MHFD-Detention Workbook

Technical Reference Manual

March 2022

MHFD

MILE HIGH FLOOD DISTRICT

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1 Introduction

The purpose of this technical reference manual is to provide background information and supporting documentation for the methods and equations used in the Mile High Flood District (MHFD) Detention Basin Design Workbook. The manual is organized to follow along with the typical user input sequence starting on the Basin worksheet and proceeding to the Outlet Structure worksheet. The other worksheets are also briefly discussed. Supporting documentation including technical memorandums and research studies are included in the appendices of this manual and links are provided to other references on the MHFD website. For guidance on how to use the MHFD-Detention workbook, please refer to the instructional videos available on the MHFD YouTube channel.

The Intro Worksheet within the MHFD-Detention workbook provides useful information regarding updates that have been made to the workbook. The version number and release rate indicate how recently the workbook was updated. For information on previous versions of the workbook, the user can click the button in the upper right for a history of the most recent revisions. The Intro worksheet also provides buttons which serve as a shortcut to other worksheets in the workbook and include a brief description of what each worksheet covers. At the bottom of the worksheet, links are provided to the MHFD website and to an email address for comments or questions.

2 Basin Worksheet

This chapter provides an outline of the Basin worksheet and the underlying equations used in calculations. User input cells are designated with blue shading and are unlocked. The remaining cells in the worksheet are locked and provide calculated results. At the top of the worksheet, the user can enter a Project name or description and a Basin ID. Values entered in these cells will automatically copied over to the Outlet Structure worksheet. The *Clear Workbook* button also appears at the top of the worksheet. Clicking the *Clear Workbook* button will clear all user inputs on the Basin worksheet and Outlet Structure worksheet as well as a hidden worksheet storing CUHP results. This essentially returns the workbook to the empty condition found when downloading it from the MHFD website. The program will ask the user for confirmation prior to clearing all inputs in case it was accidently clicked.

2.1 Watershed Inputs

The first section of the Basin worksheet, as shown in Figure 2.1, provides user input cells for a single subcatchment. These input values are used in subsequent equations and in running the Colorado Urban Hydrograph Procedure (CUHP) to calculate hydrology.

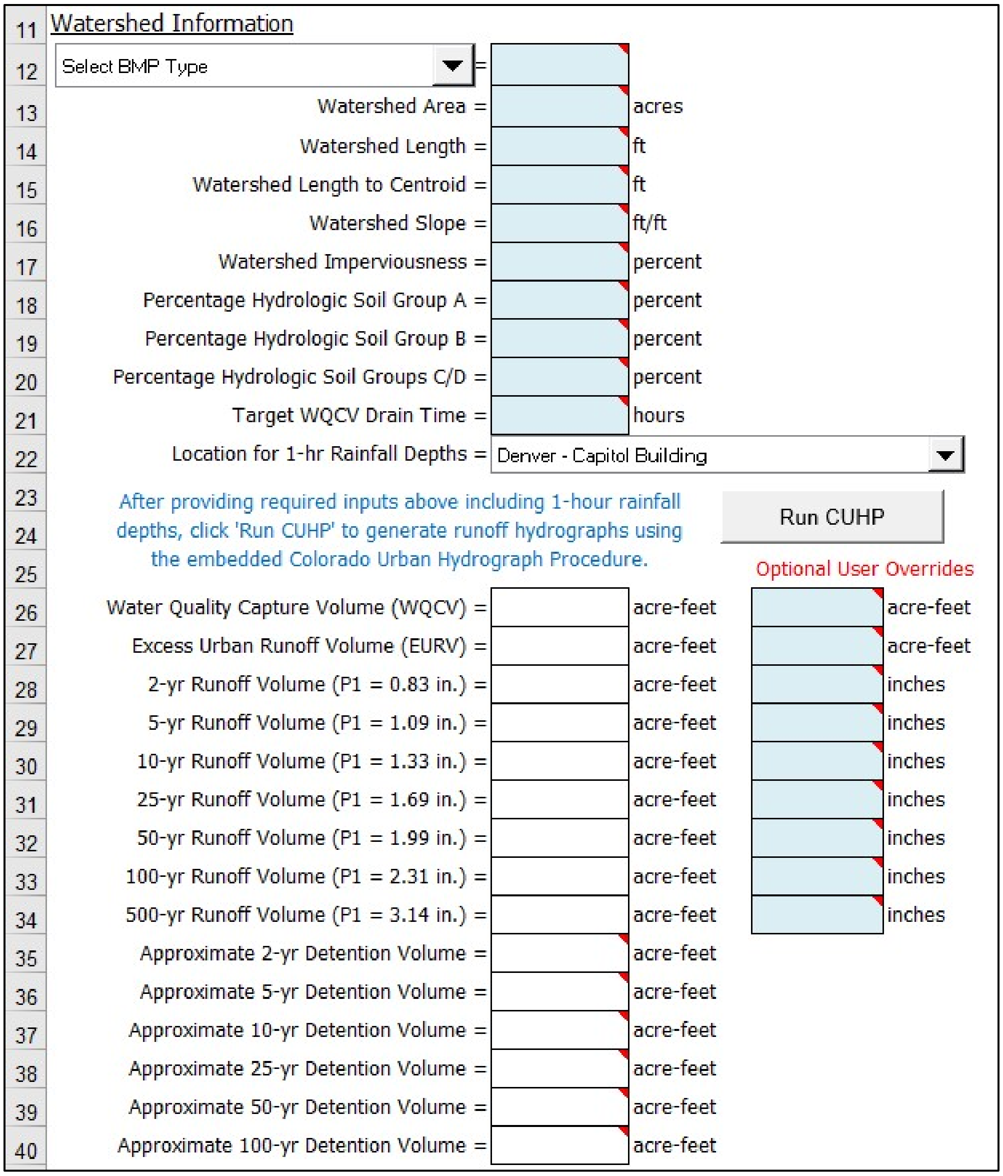


Figure 2.1 – Watershed Information

- The first user input is a pulldown list to **Select BMP Type**. The list includes:
  - Extended Detention Pond (EDB)
  - Retention Pond (RP)
  - Constructed Wetland Pond (CWP)
  - Sand Filter (SF)
  - Rain Garden (RG) – Bioretention
  - Flood Control Only – No BMP

Selection of the BMP Type will automatically set the Target WQCV Drain Time to a default value. An EDB is set to 40 hours, a CWP is set to 24 hours, and a RP, SF, and RG are all set to 12 hours. When Flood Control Only is selected, the WQCV drain time is not applicable. Selection of the EDB type will also set a default value for the Initial Surcharge Volume in the basin geometry input section. For all other BMP Types, the Initial Surcharge Volume, Initial Surcharge Depth, Depth of Trickle Channel, and Slope of Trickle Channel are all set to not applicable and the input cells are locked.

- **Watershed Area (acres)** must be between 0.01 acres (greater than zero) and 3200 acres (5 square miles). This limitation is based on MHFD guidance for CUHP.
- **Watershed Length (feet)** must be greater than zero and represents the distance from the design point of the subcatchment along the main drainageway path to the furthest point on the subcatchment boundary. The workbook automatically performs a check on the Area and Length input values to determine if the resulting subcatchment shape is consistent with MHFD guidance. The workbook evaluates the ratio r = $Length^2$ / *Area* which is a shape parameter to evaluate the length to width ratio of the subcatchment. If the r value is less than 1.0 or greater than 8.0 it will be flagged to notify the user.
- **Watershed Length to Centroid (feet)** must also be greater than zero and represents the distance from the design point of the subcatchment along the main drainageway to the subcatchment centroid. The workbook automatically performs a check on the Length and Length to Centroid input values to determine if the resulting subcatchment shape is consistent with MHFD guidance. The workbook evaluates the ratio r = *Length to Centroid / Total Length*. If the r value is less than 0.1 or greater than 0.9 it will be flagged to notify the user.
- **Watershed Slope (ft/ft)** must also be greater than zero and represents the length-weighted, corrected average slope of the subcatchment. The workbook automatically performs a check on the Slope. If the slope is less than 0.005 it is flagged as flat. If the slope is greater than 0.06 it is flagged as steep. Very flat or very steep slopes are not well-represented by the hydrological processes used in CUHP.
- **Watershed Imperviousness (percent)** must be between 2% and 100% and represents the portion of the subcatchment total surface area that is impervious.
- **Percentage Hydrologic Soil Group (percent)** for each soil type (A, B, and C/D) must be between 0% and 100%. The program checks to make sure the sum of the three percentages adds up to 100% and notifies the user if it does not.
- **Target WQCV Drain Time (hours)** is set to a default value based on the BMP Type selection. The user can override the drain time and it will be flagged as too short or too long relative to MHFD guidance in order to highlight the user change.
- **Location for 1-hr Rainfall Depths** is selected from a pulldown list with approximately 45 different locations within the District boundary as well as the option to select User Input. When a location is selected, the program obtains the 1-hour rainfall depth for each of the return periods (2-year through 500-year) from a hidden worksheet titled Program Data. The selected rainfall depths can be seen in Cells A28:A34 in parenthesis (e.g., P1 = 2.31 in.). If User Input is selected, the user can provide 1-hour rainfall depth overrides in

Cells D28:D34. The program will automatically check to ensure that the rainfall depth increases for each return period.

2.2 Run CUHP

After the user has provided the required inputs outlined above, the *Run CUHP* button can be clicked to generate runoff hydrographs using the embedded Colorado Urban Hydrograph Procedure (CUHP). The CUHP math engine code has been copied into this workbook in order to generate runoff hydrographs and peak flows consistent with the standalone CUHP workbook. The only difference between the standalone CUHP software and the embedded CUHP-Lite version in this workbook, is the limited ability to override some of the user input values. The CUHP results are identical for both workbooks given the same input.

