**1.00**	1.10	1.17	1.30	1.80
10-yr	0.35	0.54	0.68	0.95	**1.20**	1.30	1.37	1.50	2.00
25-yr	0.41	0.63	0.80	1.11	**1.40**	1.54	1.63	1.80	2.40
50-yr	0.46	0.72	0.91	1.26	**1.60**	1.74	1.83	2.00	2.70
100-yr	0.49	0.76	0.96	1.34	**1.69**	1.87	1.98	2.20	3.05
Notes: **1. Read Volume III for 6-hr and 24-hr rainfall depths**									

  Based on the depth and duration data in Table 1, rainfall intensities can be calculated for various frequencies. Rainfall intensity data forms the basis of the Intensity-Duration-Frequency (IDF) curves in Figure 2.1 of Chapter 2.

- **Depth Ratios**

  The recommended rainfall distributions, based generally on the Denver design rainfall distribution depth ratios with minor adjustments for Aspen, are provided in **Table 2.** The incremental rainfall depth rations in **Table 2** have been verified to provide reasonable agreement to Aspen's IDF formula **(Equation 2-1)** and are generally consistent with the NOAA Atlas 2 (NOAA 1973). Depth ratios (or percentages) are input parameters for CUHP models. Chapter 2, Tables 2.5 and 2.6, are depths derived using **Table 2** for the 1-hr event in the City of Aspen. For areas outside of the City of Aspen, the percentages in **Table 2** should be used in CUHP to derive depths for those areas.

Table 2 – Incremental Rainfall Depth Ratios for Aspen (Applicable to area <10 sq miles)

Design Rainfall Distributions $P(t)/P_1$ in percent					
Time *minutes*	2-yr	5-yr	10-yr	25/50-yr	100/500-yr
0	0.0	0.0	0.0	0.0	0.0
5	2.0	2.0	2.0	1.3	1.0
10	4.0	3.7	3.7	3.5	3.0
15	8.4	8.7	8.2	5.0	4.6
20	16.0	15.3	15.0	8.0	8.0
25	25.0	25.0	25.0	15.0	14.0
30	14.00	13.0	12.0	25.0	25.0
35	6.3	5.8	5.6	12.0	14.0
40	5.0	4.4	4.3	8.0	8.0
45	3.0	3.6	3.8	5.0	6.2
50	3.0	3.6	3.2	5.0	5.0
55	3.0	3.0	3.2	3.2	4.0
60	3.0	3.0	3.2	3.2	4.0
65	3.0	3.0	3.2	3.2	4.0
70	2.0	3.0	3.2	2.4	2.0
75	2.0	2.5	3.2	2.4	2.0
80	2.0	2.2	2.5	1.8	1.2
85	2.0	2.2	1.9	1.8	1.2
90	2.0	2.2	1.9	1.4	1.2
95	2.0	2.2	1.9	1.4	1.2
100	2.0	1.5	1.9	1.4	1.2
105	2.0	1.5	1.9	1.4	1.2
110	2.0	1.5	1.9	1.4	1.2
115	1.0	1.5	1.7	1.4	1.2
120	1.0	1.3	1.3	1.4	1.2

- **Extreme Rainfall Events**

In addition to evaluation of precipitation for events ranging from the 2- to 100-year events, there may be instances in Aspen when larger events may need to be considered, for example for an impoundment with significant development downstream. In such cases, it may be necessary to evaluate extreme precipitation or the Probable Maximum Precipitation (PMP) event. Two methods are currently used in Colorado:

1. Calculation in accordance with Hydrometeorlogical Report Number 49 (HMR 49) (Hansen et al. 1977). This document was developed for the Colorado River and Great Basin drainage areas.

2. Extreme Precipitation Analysis Tool (EPAT). This is a GIS based methodology developed for the State Engineers Office (SEO).

Either method is acceptable in Aspen when extreme precipitation event analysis is required. Engineers should consult with the Division Engineer to determine if the SEO has a preference prior to conducting analysis. As of the date of this Chapter, both methods are still in use, but the EPAT method is being used more frequently and HMR 49 less frequently.