2.2.1 CUHP-Lite Inputs

The required subcatchment inputs for CUHP-Lite to run are shown in Figure 2.2. This figure is a screen shot taken from a hidden worksheet (CUHP_Calcs) in the MHFD-Detention workbook where the input parameters are organized to be consistent with the standalone CUHP Subcatchments worksheet in order to minimize the need for modifications to the CUHP code.

	A	B	C	D	E	F	G	H	I	J	K	L	M
1	Colorado Urban Hydrograph Procedure (CUHP-Lite)												
2													
3	Subcatchments								Depression Storage		Horton's Infiltration Parameters		
4	Subcatchment Name	EPA SWMM Target Node	Raingage	Area (mi^2)	Length to Centroid (mi)	Length (mi)	Slope (ft/ft)	Percent Imperviousness	Pervious DS (in)	Impervious DS (in)	Initial Infiltration (in/hr)	Decay Coeff. (1/sec)	Final Infiltration (in/hr)
5	Pre-WQCV		WQCV	0.0313	0.2273	0.3788	0.0200	2	0.35	0.10	3.0	0.0018	0.5
6	Pre-EURV		EURV	0.0313	0.2273	0.3788	0.0200	2	0.35	0.10	3.0	0.0018	0.5
7	Pre-2yr		2yr	0.0313	0.2273	0.3788	0.0200	2	0.35	0.10	3.0	0.0018	0.5
8	Pre-5yr		5yr	0.0313	0.2273	0.3788	0.0200	2	0.35	0.10	3.0	0.0018	0.5
9	Pre-10yr		10yr	0.0313	0.2273	0.3788	0.0200	2	0.35	0.10	3.0	0.0018	0.5
10	Pre-25yr		25yr	0.0313	0.2273	0.3788	0.0200	2	0.35	0.10	3.0	0.0018	0.5
11	Pre-50yr		50yr	0.0313	0.2273	0.3788	0.0200	2	0.35	0.10	3.0	0.0018	0.5
12	Pre-100yr		100yr	0.0313	0.2273	0.3788	0.0200	2	0.35	0.10	3.0	0.0018	0.5
13	Pre-500yr		500yr	0.0313	0.2273	0.3788	0.0200	2	0.35	0.10	3.0	0.0018	0.5
14	Post-WQCV		WQCV	0.0313	0.2273	0.3788	0.0200	40	0.35	0.10	3.0	0.0018	0.5
15	Post-EURV		EURV	0.0313	0.2273	0.3788	0.0200	40	0.35	0.10	3.0	0.0018	0.5
16	Post-2yr		2yr	0.0313	0.2273	0.3788	0.0200	40	0.35	0.10	3.0	0.0018	0.5
17	Post-5yr		5yr	0.0313	0.2273	0.3788	0.0200	40	0.35	0.10	3.0	0.0018	0.5
18	Post-10yr		10yr	0.0313	0.2273	0.3788	0.0200	40	0.35	0.10	3.0	0.0018	0.5
19	Post-25yr		25yr	0.0313	0.2273	0.3788	0.0200	40	0.35	0.10	3.0	0.0018	0.5
20	Post-50yr		50yr	0.0313	0.2273	0.3788	0.0200	40	0.35	0.10	3.0	0.0018	0.5
21	Post-100yr		100yr	0.0313	0.2273	0.3788	0.0200	40	0.35	0.10	3.0	0.0018	0.5
22	Post-500yr		500yr	0.0313	0.2273	0.3788	0.0200	40	0.35	0.10	3.0	0.0018	0.5
23													

Figure 2.2 – CUHP Inputs

The hidden CUHP-Lite worksheet in the MHFD-Detention workbook is setup to include 18 subcatchments as a way to calculate both pre-development and post-development hydrology for each of the nine return periods (WQCV through 500-year) in a single run of the CUHP code. The table is blank when starting a new workbook, but Figure 2.2 shows the table populated with values from an example problem for purposes of explaining the input values. The yellow highlighted cells show the user input values taken from the Basin worksheet and default values based on MHFD recommendations.

Direct input values provided by the user on the Basin worksheet for this example include:

- **Area** (20 acres converted by program to 0.0313 square miles)
- **Length to Centroid** (1,200 feet converted by program to 0.2273 miles)
- **Length** (2,000 feet converted by program to 0.3788 miles)
- **Slope** (0.02 ft/ft or 2%)
- **Post-Development Percent Imperviousness** (40% as seen in Cell H14)

The remaining input values shown in the table are based on MHFD recommendations and the user input Soil Type Percentages provided by the user on the Basin worksheet. Figure 2.3 shows the default values recommended by MHFD for use with CUHP.

- **Pre-Development Percent Imperviousness** is set to a default value of 2% as seen in Cell H5 of Figure 2.2. This value cannot be changed by the user directly. However, the user can override the resulting pre-development peak flows on the Outlet Structure worksheet which will be discussed later.
- **Depression Storage** values are set to MHFD recommended values of 0.35 for pervious lawn grass and 0.10 for impervious paved areas and flat roofs. The user is not able to override these values in the MHFD-Detention workbook.
- **Horton Infiltration Parameters** are calculated based on the area-weighted average of the user input Soil Type percentages for Hydrologic Soil Groups A, B, and C/D. In this example, the subcatchment consisted of 100% Type C/D soils and the values were taken directly from the C/D rows in the recommended table. However, if the subcatchment were 50% Type A and 50% Type B, the program would calculate the area-weighted average (e.g., initial infiltration rate of 4.75 in/hr). The user is not able to directly override these values in the MHFD-Detention workbook.

Typical Depression Losses for Various Land Covers (All Values in Inches)		
Land Cover	Range in Depression (Retention) Losses	Recommended
Impervious:		
Large paved areas	0.05 - 0.15	0.1
Roofs-flat	0.1 - 0.3	0.1
Roofs-sloped	0.05 - 0.1	0.05
Pervious:		
Lawn grass	0.2 - 0.5	0.35
Wooded areas and open fields	0.2 - 0.6	0.4

Recommended Horton's Equation Parameters			
NRCS Hydrologic Soil Group	Infiltration (inches per hour)		Decay Coefficient - a
	Initial - f_i	Final - f_o	
A	5.0	1.0	0.0007
B	4.5	0.6	0.0018
C	3.0	0.5	0.0018
D	3.0	0.5	0.0018

Figure 2.3 – Default CUHP Parameters

The standalone CUHP software also allows the user to provide DCIA levels (0, 1, or 2) and user overrides for the Directly Connected Impervious Fraction (DCIF), Receiving Pervious Fraction (RPF), C_T coefficient, and C_P coefficient for each subcatchment. The user is not able to override these values in CUHP-Lite within the MHFD-Detention workbook.

In addition to the subcatchment input parameters, CUHP also needs information for the time step interval and design storm distributions in order to calculate the storm hydrographs. The MHFD-Detention workbook uses a default time-step of 5 minutes which cannot be modified by the user. As for design storm distributions, the MHFD-Detention workbook uses the 1-hour rainfall depths provided by the user on the Basin worksheet and distributes this depth using the 2-hour design storm distributions provided in Table 5-2, Volume 1 of the Urban Storm Drainage Criteria Manual (USDCM).

If the CUHP-Lite input limitations create a problem and the user is justified in using different values for any of these CUHP input parameters, the user can override the CUHP calculated hydrology on the Outlet Structure worksheet with user input pre-development peak discharges and post-development storm hydrographs developed separately in the standalone CUHP software. These hydrology overrides on the Outlet Structure worksheet are discussed later in this manual. However, even if the user plans to override the hydrology, they are still required to run CUHP-Lite to progress through the MHFD-Detention workbook.

2.2.2 CUHP-Lite Results

Once the *CUHP Run* button is clicked the program will run and calculate the runoff volumes, unit hydrographs, storm hydrographs, and peak discharges for each of the 18 subcatchment scenarios (pre- and post-development for 9 different return periods). For more information on the CUHP software and the methods used to calculate hydrology, please refer to the CUHP 2005 User Manual available on the MHFD website.

The CUHP calculated results are stored on the hidden CUHP-Lite worksheet in the MHFD-Detention workbook. Only the relevant information required for use with the MHFD-Detention workbook is copied back to the Basin and Outlet Structure worksheets and made visible to the user. On the Basin worksheet the 2-year through 500-year runoff volumes (acre-feet) are copied into Cells B28:B34. On the Outlet Structure worksheet, the 2-year through 500-year pre-development peak flows (cfs) are copied into Cells D65:J65 and the 2-year through 500-year post-development storm hydrographs (5-minute increment flow rates, cfs) are copied into Cells D4537:J4608 (bottom of hidden Routing Table).

It should be noted that the WQCV and EURV results calculated by CUHP are no longer used in the MHFD-Detention workbook (v4.01 and later) since these capture volumes are not accurately represented by hydrographs. Instead, the WQCV and EURV are calculated using empirical equations provided in the USDCM and the volumes are routed through the outlet structure by starting at the brim full capacity.

2.3 Watershed Results

After CUHP-Lite has been run, the Watershed results in Cells B26:B40 will populate with calculated values.

The **WQCV** in cell B26 is calculated as a function of imperviousness, BMP type and watershed area as shown in the following equation taken from Volume 3 of the USDCM (modified to convert from watershed inches to acre-feet). The development of this equation is documented in *Sizing a Capture Volume for Stormwater Quality Enhancement* (1989) which is provided in Appendix A.

$$WQCV = a(0.91I^3 - 1.19I^2 + 0.78I)/12 * Area$$

Where:

WQCV = Water Quality Capture Volume (acre-feet)

a = Coefficient corresponding to BMP Type and based on WQCV drain time

(1.0 for 40-hr, 0.9 for 24-hr, 0.8 for 12-hr)

I = Imperviousness (percent expressed as a decimal)

Area = Watershed Area (acres)

The user can override the program calculated WQCV by entering an override value in Cell D26. The program will automatically check to make sure that the WQCV override is not larger than the EURV and notify the user with a message if it is. Otherwise, the override WQCV will be shown in Cell B26 and will be shaded pink to highlight that it has been overridden by the user.

The **EURV** in cell B27 is calculated as a function of imperviousness, soil type, and watershed area as shown in the following equation. The Technical Memorandum entitled *Determination of the EURV for Full Spectrum Detention Design*, dated December 22, 2016 documents the derivation of this equation (Appendix B). The same equation is also shown in a different form for watershed inches in the Storage chapter of the USDCM Volume 2.

$$EURV = (0.140I^{1.28} * \%A - 1.113I^{1.08} * \%B + 0.100I^{1.08} * \%C/D) * Area$$

Where:

EURV = Excess Urban Runoff Volume (acre-feet)

I = Imperviousness (percent expressed as a decimal)

%A, %B, %C/D = Percentage of each Hydrologic Soil Group (sum to 100%)

Area = Watershed Area (acres)

The user can override the program calculated EURV by entering an override value in Cell D27. The program will automatically check to make sure that EURV override is not smaller than the

WQCV and notify the user with a message if it is. Otherwise, the override EURV will be shown in Cell B27 and will be shaded pink to highlight that it has been overridden.

As discussed in Section 2.2.2, the 2-year through 100-year **Runoff Volumes** in Cells B28:B34 are calculated by the CUHP code and copied here from the hidden CUHP-Lite worksheet.

The **Approximate Detention Volumes** in Cells B35:B40 for the 2-year through 100-year storms are calculated using empirical equations. The derivation of these equations is documented in a Technical Memorandum entitled *Estimation of Runoff and Storage Volumes for Use with Full Spectrum Detention*, dated January 5, 2017 (Appendix C). These equations solve for the approximate storage volume in acre-feet as a function of the one-hour rainfall depth (P1, inches) corresponding to a return period, the watershed area in acres, the percentage imperviousness (expressed as a decimal), and the percent of each hydrologic soil group (expressed as a decimal). These approximate detention volumes provide a starting point for the user to size preliminary basin geometry in the next section of the Basin worksheet. However, the actual storage volumes required for design will be determined by routing the design storms through the basin on the Outlet Structure worksheet.

2.4 Storage Volume Zones

The next section of the Basin worksheet involves defining the Storage Volume Zones and Basin Geometry as seen in Figure 2.4. Rows 43 through 45 allow the user to select the desired storage volumes for Zones 1 through 3 using a pulldown list. At a minimum, a storage volume for Zone 1 is required to proceed further into the workbook. Zones 2 and 3 are optional and can be left blank if not applicable. For a full spectrum detention facility; Zone 1 represents the WQCV, Zone 2 represents the difference between the EURV and WQCV, and Zone 3 represents the difference between the 100-year volume and the EURV. Therefore, the total detention basin volume (Cell B46) is equal to the sum of Zones 1 through 3 which is equal to the 100-year full spectrum detention volume and is inclusive of the WQCV and the EURV. For more information on the different storage zones for the different BMP Types, see Chapter 12 of the USDCM Volume 2.

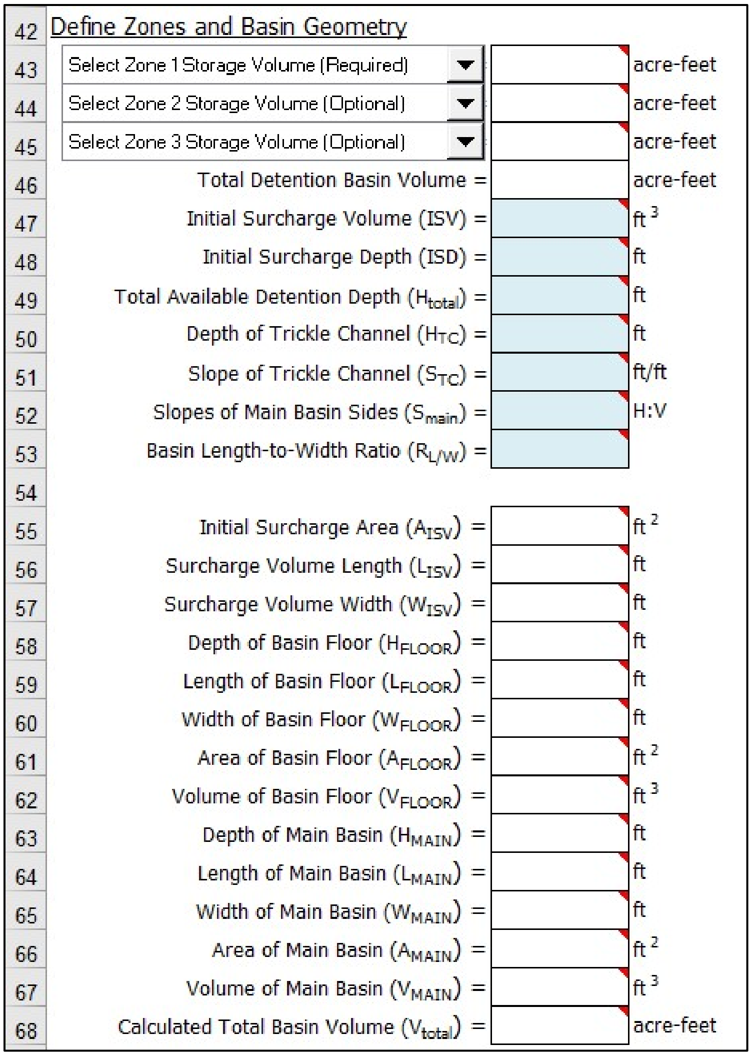


Figure 2.4 – Zones and Basin Geometry

2.4.1 Zone 1 Storage Volume

Selection of a **Zone 1 Storage Volume (acre-feet)** is required and the available options in the pulldown list include:

- WQCV
- EURV – WQCV
- Approximate 2-year Detention Volume
- Approximate 5-year Detention Volume
- Approximate 10-year Detention Volume
- Approximate 25-year Detention Volume
- Approximate 50-year Detention Volume
- Approximate 100-year Detention Volume
- User Defined Volume

When **WQCV** is selected, the program references the value for WQCV from Cell B26 above for Cell B43. If the user provided a WQCV override in Cell D26, that is the value that will be used in Cell B43. It should be noted that if Flood Control Only was selected for BMP Type, selection of WQCV for Zone 1 is not a valid selection.

When **EURV – WQCV** is selected, the program will enter an equation in Cell B43 equal to the EURV (Cell B27) minus the WQCV (Cell B26). This selection assumes that the WQCV is treated in a separate upstream BMP. In this case, user-input inflow hydrographs must be copied into the Outlet Structure worksheet to represent the outflow hydrographs from the upstream BMP(s). The inflow hydrographs can be obtained from another program (e.g., CUHP/SWMM) or can be developed using a separate upstream MHFD-Detention workbook. The user will be notified with a message explaining that this is required.

When one of the **Approximate Detention Volumes** for a design storm (2-year through 100-year) is selected, the program references the corresponding design storm detention volume from Cells B35:B40 above for Cell B43.

If **User Defined** is selected, the program will unlock Cell B43 and shade the cell light blue to indicate that it is now a user input cell. The user can then enter their own value for the Zone 1 Storage Volume.

It should be noted, that when the user selects the Zone 1 Storage Volume, any values previously entered for Zones 2 and 3 will be cleared. Also, if Zone 1 is anything other than WQCV, a note will be shown in Cell D43 to highlight that the WQCV is not provided.

2.4.2 Zone 2 Storage Volume

Selection of a **Zone 2 Storage Volume (acre-feet)** is optional and the pulldown list includes:

- EURV - Zone 1
- Approximate 2-year Detention Volume – Zone 1
- Approximate 5-year Detention Volume – Zone 1
- Approximate 10-year Detention Volume – Zone 1
- Approximate 25-year Detention Volume – Zone 1
- Approximate 50-year Detention Volume – Zone 1
- Approximate 100-year Detention Volume – Zone 1
- User Defined Volume

When **EURV – Zone 1** is selected, the program will enter an equation in Cell B44 equal to the EURV (Cell B27) minus the Zone 1 Volume (Cell B43). The program will also check to make sure that the EURV is greater than the Zone 1 Volume to avoid returning a negative value.

When one of the **Approximate Detention Volumes – Zone 1** is selected, the program will enter an equation in Cell B44 equal to the corresponding design storm detention volume from Cells

B35:B40 minus the Zone 1 Volume (Cell B43). The program will also check to make sure that the approximate detention volume is greater than the Zone 1 Volume to avoid returning a negative value.

If **User Defined – Zone 1** is selected, the program will unlock Cell B44 and shade the cell light blue to indicate that it is now a user input cell. The user can then enter their own value for the Zone 2 Storage Volume.

It should be noted, that when the user selects the Zone 2 Storage Volume, any value previously entered for Zone 3 will be cleared.