Chapter 3 – Rainfall

➢ Soil Types

Table 3.2 Soil Types in the Aspen Area

Type A Soils	Type B Soils	Type C Soils	Type D Soils
	Almy	Acree	Ansari
	Ansel	Arle	Camborthids
	Antrobus	Callings	Dollard
	Anvik	Cochetopa	Earsman
	Atencio	Cushool	Fluvaquents
	Azeltine	Fughes	Gypsiorthids
	Brownsto	Gothic	Iyers
	Charcol	Gypsum land	Kilgore
	Coulterg	Irrawaddy	Moyerson
	Curecanti	Jerry	Rentsac
	Dahlquist	Kobar	Rock outcrop
	Dotsero	Kobar, dry	Rock outcrop, shale
	Empedrado	Miracle	Rogert
	Etoe	Moen	Starley
	Evanston	Mord	Starman
	Forelle	Redrob	Tanna
	Forsey	Showalter	Torriorthents
	Goslin	Sligting	
	Grotte	Woodhall	
	Ipson	Woosley	
	Millerlake		
	Mine		
	Monad		
	Morval		
	Mussel		
	Pinelli		
	Skylick		
	Southace		
	Tridell		
	Uracca, moist		
	Vandamore		
	Yamo		
	Yeljack		
	Youga		
	Zillman		

➢ Infiltration Rates

An infiltration rate reflects the ability of the soil medium to absorb water. This parameter is usually given in inch per hour or millimeter per hour. Infiltration rates are described by a decay function with a high rate at the beginning of the event when the soil is dry, and a low rate when the soil becomes saturated.

Table 3 Infiltration Rates for Different Soil Groups (UDFCD 2001)

Soil Type	Initial Rate Inch/hr	Final Rate Inch/hr	Decay Coefficient 1/sec for CUHP	Decay Coefficient 1/hr for SWMM
A	5.0	1.0	0.0007	2.52
B	4.5	0.6	0.0018	6.48
C	3.0	0.5	0.0018	6.48
D	3.0	0.5	0.0018	6.48

Table 3 is recommended for design infiltration rates under the average soil antecedent moisture condition. When the watershed has several different types of soils, the representative infiltration rate can be determined as the area-weighted value.

$$f(t) = f_c + (f_o - f_c)e^{-kt}$$ **(Equation 3-1)**

in which,

$f(t)$ = infiltration rate at elapsed time t (in/hr),

f_o = initial infiltration rate (in/hr),

fc = final infiltration rate (in/hr),

e = natural logarithm base**,** and

k = decay coefficient (1/sec or 1/hr).

Chapter 4 – Street Drainage System Design

- ### Example Calculation of Allowable Street Hydraulic Capacity for a Collector Street

A collector street in the City of Aspen has a half-width of 29 feet, including the traffic lane of 11 feet and a parking width of 18 feet. The street cross section in **Figure 4.4** has n = 0.016, W = 2.5 feet, D_s = 2 inches, S_o = 3.0 %, and S_x = 2%. The curb height, H_c, for this street is 6 inches. D_m = 6 inches for minor or 12 inches for a major event To reserve the middle width of 10 feet in one traffic direction, the allowable water spread is reduced to 19 feet for this street.

Solution

According to **Table 4.2**, a collector street shall be designed not to overtop the curb height under a minor storm. Thus, the gutter-full capacity for this street is defined by setting the gutter flow depth equal to the curb height of 6 inches. For this case, $D = D_m = H_c$ = 6.0 inches.

The cross slope across the gutter width is calculated as:

$$S_w = S_x + \frac{D_s}{W} = 0.02 + \frac{2}{12 \times 2.5} = 0.087 \text{ ft/ft,}$$

The gutter-full water spread flow is calculated as:

$$T = \frac{(D_m - D_s)/12}{S_x} = \frac{(6-2)/12}{0.02} = 16.7 \text{ feet}$$

$$T_x = T - W = 16.7 - 2.5 = 14.2 \text{ feet}$$

$$T_s = \frac{D_m}{S_w} = \frac{6}{12 \times 0.087} = 5.74 \text{ feet}$$

$$Q_w = \frac{0.56}{0.016}(0.087)^{1.67}[5.74^{2.67} - (5.74 - 2.5)^{2.67}]\sqrt{0.030} = 8.58 \text{ cfs}$$

$$Q_x = \frac{0.56}{0.016} 0.02^{1.67} 14.2^{2.67} \sqrt{0.03} = 10.45 \text{ cfs}$$

$$Q_g = Q_w + Q_x = 8.58 + 10.45 = 19.0 \text{ cfs}$$

The available water spread on this street is set to be 19 feet. The spread width capacity is calculated as:

$$T_x = T - W = 19.0 - 2.5 = 16.5 \text{ feet}$$

$$T = \frac{(D_m - D_s)/12}{S_x} = \frac{(D_m - 2)/12}{0.02} = 15.0 \text{ feet. So, } D_m = 5.6 \text{ inches}$$

$$T_s = \frac{D_m}{S_w} = \frac{5.6}{12 \times 0.087} = 5.77 \text{ feet}$$

$$Q_w = \frac{0.56}{0.016}(0.087)^{1.67}[5.77^{2.67} - (5.77 - 2.5)^{2.67}]\sqrt{0.030} = 10.32 \text{ cfs}$$

$$Q_x = \frac{0.56}{0.016} 0.02^{1.67} 16.5^{2.67} \sqrt{0.03} = 15.7 \text{ cfs}$$

$$Q_m = Q_w + Q_x = 10.32 + 15.7 = 26.0$$

From **Figure 4.5**, the reduction factor for S_o= 3% is 0.75 for a minor storm. The allowable street hydraulic capacity is determined as:

$$Q_a = \min(R \times Q_g, Q_m) = \min(0.75 \times 19.0,\ \ 26.0) = 14.3 \text{ cfs for minor event.}$$

For this case, the allowable street capacity is determined to be 14.3 cfs for a minor event.

Solution for a Major Storm

According to **Table 4.2**, the water depth in a collector gutter can be 12 inches during a major storm event. To calculate the allowable street hydraulic capacity, repeat the above process. The gutter-full capacity is determined to be 44 cfs for a major storm event. The reduction factor in **Figure 4.5** for a major storm is 0.6 for S_0=3%. The allowable street hydraulic capacity is determined as

$$Q_a = \min(R \times Q_g, Q_m) = \min(0.60 \times 44.0,\ \ 26.0) = 26.0 \text{ cfs for major event.}$$

- **Example for Street Design Flow**

Use the Rational method to find the 10-year local design flow to be 10.5 cfs. With a carryover flow of 1.2 cfs (not captured from the upstream inlet), the design flow is calculated as:

$$Q_s = 10.5 + 1.2 = 11.7 \text{ cfs}$$

It takes an iterative process to analyze the design flow in the street section that is described in Section 7.3. For this case, the design flow condition is determined to be: T=13.5 ft, D=0.44 ft, V=6.94 fps, Q_w=6.43 cfs and Q_x=5.32 cfs.

➢ Example for on-Grade Grate

Referring to the Example for Design Flow above, the design flow on the street has: T=13.5 ft, D=0.44 ft, V=6.94 fps, Q_s = 11.7 cfs, Q_w=6.43 cfs and Q_x=5.32 cfs. A typical bar grate has a unit width, W_o, of 1.50 feet and a unit length, L_o, of 2.50 feet. Determine the number of inlet grates in Figure 4.10.a in order to intercept more than 75% of the design flow of 11.7 cfs.

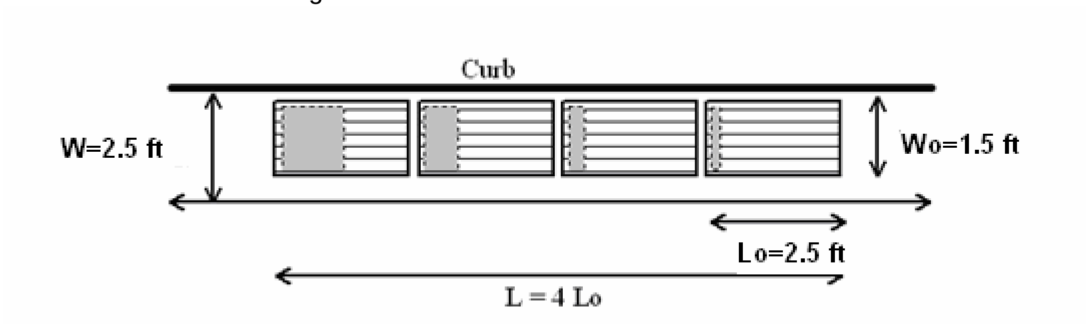

Figure 4.10.a Design Example for On-Grade Grate Inlet

Consider four grates. The total grate length is:

$$L_g = nL_o = 4.0 \times 2.5 = 10.0 \text{ ft}$$

From **Table 4.5**, the clogging factor is 0.23. The effective grate length free from clogging is:

$$L_e = (1-0.23) \times 10.0 = 7.7 \, ft$$

$$R_x = \frac{1}{\left(1 + \frac{0.15V^{1.8}}{S_x L_e^{2.3}}\right)} = \frac{1}{1 + \frac{0.15 \times 6.94^{1.8}}{0.02 \times 7.7^{2.3}}} = 0.31$$

The intercepted flow is calculated as:

$$Q_i = Q_w + R_x Q_x = 6.43 + 0.31 \times 5.32 = 8.1 \text{ cfs}$$

Using four units, the interception ratio for this example is: 8.1/11.7 =70% and the carry-over flow is 3.6 cfs for this case.