2.4.3 Zone 3 Storage Volume

Selection of a **Zone 3 Storage Volume (acre-feet)** is optional and the pulldown list includes:

- Approximate 2-year Detention Volume - Zones 1 & 2
- Approximate 5-year Detention Volume - Zones 1 & 2
- Approximate 10-year Detention Volume - Zones 1 & 2
- Approximate 25-year Detention Volume - Zones 1 & 2
- Approximate 50-year Detention Volume - Zones 1 & 2
- Approximate 100-year Detention Volume – Zones 1 & 2
- Approximate 100-year Detention Volume + ½ WQCV – Zones 1 & 2
- User Defined Volume

When one of the **Approximate Detention Volumes – Zones 1 & 2** is selected, the program will enter an equation in Cell B45 equal to the corresponding design storm detention volume from Cells B35:B40 minus the Zone 1 and Zone 2 Volumes (Cell B43 and Cell B44). The program will also check to make sure that the approximate detention volume is greater than the sum of the Zone 1 and Zone 2 Volumes to avoid returning a negative value.

When the **Approximate 100-year Detention Volume + ½ WQCV – Zones 1 & 2** is selected, the program will enter an equation in Cell B45 equal to the approximate 100-year detention volume (Cell B40) plus half of the WQCV (Cell B26) minus the Zone 1 and 2 Volumes (Cell B43 and Cell B44). The program will also check to make sure that the approximate 100-year detention volume is greater than the sum of the Zone 1 and Zone 2 Volumes to avoid returning a negative value.

If **User Defined – Zones 1 & 2** is selected, the program will unlock Cell B45 and shade the cell light blue to indicate that it is now a user input cell. The user can then enter their own value for the Zone 3 Storage Volume.

2.4.4 Total Detention Basin Volume

The Total Detention Basin Volume in Cell B46 is the sum of the three zone volumes. For a full spectrum detention facility consisting of the WQCV, EURV and approximate 100-year detention volume, the total detention volume should be equal to the 100-year detention volume. If the total detention volume is less than the approximate 100-year detention volume, a note will be shown in Cells D44:E46 to highlight that the total detention volume is less than the 100-year volume.

2.5 Basin Geometry Input

The next section on the Basin worksheet requires user input values to define basin geometry constraints as shown in the blue cells on Figure 2.4. These geometry constraints are used to help size preliminary basin geometry consistent with local criteria and site constraints.

- **Initial Surcharge Volume (ISV, cubic feet)** is only applicable to an EDB. For all other BMP types this input cell will be locked and populated with N/A. The initial surcharge volume is not provided in the micropool nor does it include the micropool volume. It is the available volume that begins at the top water surface elevation of the micropool and extends upward to a grade break within the basin (typically the invert of the trickle channel). The area of the initial surcharge volume, when full, is typically the same or slightly larger than that of the micropool. The program will automatically calculate an ISV equal to 0.3% of the WQCV. The user can override this default value if desired.
- **Initial Surcharge Depth (ISD, feet)** is only applicable to an EDB. For all other BMP types this input cell will be locked and populated with N/A. MHFD recommends an initial surcharge volume depth of between 4 and 6 inches (0.33 to 0.5 feet).
- **Total Available Detention Depth (feet)** applies to all BMP types. The depth is measured from the invert of Zone 1 (e.g., bottom of ISV, permanent pool surface, or filtration media surface) to the top of the highest Zone utilized (do not include freeboard in total depth). This depth is often dictated by site conditions and downstream tie-in elevations for the outlet structure. The maximum depth allowed by the program is 28 feet, which is a limitation set by the number of rows provided in the stage-area-volume table within the workbook.
- **Depth of Trickle Channel (H_{TC}, feet)** is only applicable to an EDB or a Flood Control Only basin. For all other BMP types this input cell will be locked and populated with N/A. The trickle channel conveys low flows from the forebay to the micropool and should have a minimum flow capacity equal to the maximum release from the forebay outlet. A typical value for a concrete trickle channel is 6 inches (0.5 feet). The recommended minimum depth of a soft bottom trickle channel is 1.5 feet. This depth will help limit potential wetland growth in the trickle channel, preserving the bottom of the basin.
- **Slope of Trickle Channel (S_{TC}, ft/ft)** is only applicable to an EDB or a Flood Control Only basin. For all other BMP types this input cell will be locked and populated with N/A. For concrete trickle channels, a slope between 0.004 and 0.01 ft/ft is recommended to encourage settling while reducing the potential for low points within

the concrete pan. For soft bottom trickle channels, it is recommended that they be designed with a consistent longitudinal slope from forebay to micropool and that they not meander. The program will accept slope input values from 0 to 0.03 ft/ft. If a value of zero is entered, the program will automatically replace this entry with a value of 0.000001 to avoid mathematical errors resulting from division by zero.

- **Slope of Main Basin Sides (S_{main}, H:V)** applies to all BMP types. Basin side slopes should be stable and gentle to facilitate maintenance and access. Typical side slopes range from 3 to 4 (representing 3H:1V or 4H:1V). Side slopes should be no steeper than 3:1 and will be flagged by the program in Cell D52 when less than 3:1. The use of vertical walls is highly discouraged due to maintenance constraints but the user can enter zero for vertical walls.
- **Basin Length to Width Ratio ($R_{L/W}$)** applies to all BMP types. It is recommended to have a basin length (measured along the flow path from inlet to outlet) to width ratio of at least 2:1. A ratio less than 2:1 will be flagged by the program in Cell D53. A longer flow path from inlet to outlet will minimize short circuiting and improve reduction of total suspended solids (TSS).

2.6 Basin Geometry Results

Once the zone volumes have been selected and the basin geometry constraints have been provided by the user, the program will automatically start sizing the preliminary basin geometry in Cells B55:B68. The program calculations for basin geometry are based on the equations documented in a Technical Memorandum entitled *Modeling Detention Basins*, dated February 1, 2016 (Appendix D). These equations were developed for an EDB to account for the ISV and a sloped basin bottom that provides positive drainage. Figure 2.5 shows the preliminary basin geometry for an EDB. For all other BMP types, the ISV and basin floor volume are set to zero and the program only solves for the main basin volume above the permanent pool or filtration media. The equations are visible in Cells B55:B68. The Calculated Total Basin Volume (V_{TOTAL}) in Cell B68 is equal to the sum of the Initial Surcharge Volume (ISV), Volume associated with Trickle Channel Depth (H_{TC}*A_{ISV}), Volume of Basin Floor (V_{FLOOR}), and Volume of Main Basin (V_{MAIN}).

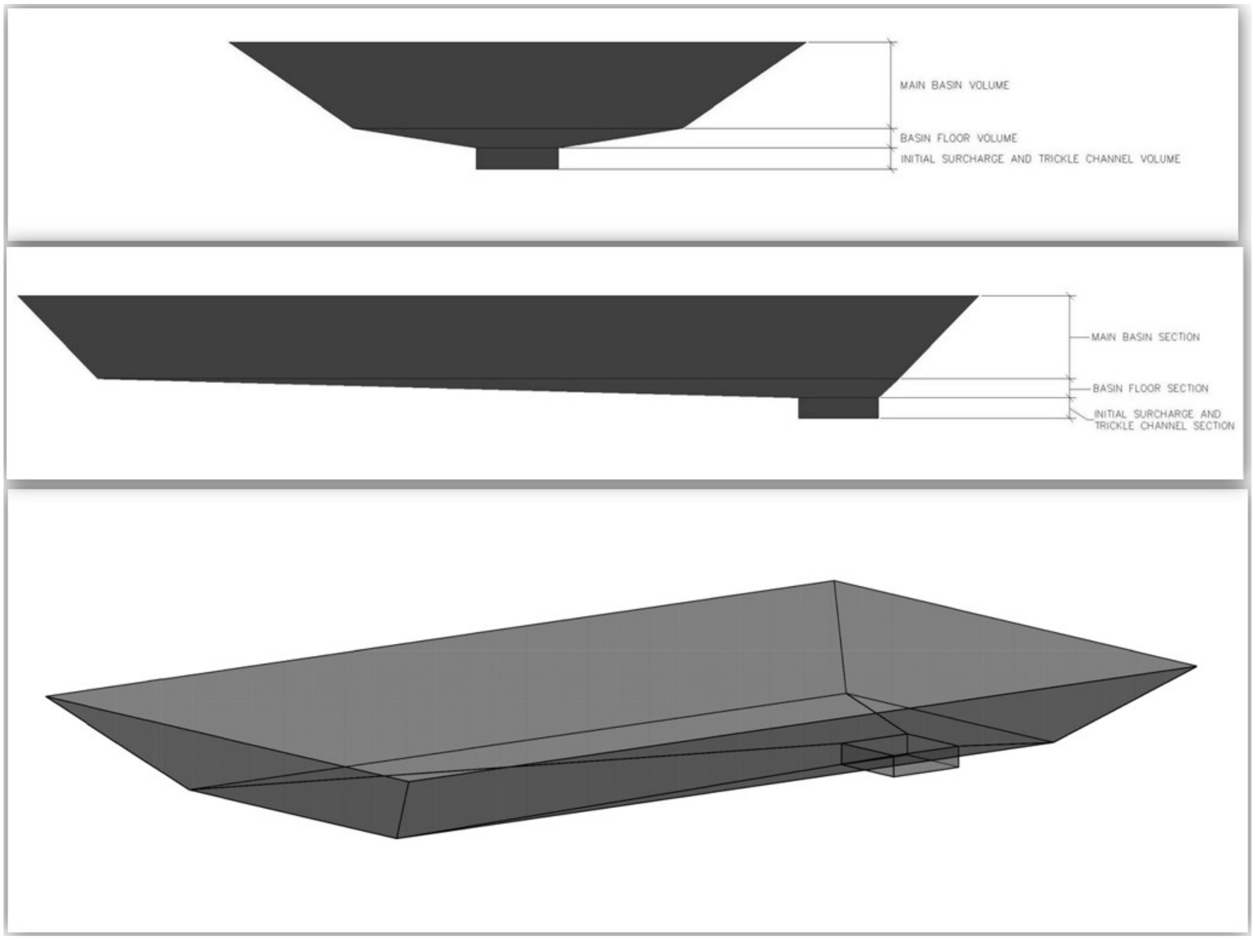


Figure 2.5 – Preliminary Basin Geometry

It should be noted that all volume calculations in the MHFD-Detention workbook use the conic approximation method (not the average end area method). The conic approximation method calculates the volume between two sectional areas; the two areas being added along with the square root of their product and multiplied by a third of distance between the areas to determine the volume, as expressed in the following equation

$$V = \left(\frac{h}{3}\right)\left(A_1 + A_2 + \sqrt{A_1 * A_2}\right)$$

Where:

V = Volume within slice (cubic feet)

A_1 = Area at bottom of slice (square feet)

A_2 = Area at top of slice (square feet)

h = Depth increment of slice (feet)

In order to solve for the preliminary basin geometry of an EDB, the program iteratively changes the depth of the basin floor (H_{FLOOR}, measured from the top of the trickle channel at the micropool to the height where the basin floor section transitions to the main basin side slopes) to match the calculated total basin volume (Cell B68) to the required detention basin volume (Cell B46). The program will start by doing an initial check to make sure there is a valid solution based on the user inputs for total available detention depth, trickle channel slope and L:W ratio.

The initial check is performed by first setting the H_{FLOOR} depth equal to the total available detention depth (H_{TOTAL}) minus the initial surcharge depth (ISD) and depth of trickle channel (S_{TC}). If the resulting total basin volume (Cell B68) calculated is less than the required detention basin volume (Cell B46), then there is no valid solution with the user provided basin geometry constraints. In this situation, the program will ask the user if they would like it to determine the maximum trickle channel slope for the given depth and L:W ratio. It can then decrease the trickle channel slope in increments of 0.001 ft/ft until the basin geometry is able to provide the required detention volume.

Once the initial check is successfully completed, the program will then determine the H_{FLOOR} depth necessary to exactly match the required detention volume while satisfying the basin geometry constraints. This is done by resetting H_{FLOOR} to a depth of 0.001 feet and then incrementally increasing the depth until the calculated volume matches the required detention volume. It should be noted that the depth of main basin (H_{MAIN}) is calculated using the following equation where the depths of the individual sections always add up to the total available depth.

$$H_{MAIN} = H_{TOTAL} - H_{FLOOR} - H_{TC} - ISD$$

For all other BMP types, the program calculations are much simpler and only focus on the main basin geometry above the permanent pool or filtration media. The program starts by setting the initial surcharge values and depth of basin floor to zero. Next, it sets the Length of Basin Floor (L_{FLOOR}) to zero and then incrementally increases the floor length until the calculated volume at the total available detention depth matches the required detention volume. When a sand filter or rain garden are being designed, if the resulting bottom area is too small relative to the tributary impervious area a message will be shown in Cells A70:E72 that states "The area of the basin floor is not adequately sized for the watershed and may clog prematurely. Decrease the available detention depth to increase the basin floor area or enter stage/area data using the optional override.".

2.7 Stage-Area-Volume Tables

Once the basin geometry has been iteratively sized by the program to match the required detention volume, the program will then automatically populate the hidden Stage-Area-Volume table in Cells F116:O3116. The table rows are hidden by default but can be made visible by clicking the button in Cell F112. The hidden table consists of equations that create a full stage-area-volume relationship at 0.01-foot increments based on the computed basin dimensions. The table consists of the following columns:

- **Stage Description** in column F is the last column filled out in the hidden stage-area-volume table. Once the remaining columns discussed below are calculated, the program will then go back and run a code routine to step through each row of the hidden table and provide stage description labels to the rows corresponding to the selected zone volumes. For EDBs, the top of the ISV and the basin floor will also be labeled unless override stage-area values have been entered by the user.
- **Stage (feet)** in column G is the stage from 0 to 30 feet at increments of 0.01-feet. If the user provides their own stage overrides in Cells H12:H109, this column is not applicable.
- **Override Stage (feet)** in Column H is only applicable if the user has provided their own override stage values in the condensed Stage-Area-Volume table in Cells H12:H109. If they have provided overrides, the program will populate this column with stage increments of 0.01-feet starting at zero and stopping at the maximum stage value provided by the user.
- **Length (feet)** in Column I is interpolated from the basin geometry. For an EDB, the first row and all rows up to the stage at the top of the ISV and trickle channel depth, the length is equal to the ISV length. For stage increments above the trickle channel depth and below the floor depth, the corresponding length is the sum of the ISV length, the horizontal length along the trickle channel slope in front of the ISV, and the horizontal length along the main basin side slope behind the ISV. For stage increments above the floor depth, the corresponding length is the sum of the floor length and two times the horizontal length along the main basin side slope. For all other BMP types, the corresponding length is the sum of the basin floor length and two times the horizontal length along the main basin side slope. If the user provides their own area overrides in Cells L12:L109, this column is not applicable.
- **Width (feet)** in Column J is also interpolated from the basin geometry. For an EDB, the first row and all rows up to the stage at the top of the ISV and trickle channel depth, the width is equal to the ISV width. For stage increments above the trickle channel depth and below the floor depth, the corresponding width is the sum of the ISV width and the horizontal width along the trickle channel slope multiplied by the L/W ratio of the basin (this steepens the trickle channel slope to account for the shorter width of the basin). For stage increments above the floor depth, the corresponding width is the sum of the floor width and two times the horizontal width along the main basin side slope. For all other BMP types, the corresponding width is the sum of the basin floor width and two times the horizontal width along the main basin side slope. If the user provides their own area overrides in Cells L12:L109, this column is not applicable.
- **Area (square feet)** in Column K is calculated as length times width for each row. If the user provides their own area overrides in Cells L12:L109, this column is not applicable.

- **Override Area (square feet)** in Column L is only applicable if the user has provided their own override area values in the condensed Stage-Area-Volume table in Cells L12:L109. If the user has entered an area override value for each override stage value entered, the program will automatically interpolate between these stage-area pairs using a code routine to fill in the area column (Cells L116:L3116).
- **Area (acres)** in Column M converts the Area in column K or L from square feet to acres.
- **Volume (cubic feet)** in Column N calculates the cumulative volume for each row using the conic approximation method. The program determines whether to use the stage and area values in columns G and K or the user override values in columns H and L.
- **Volume (acre-feet)** in Column O converts the Volume in column N from cubic feet to acre-feet.

Once the full Stage-Area-Volume table hidden in Cells F116:O3116 is completed, the program will then automatically calculate the summary Stage-Area-Volume table in Cells F9:O112 by condensing the full table into larger stage increments. The default depth increment in Cell G8 is 0.1-feet, but the user can override this value with any increment greater than or equal to 0.01 feet. Based on the provided depth increment, the program will fill out the condensed table with the following columns:

- **Stage - Storage Description** in Cells F12:F109 are typically filled in by the program at the same time the Stage values are filled in as described in the next bullet point. When the important stages are added (e.g., ISV depth, floor depth, and zone volume depths) the program will insert the corresponding label in the same row of column F. However, if the user provides their own stage overrides in Cells H12:H109, this column is unlocked and shaded light blue so that the user can provide their own descriptions corresponding to each stage-area pair entered.
- **Stage (feet)** values in Cells G11:G109 are filled in using a code routine to step up the stage value by the selected depth increment (Cell G8) while also making sure to include other important stages such as the top of the ISV, the depth where an EDB transitions from the trickle channel slope to the main basin side slope (referred to as the floor), and the stage corresponding to each of the three selected zone volumes. If the user provides their own stage overrides in Cells H12:H109, this column is not applicable.
- **Optional Override Stage (feet)** in Cells H12:H109 allows the user to provide their own stage values. If overrides are entered, the hidden stage-area-volume table below will be updated to reflect the new values at stage increments of 0.01-feet, starting at zero and stopping at the maximum stage value provided by the user.
- **Length (feet)** in Cells I11:I109 is determined by using an index equation to look up the adjacent stage value (Cells G11:G109) in the hidden stage-area-volume table (Cells G116:O3116) and return the corresponding length (Cells I116:I3116). If the user provides their own area overrides in Cells L11:L109, this column is not applicable.

- **Width (feet)** in Cells J11:J109 is also determined using an index equation to look up the adjacent stage value (Cells G11:G109) in the hidden stage-area-volume table (Cells G116:O3116) and return the corresponding width (Cells J116:J3116). If the user provides their own area overrides in Cells L11:L109, this column is not applicable.
- **Area (square feet)** in Cells K11:K109 is also determined using an index equation to look up the adjacent stage value (Cells G11:G109) in the hidden stage-area-volume table (Cells G116:O3116) and return the corresponding area (Cells K116:K3116). If the user provides their own area overrides in Cells L11:L109, this column is not applicable.
- **Override Area (square feet)** in Cells L11:L109 allows the user to provide their own area values. If the user has entered an area override value for each override stage value entered, the program will automatically interpolate between these stage-area pairs using a code routine to fill in the area column (Cells L116:L3116) in the hidden stage-area-volume table. If there is not an area value for each corresponding stage value, the interpolated results will be cleared and a warning message will be shown in Cell I7 that states "Must enter an equal number of stage and area values!".
- **Area (acres)** in Cells (M11:M109) converts the Area in column K or L from square feet to acres.
- **Volume (cubic feet)** in Cells N12:N109 is determined using an index equation to look up the adjacent stage value (Cells G11:G109, or Cells H11:H109 if override stage values were entered) in the hidden stage-area-volume table (Cells G116:O3116) and return the corresponding volume (Cells N116:N3116).
- **Volume (acre-feet)** in Cells O12:O109 converts the Volumes in column N from cubic feet to acre-feet.

2.8 Figures

The second page of the Basin worksheet includes two figures to summarize the basin geometry and stage-area-volume relationship. The program will automatically rescale the x-axis and y-axes on both figures to best display the calculated results regardless of the size of the basin.

The top figure relates the basin length (feet) in blue and basin width (feet) in red on the left Y-axis to basin stage (X-axis). It also relates the basin area (square feet) in green on the right Y-axis to basin stage (X-axis). This helps the user to visualize the break points in the basin side slopes for an EDB where the ISV transitions to the trickle channel slope along the basin floor and then to the main side slopes above the basin floor.

The bottom figure relates the basin area (acres) in green on the left Y-axis to basin stage (X-axis), similar to the top figure but in acres instead of square feet. The bottom figure also relates the basin volume (acre-feet) in blue on the right Y-axis to basin stage (X-axis).

3 Outlet Structure Worksheet

This chapter walks through the various sections of the Outlet Structure worksheet and the underlying equations used in the calculations. This worksheet is heavily dependent on the Basin worksheet input and calculated results, including the CUHP inflow hydrograph results which are stored on a hidden worksheet. Therefore, the Basin worksheet must be completed before attempting to size an outlet structure. A message to this effect will pop up when the user activates the Outlet Structure worksheet. It should also be noted that MHFD-Detention does not consider the downstream conveyance limitations on the outlet structure such as inlet vs. outlet control of the outlet pipe or downstream tailwater conditions. These limitations must be considered outside of this workbook. If the Basin worksheet has been completed, several results will be copied over to the Outlet Structure worksheet when activated. These results include the estimated stage and volume for each of the three storage zones and the CUHP runoff volumes, pre-development peak flows, and post-development inflow hydrographs.

At the top of the worksheet (Figure 3.1), the Project name or description and Basin ID are copied over from the Basin worksheet. The *Clear Input Parameters (including Tables)* button also appears at the top of the worksheet. Clicking this button will clear all user inputs and results on the Outlet Structure worksheet but does not go back and modify the Basin worksheet input or results in any way. The program will ask the user for confirmation prior to clearing the Outlet Structure worksheet in case it was accidently clicked.

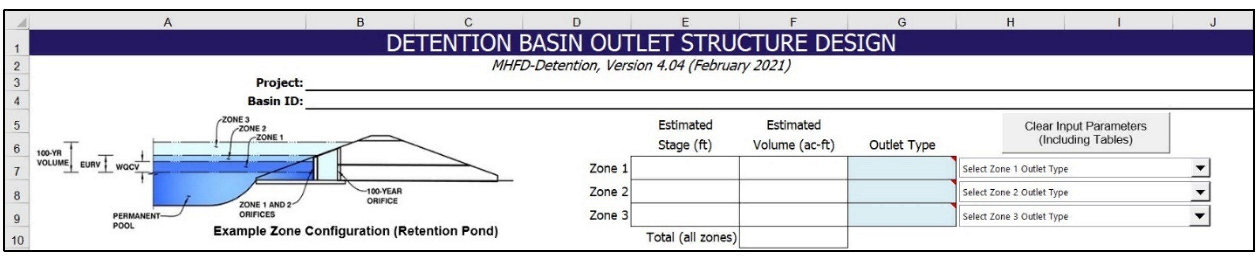


Figure 3.1 – Zone Outlet Type Selection

There are several other buttons the user can click on the Outlet Structure worksheet. However, many of them may not be visible depending on the BMP Type, Zone Volumes, and Outlet Types selected. When changes are made to the workbook, the program will automatically check to see if the visible buttons are still appropriate, and hide them if they are not. These various buttons will be discussed in their respective sections below.

3.1 General Overview of Worksheet Layout

The Outlet Structure worksheet consists of several different sections that are all dependent on each other in one way or another. Therefore, a general overview will be given for the entire worksheet prior to detailed descriptions on any one section. This overview will be broken into the four pages that are set up for printing and the two hidden tables where most of the calculations are performed.

The first printed page of the Outlet Structure worksheet (Columns A:J) is where the majority of the user input values that define the outlet structure configuration are entered. These input values include selecting an Outlet Type for each Zone and then assigning dimensions to each component of the outlet structure (e.g., an orifice plate, overflow weir, outlet pipe, and emergency spillway). Adjacent to the input values are several calculated parameters associated with each outlet component. These calculated parameters are required for other calculations in the worksheet and in the code routines. At the bottom of the first page is a table of routed hydrograph results. This table provides a summary of the calculations performed in the hidden stage-storage-discharge table and Modified Puls routing table, along with the CUHP hydrology results calculated on the Basin worksheet.

The second printed page of the Outlet Structure worksheet (Columns K:U) includes three charts. The first chart plots the inflow and outflow hydrographs for each storm event. The second chart plots the ponding depth over time for each storm event. The third chart plots the stage-area-volume-discharge relationship for the basin design.

The third printed page of the Outlet Structure worksheet (Columns V:AF) is where the inflow hydrographs for each storm event are shown. The WQCV and EURV inflow hydrographs are all zeros since the program routes these events starting at brim full capacity. The 2-year through 500-year storm events all default to the CUHP inflow hydrographs generated on the Basin worksheet. The user has the ability to override these inflow hydrographs by copying and pasting new values into these cells.

The fourth printed page of the Outlet Structure worksheet (Columns AG:AQ) allows the user to create a summary stage-area-volume-discharge relationship at stage increments of their choice. This is useful for generating report tables and for focusing on important elevations such as slope transitions or a change in the controlling outlet structure component.

Below these four printed pages are two hidden tables that can be viewed by clicking a button to unhide the corresponding rows.

The first hidden table on the Outlet Structure worksheet (Rows 84:3088) is the Stage-Storage-Discharge Table. This table includes the Stage-Area-Volume relationship (Cells B87:D3087) copied over from the hidden table on the Basin worksheet. It also includes a discharge column where the user can provide override discharge values (Cells E87:E3087). The remaining columns are used to calculate the discharge at each stage increment for each individual outlet structure component, find the total discharge for the combined outlet structure, and determine which component controls the flow rate at any given stage.

The second hidden table on the Outlet Structure worksheet (Rows 3089:4609) is the Modified Puls reservoir routing table, also known as level pool reservoir routing. By default, each row of the table corresponds to a time interval of 5 minutes resulting in a total duration of 120 hours at the bottom of the table. The columns in the table are grouped by storm event (e.g., Columns D:T are for the WQCV, Columns U:AK are for the EURV, all the way through to the 500-year event in Columns EJ:EZ). Within each group of columns, the program keeps track of the inflow, outflow, and change in storage for each time interval. The drain time for each storm event is also calculated in this table. Below the routing table is another smaller table (Cells A4535:J4609) where the CUHP inflow hydrographs are stored in case the user overwrites the hydrograph values on the third printed page described above.

The remaining sections in this Chapter will go through each part of the worksheet in more detail.

3.2 Zone Outlet Type Selection

The first section on the Outlet Structure worksheet allows the user to select an Outlet Type for each of the three storage zones as seen in Figure 3.1. At a minimum, an Outlet Type for Zone 1 is required in order to drain the detention basin. Zones 2 and 3 are optional and can be left blank when not applicable. The summary table in this section copies over the appropriate stage and volume values for each zone from the Basin worksheet. The estimated volumes (acre-feet) are copied from Cells B43:B45 on the Basin worksheet. The estimated stage (feet) values are copied from hidden cells on the Basin worksheet which use an index equation to look up the estimated volumes in the hidden stage-area-volume table (Cells F116:O3116 on Basin worksheet) and return the corresponding stage values. For each zone, the user can select an Outlet Type from the pulldown list to the right of the blue cells. The available options in each pulldown list vary depending on the BMP Type and Zone Storage Volumes selected on the Basin worksheet as described in the following sections.

Any time a user goes back and makes changes to the Basin worksheet, the program will check to make sure the Outlet Types selected are still valid options upon returning to the Outlet Structure worksheet. If they are no longer appropriate, the program will notify the user with a message and then clear the invalid outlet type selections and any associated input values in the sections below.

When a user selects one of the options from the Outlet Type pulldown list the program will automatically set default values in the appropriate cells within the Outlet Structure worksheet. If this Outlet Type selection conflicts with the other zone outlet types or other input values already entered by the user, the program will notify the user and clear the conflicting information.

3.2.1 Zone 1 Outlet Type

The following outlet types are potential options for the Zone 1 Outlet Type pulldown. Beside each option is a description of when the outlet type is available to be selected and which default values will be populated by the program.

- **Filtration Media with Underdrain –** Only available when the BMP type is a Sand Filter or Rain Garden. This option is not dependent on the Zone 1 Volume selection. When selected, the Underdrain Orifice Invert Depth and Orifice Diameter (Cells B12:B13) will be unlocked and cleared to remove any previous values that may still be present.