➢ Example for In-sump Grate

A bar grate inlet in **Figure 4.**6 has a unit length of 2.5 ft and a unit width of 1.5 ft. The steel bars occupy 40% of the grate surface area. Calculate the interception capacity for one bar grate under a water depth of 0.5 foot.

When the inlet operates like a weir, the capacity is determined to be:

$$P_e = 2 \times 1.5 + (1-0.5) \times 2.5 = 4.25 \text{ ft}$$

With C_w=3.0, the weir capacity is calculated as:

$$Q_w = 3.0 \times 4.25 \times 0.5^{1.5} = 4.5 \text{ cfs}$$

The net opening area for the grate is calculated as the difference between the grate area and the steel-bar area as:

$$m = 1 - 0.4 = 0.6$$

$$A_e = (1-0.5)\times 0.6\times 2.5\times 1.5 = 1.13 \text{ sq feet.}$$

With C_0 =0.65, the interception capacity is calculated as:

$$Q_o = 0.65\times 1.13\times \sqrt{64.4\times 0.5} = 4.2 \text{ cfs}$$

For this case, the weir flow dictates the interception capacity as:

$$Q_i = \min(4.2,\ 4.5) = 4.2 \text{ cfs.}$$

- ### Example for On-grade Curb Opening Inlet

Referring to the Example for Design Flow above, the design flow on the street has: Q_s = 11.7 cfs, Q_w=6.43 cfs and Q_x=5.32 cfs. The curb opening inlet in **Figure 4.10.b** has a length of 5 feet and open height of 4 inches. Considering a clogging factor of 0.12 for a single unit, determine the interception rate for 4 units of curb-opening inlet.

Figure 4.10.b Curb Opening Inlet

For this case, the gutter slope is

$$S_w = S_x + \frac{D_s}{W} = 0.02 + \frac{\left(\frac{2}{12}\right)}{2.5} = 0.087 \text{ ft/ft}$$

The equivalent transverse slope is calculated as:

$$S_e = 0.02 + 0.087\times \frac{6.43}{11.7} = 0.068 \text{ ft/ft}$$

The required length of the curb opening inlet is:

$$L_t = 0.60\times 11.70^{0.42}\times 0.03^{0.30}\times\left(\frac{1}{0.016\times 0.068}\right)^{0.6} = 35.5 \text{ ft}$$

Try four units. The total length of the inlet is:

$L = 5.0 \times 4.0 = 20$ ft

The clogging factor for 4 units of curb-opening inlet is 0.04 from **Table 4.5**. The effective length of the curb opening inlet is:

$L_e = (1 - 0.04) \times 20 = 19.2$ ft

Substituting the effective length into Eq 4-29 yields:

$$Q_i = 11.7 \times \left[1 - \left(1 - \frac{19.2}{35.5}\right)^{1.80}\right] = 8.83 \text{ cfs}$$

This inlet has an interception ratio of 75%. The carry-over flow is 2.87 cfs for this case.

- **Example for in-sump Curb Opening Inlet**

As illustrated in **Figure 4.13**, the 3-ft curb opening inlet with a depression pan is used as the in-sump inlet. The clogging factor for a single curb-opening inlet is 12%. A 3-inch concrete cover is needed to protect to the inlet. The curb height is 6 inches along the street gutter. No overtopping is allowed. Determine the interception capacity.