- **Orifice Plate –** Always available except when the BMP type is a SF or RG or when the Zone 1 Volume is a Design Storm (2-year through 100-year). When selected, the program will set several defaults including:
  - Underdrain orifice input cells (B12:B13) will show N/A and the cells will be locked since they are no longer applicable.
  - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are unlocked and cleared in case a previous outlet type selection locked them and/or other values are still present in the cells.
  - Centroid of Lowest Orifice (Cell B16) is set to zero.
  - Depth at top of Zone using Orifice Plate (Cell B17) is set equal to the Estimated Stage for Zone 1 (Cell E7).
  - Orifice vertical spacing (Cell B18) is set to a default value of one-third the depth of Zone 1 (Cell B17) and converted to inches.
  - Orifice area per row (Cell B19) is left blank for the user to determine.
  - Elliptical slot axis ratio (Cell B20) is hidden since it does not apply to the orifice plate.
  - Stage of orifice centroid for the first three rows of the orifice plate (Cells B24:D24) are set to the default values of Cell B16 (zero), one-third of Cell B17, and two-thirds of Cell B17, respectively.
  - The orifice area for rows 1 through 3 (Cells B25:D25) are set equal to the default orifice area per row (Cell B19) which is currently blank. When the user selects a value for Cell B19, the three rows in the table will automatically be updated to the same value.
- **Elliptical Slot –** Always available except when the BMP type is a SF or RG or when the Zone 1 Volume is a Design Storm (2-year through 100-year). When selected, the program will set several defaults including:
  - Underdrain orifice input cells (B12:B13) will show N/A and the cells will be locked since they are no longer applicable.
  - Elliptical Slot input cells (B16:B19) are unlocked and cleared in case a previous outlet type selection locked them and/or other values are still present in the cells.
  - Elliptical slot axis ratio (Cell B20) is made visible and the cell description and units are also shown in the adjacent cells.
  - Orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked since they do not apply to the elliptical slot.
  - Invert of Elliptical Slot (Cell B16) is set to zero.
  - Depth at top of Zone using Elliptical Slot (Cell B17) is set equal to the Estimated Stage for Zone 1 (Cell E7).

  - Elliptical Slot Height (Cell B18) is set to a default value that is 0.33 feet below the zone depth in Cell B17. This accounts for mounting the plate to the structure.
  - Elliptical Slot Gap Width (Cell B19) is left blank for the user to determine.
  - Elliptical Slot Axis Ratio (Cell B20) is set to a default value of 14.
- **Vertical Orifice (Circular) –** Always available except when the BMP type is a SF or RG or when the Zone 1 Volume is WQCV. When selected, the program will set several defaults including:
  - Underdrain orifice input cells (B12:B13) will show N/A and the cells will be locked since they are no longer applicable.
  - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked since they do not apply.
  - Elliptical slot axis ratio (Cell B20) is hidden since it does not apply.
  - Invert of Vertical Orifice (Cell B33) is set to zero.
  - Depth at top of Zone using Vertical Orifice (Cell B34) is set equal to the Estimated Stage for Zone 1 (Cell E7).
  - Vertical Orifice Diameter (Cell B35) is left blank for the user to determine.
  - Vertical Orifice Width (Cell B36) is hidden since it does not apply to the circular orifice.
- **Vertical Orifice (Rectangular) –** Always available except when the BMP type is a SF or RG or when the Zone 1 Volume is WQCV. When selected, the program will set several defaults including:
  - Underdrain orifice input cells (B12:B13) will show N/A and the cells will be locked since they are no longer applicable.
  - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked since they do not apply.
  - Elliptical slot axis ratio (Cell B20) is hidden since it does not apply.
  - Invert of Vertical Orifice (Cell B33) is set to zero.
  - Depth at top of Zone using Vertical Orifice (Cell B34) is set equal to the Estimated Stage for Zone 1 (Cell E7).
  - Vertical Orifice Height (Cell B35) is set to a default of 2 inches.
  - Vertical Orifice Width (Cell B36) is made visible since it applies to the rectangular orifice. The cell is left blank for the user to determine.
- **Weir and Pipe (w/ Circular Orifice Plate) –** Always available except when the BMP type is a SF or RG or when the Zone 1 Volume is WQCV. When selected, the program will set several defaults including:
  - Underdrain orifice input cells (B12:B13) will show N/A and the cells will be locked since they are no longer applicable.
  - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked since they do not apply.
  - Elliptical slot axis ratio (Cell B20) is hidden since it does not apply.

  - Overflow Weir Front Edge Height (Cell B40) is set to zero.
  - The remaining overflow weir input cells (Cells B41:B45) are cleared and left for the user to determine.
  - The outlet pipe input values (Cells B49:B50) are also cleared and left for the user to determine.
  - The third outlet pipe input value (Cell B51) is hidden since it does not apply to the circular orifice.
- **Weir and Pipe (w/ Restrictor Plate) –** Always available except when the BMP type is a SF or RG or when the Zone 1 Volume is WQCV. When selected, the program will set several defaults including:
  - Underdrain orifice input cells (B12:B13) will show N/A and the cells will be locked since they are no longer applicable.
  - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked since they do not apply.
  - Elliptical slot axis ratio (Cell B20) is hidden since it does not apply.
  - Overflow Weir Front Edge Height (Cell B40) is set to zero.
  - The remaining overflow weir input cells (Cells B41:B45) are cleared and left for the user to determine.
  - The outlet pipe input values (Cells B49:B50) are also cleared and left for the user to determine.
  - Restrictor Plate Height Above Pipe Invert (Cell B51) is made visible since it applies to the circular pipe with restrictor plate. The cell is left blank for the user to determine.
- **Weir and Pipe (w/ Rectangular Orifice Plate) –** Always available except when the BMP type is a SF or RG or when the Zone 1 Volume is WQCV. When selected, the program will set several defaults including:
  - Underdrain orifice input cells (B12:B13) will show N/A and the cells will be locked since they are no longer applicable.
  - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked since they do not apply.
  - Elliptical slot axis ratio (Cell B20) is hidden since it does not apply.
  - Overflow Weir Front Edge Height (Cell B40) is set to zero.
  - The remaining overflow weir input cells (Cells B41:B45) are cleared and left for the user to determine.
  - The outlet pipe input values (Cells B49:B50) are also cleared and left for the user to determine.
  - Rectangular Orifice Height (Cell B51) is made visible since it applies to the rectangular orifice. The cell is left blank for the user to determine.

- **Overflow Weir (No Pipe) –** Always available except when the BMP type is a SF or RG or when the Zone 1 Volume is WQCV. When selected, the program will set several defaults including:
    - Underdrain orifice input cells (B12:B13) will show N/A and the cells will be locked since they are no longer applicable.
    - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked since they do not apply.
    - Elliptical slot axis ratio (Cell B20) is hidden since it does not apply.
    - Overflow Weir Front Edge Height (Cell B40) is set to zero.
    - Overflow Weir Bottom Length (Cell B41) and Overflow Weir Side Slopes (B42) are cleared and left for the user to determine.
    - The remaining overflow weir input cells (B43:B45) are set to N/A since they don't apply when there is no dropbox or overflow grate.
    - The outlet pipe input values (Cells B49:B50) are also set to N/A since they don't apply when there is no dropbox, overflow grate, or outlet pipe.
    - The third outlet pipe input value (Cell B51) is hidden since it does not apply when no pipe is included.
    - Spillway position relative to Overflow Weir (Cell B58) is made visible and unlocked so that the user can decide whether the overflow weir overlaps or is offset from the emergency spillway.

3.2.2 Zone 2 Outlet Type

The following outlet types are potential options for the Zone 2 Outlet Type pulldown. Beside each option is a description of when the outlet type is available to be selected and which default values will be populated by the program. The Zone 2 Outlet Type cannot be selected until the Zone 1 Outlet Type is selected because there are some inherent dependencies.

- **Filtration Media with Underdrain –** Only available when the BMP type is a Sand Filter or Rain Garden and when the Zone 1 Volume is WQCV and the Zone 2 Volume is EURV-WQVC. When selected, the program will set several defaults including:
    - Underdrain Orifice Invert Depth and Orifice Diameter (Cells B12:B13) will not be changed in case these values were already entered for Zone 1.
    - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked since they are no longer compatible for use in Zone 3.
    - Elliptical slot axis ratio (Cell B20) is hidden since it is no longer compatible for use in Zone 3.
- **Orifice Plate –** Available for any BMP type as long as the Zone 2 Volume is EURV-WQCV or User Defined. However, if the user selected a Zone 1 Outlet Type of Elliptical Slot or Overflow Weir (No Pipe), the program will reject the Orifice Plate for Zone 2

because these options are not structurally compatible. Otherwise, when selected, the program will set several defaults including:

- Underdrain Orifice Invert Depth and Orifice Diameter (Cells B12:B13) will not be changed in case these values are being utilized for Zone 1.
- Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are unlocked and cleared in case a previous outlet type selection locked them and/or other values are still present in the cells.
- Elliptical slot axis ratio (Cell B20) is hidden since it does not apply to the orifice plate.
- Centroid of Lowest Orifice (Cell B16) will either be set to zero if Zone 1 also utilizes the orifice plate or it will be set to the top stage of Zone 1 (Cell E7) if the Zone 1 outlet type is filtration media with underdrain.
- Depth at top of Zone using Orifice Plate (Cell B17) is set equal to the Estimated Stage for Zone 2 (Cell E8).
- Orifice Vertical Spacing (Cell B18) is set to a default value of one-third the depth of the Zone(s) being drained by the orifice plate. When both Zones 1 and 2 utilize the orifice plate, the depth at the top of Zone 2 (Cell E8) is divided by three and converted to inches. If only Zone 2 drains through the orifice plate, the difference between the Zone 2 and Zone 1 depths (Cell E8 – Cell E7) is divided by three and converted to inches.
- Orifice Area per Row (Cell B19) is cleared and left for the user to determine.
- Stage of Orifice Centroid for Row 1 (Cell B24) is set to the Centroid of Lowest Orifice (Cell B16).
- Stage of Orifice Centroid for Rows 2 and 3 (Cells C24:D24) are then set at stages corresponding to the orifice vertical spacing (Cell B18) and converted to feet.
- The orifice area for Rows 1 through 3 (Cells B25:D25) are set to reference the default orifice area per row (Cell B19) which is currently blank. When the user selects a value for Cell B19, the three rows in the table will automatically be set to the same value.