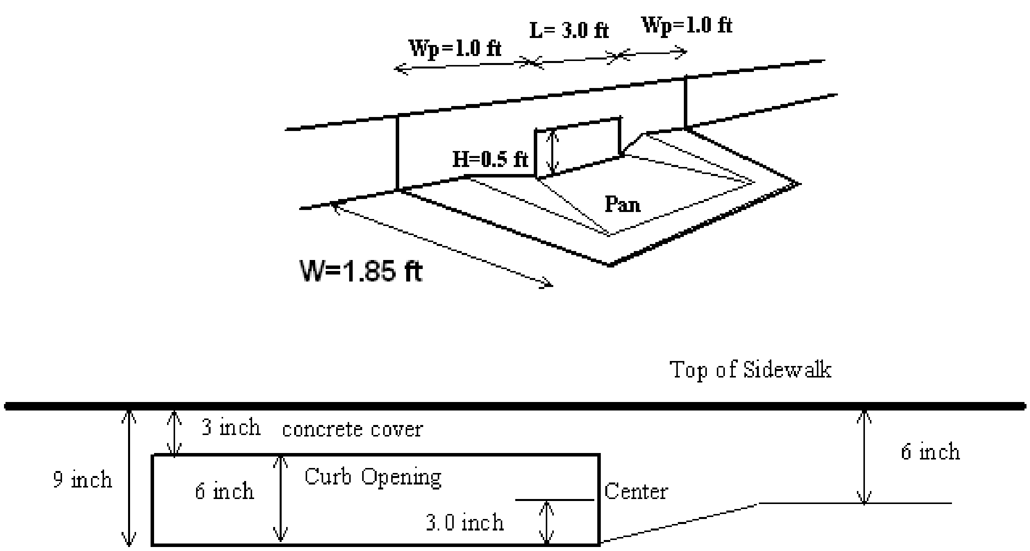

Figure 4.13 Example of Curb Opening Inlet in Sump

Considering the 3-inch concrete cover on top of the 6-inch opening, the water depth is calculated as:

$Y_s = 3 + 6 = 9$ inches

Consider k = 2.0. The effective weir length for the depression pan is:

$P_e = (1 - 0.12) \times (3.0 + 2.0 \times 1.0) + 2 \times 1.85 = 8.10$ ft

The weir flow capacity is estimated as:

$$Q_w = 3.0 \times 8.10 \times (\frac{9}{12})^{1.5} = 15.7 \text{ cfs}$$

The unclogged curb opening area is

$$A = (1 - 0.12) \times 3 \times \frac{6}{12} = 1.32 \text{ sq foot}$$

The center of the curb opening area is 3 inches above the flow line. The orifice flow capacity is estimated as:

$$Q_o = 0.65 \times 1.32 \times \sqrt{2g(9/12 - 3/12)} = 4.87 \text{ cfs}$$

The interception capacity for this curb opening is

$$Q_i = \min(Q_w,\ Q_o) = 4.87 \text{ cfs}$$

- **Example for Circular Sewer**

Design a circular sewer to deliver a discharge of 40 cfs on a slope of 1.0 % with a Manning's roughness coefficient of 0.015.

1. Find the hydraulically required pipe size

$$d = (\frac{0.015 \times 40.0}{0.462\sqrt{0.01}})^{\frac{3}{8}} \times 12 = 31.36 \text{ inches}$$

2. Use a 36-inch circular sewer that has a full flow capacity as

$$Q_f = \frac{1.49}{0.015} \times 0.75^{\frac{2}{3}} \times 7.07 \times \sqrt{0.01} = 57.92 \text{ cfs}$$

3. Determine the design flow condition in the 36-inch pipe.

$$\frac{Q}{Q_f} = \frac{40}{57.92} = 0.69 = \frac{1}{\pi}\left(\frac{1}{\theta}\right)^{\frac{2}{3}}(\theta - \sin\theta\cos\theta)^{\frac{5}{3}}$$

By trial and error, the central angle is found to be 1.79 radians or 102.8 degrees. The flow condition for the design discharge in the 36-inch sewer can be calculated as:

$$Y = \frac{d}{2}(1 - \cos\theta) = \frac{3}{2}(1 - \cos 1.79) = 1.83 \text{ ft}$$

$$A = \frac{d^2}{4}(\theta - \sin\theta\cos\theta) = \frac{3^2}{4}(1.79 - \sin 1.79 \times \cos 1.79) = 4.52 \text{ ft}^2$$

$$V = \frac{Q}{A} = \frac{40.0}{4.52} = 8.85 \text{ fps}$$

$$T = d\sin\theta = 3\sin 1.79 = 2.93 \text{ ft}$$

The above analysis is based on the assumption of normal flow conditions. In fact, a sewer in a system is subject to downstream backwater effects. Under a surcharge condition, the sewer likely becomes full-flowing with a full-flow velocity as:

$$V_f = \frac{40}{3.1416 \times 3.0^2 / 4} = 5.66 \text{ fps}$$