- **Elliptical Slot –** Available for any BMP type as long as the Zone 2 Volume is EURV-WQCV or User Defined. However, if the user selected a Zone 1 Outlet Type of Orifice Plate or Overflow Weir (No Pipe), the program will reject the Elliptical Slot for Zone 2 because these options are not structurally compatible. Otherwise, when selected, the program will set several defaults including:
  - Underdrain Orifice Invert Depth and Orifice Diameter (Cells B12:B13) will not be changed in case these values are being utilized for Zone 1.
  - Elliptical Slot input cells (B16:B19) are unlocked and cleared in case a previous outlet type selection locked them and/or other values are still present in the cells.
  - Elliptical slot axis ratio (Cell B20) is made visible and the cell description and units are also shown in the adjacent cells.

- Orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked since they do not apply to the elliptical slot.
  - Invert of Elliptical Slot (Cell B16) will either be set to zero if Zone 1 also utilizes the elliptical slot or it will be set to the top stage of Zone 1 (Cell E7) if the Zone 1 outlet type is filtration media with underdrain.
  - Depth at top of Zone using Elliptical Slot (Cell B17) is set equal to the Estimated Stage for Zone 2 (Cell E8).
  - Elliptical Slot Height (Cell B18) is set to a default value that is 0.33 feet below the depth of the zone(s) being drained by the elliptical slot. When both Zones 1 and 2 utilize the elliptical slot, the depth at the top of Zone 2 (Cell E8) minus 0.33 feet is used. If only Zone 2 drains through the elliptical slot, the difference between the Zone 2 and Zone 1 depths (Cell E8 – Cell E7) minus 0.33 feet is used.
  - Elliptical Slot Gap Width (Cell B19) is cleared and left for the user to determine.
  - Elliptical Slot Axis Ratio (Cell B20) is set to a default value of 14.
- **Vertical Orifice (Circular) –** Always available except when the Zone 2 Volume is not selected on the Basin worksheet. However, if the user selected a Zone 1 Outlet Type of Overflow Weir (No Pipe), the program will reject the Vertical Orifice for Zone 2 because these options are not structurally compatible. Otherwise, when selected, the program will set several defaults depending on the outlet type for Zone 1:
  - Underdrain Orifice Invert Depth and Orifice Diameter (Cells B12:B13) will not be changed in case these values are being utilized for Zone 1.
  - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked unless the Zone 1 Outlet Type is an orifice plate or elliptical slot, in which case the existing inputs will remain unchanged.
  - Elliptical slot axis ratio (Cell B20) will be hidden unless the Zone 1 Outlet Type is an elliptical slot, in which case it will remain visible.
  - Invert of Vertical Orifice (Cell B33) is set to the estimated stage of Zone 1 (Cell E7) unless the Zone 1 Outlet Type is also a vertical orifice, in which case the second input column is used to set the Invert of Vertical Orifice (Cell C33) to the estimated stage of Zone 1 (Cell E7).
  - Depth at top of Zone using Vertical Orifice (Cell B34) is set equal to the estimated stage for Zone 2 (Cell E8) unless the Zone 1 Outlet Type is also a vertical orifice, in which case the second input column is used to set the Depth at top of Zone (Cell C34) to the estimated stage of Zone 2 (Cell E8).
  - Vertical Orifice Diameter (Cell B35, or Cell C35 if Zone 1 Outlet Type is also a vertical orifice) is left blank for the user to determine.
  - Vertical Orifice Width (Cell B36, or Cell C36 if Zone 1 Outlet Type is also a vertical orifice) is hidden since it does not apply to the circular orifice.

- **Vertical Orifice (Rectangular) –** Always available except when the Zone 2 Volume is not selected on the Basin worksheet. However, if the user selected a Zone 1 Outlet Type of Overflow Weir (No Pipe), the program will reject the Vertical Orifice for Zone 2 because these options are not structurally compatible. Otherwise, when selected, the program will set several defaults depending on the outlet type for Zone 1:
    - Underdrain Orifice Invert Depth and Orifice Diameter (Cells B12:B13) will not be changed in case these values are being utilized for Zone 1.
    - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked unless the Zone 1 Outlet Type is an orifice plate or elliptical slot, in which case the existing inputs will remain unchanged.
    - Elliptical slot axis ratio (Cell B20) will be hidden unless the Zone 1 Outlet Type is an elliptical slot, in which case it will remain visible.
    - Invert of Vertical Orifice (Cell B33) is set to the estimated stage of Zone 1 (Cell E7) unless the Zone 1 Outlet Type is also a vertical orifice, in which case the second input column is used to set the Invert of Vertical Orifice (Cell C33) to the estimated stage of Zone 1 (Cell E7).
    - Depth at top of Zone using Vertical Orifice (Cell B34) is set equal to the estimated stage for Zone 2 (Cell E8) unless the Zone 1 Outlet Type is also a vertical orifice, in which case the second input column is used to set the Depth at top of Zone (Cell C34) to the estimated stage of Zone 2 (Cell E8).
    - Vertical Orifice Height (Cell B35, or Cell C35 if Zone 1 Outlet Type is also a vertical orifice) is set to a default of 2 inches.
    - Vertical Orifice Width (Cell B36, or Cell C36 if Zone 1 Outlet Type is also a vertical orifice) is made visible since it applies to the rectangular orifice. The cell is left blank for the user to determine.
- **Weir and Pipe (w/ Circular Orifice Plate) –** Always an available option for Zone 2. However, if the user selected a Zone 1 Outlet Type of Overflow Weir (No Pipe), the program will reject the Weir and Pipe for Zone 2 because these options are not structurally compatible. Otherwise, when selected, the program will set several defaults depending on the outlet type for Zone 1:
    - Underdrain Orifice Invert Depth and Orifice Diameter (Cells B12:B13) will not be changed in case these values are being utilized for Zone 1.
    - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked unless the Zone 1 Outlet Type is an orifice plate or elliptical slot, in which case the existing inputs will remain unchanged.
    - Elliptical slot axis ratio (Cell B20) will be hidden unless the Zone 1 Outlet Type is an elliptical slot, in which case it will remain visible.
    - Vertical Orifice input values (Cells B33:B36) are not changed.
    - Overflow Weir Front Edge Height (Cell B40) is set to the estimated stage of Zone 1 (Cell E7) unless the Zone 1 Outlet Type is also a weir and pipe, in which case

    the second input column is used to set the Overflow Weir Front Edge Height (Cell C40) to the estimated stage of Zone 1 (Cell E7).
    - The remaining overflow weir input values (Cells B41:B45, or Cells C41:C45 if Zone 1 Outlet Type is also a weir and pipe) are cleared and left for the user to determine.
    - The outlet pipe input values (Cells B49:B50, or Cells C49:C50 if Zone 1 Outlet Type is also a weir and pipe) are also cleared and left for the user to determine.
    - The third outlet pipe input value (Cell B51, or Cell C51 if Zone 1 Outlet Type is also a weir and pipe) is hidden since it does not apply to the circular orifice.
- **Weir and Pipe (w/ Restrictor Plate) –** Always available except when the Zone 2 Volume is not selected on the Basin worksheet. However, if the user selected a Zone 1 Outlet Type of Overflow Weir (No Pipe), the program will reject the Weir and Pipe for Zone 2 because these options are not structurally compatible. Otherwise, when selected, the program will set several defaults depending on the outlet type for Zone 1:
    - Underdrain Orifice Invert Depth and Orifice Diameter (Cells B12:B13) will not be changed in case these values are being utilized for Zone 1.
    - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked unless the Zone 1 Outlet Type is an orifice plate or elliptical slot, in which case the existing inputs will remain unchanged.
    - Elliptical slot axis ratio (Cell B20) will be hidden unless the Zone 1 Outlet Type is an elliptical slot, in which case it will remain visible.
    - Vertical Orifice input values (Cells B33:B36) are not changed.
    - Overflow Weir Front Edge Height (Cell B40) is set to the estimated stage of Zone 1 (Cell E7) unless the Zone 1 Outlet Type is also a weir and pipe, in which case the second input column is used to set the Overflow Weir Front Edge Height (Cell C40) to the estimated stage of Zone 1 (Cell E7).
    - The remaining overflow weir input values (Cells B41:B45, or Cells C41:C45 if Zone 1 Outlet Type is also a weir and pipe) are cleared and left for the user to determine.
    - The outlet pipe input values (Cells B49:B50, or Cells C49:C50 if Zone 1 Outlet Type is also a weir and pipe) are also cleared and left for the user to determine.
    - Restrictor Plate Height Above Pipe Invert (Cell B51 or Cell C51 if Zone 1 Outlet Type is also a weir and pipe) is made visible since it applies to the circular pipe with restrictor plate. The cell is left blank for the user to determine.
- **Weir and Pipe (w/ Rectangular Orifice Plate) –** Always an available option for Zone 2. However, if the user selected a Zone 1 Outlet Type of Overflow Weir (No Pipe), the program will reject the Weir and Pipe for Zone 2 because these options are not structurally compatible. Otherwise, when selected, the program will set several defaults depending on the outlet type for Zone 1:

- Underdrain Orifice Invert Depth and Orifice Diameter (Cells B12:B13) will not be changed in case these values are being utilized for Zone 1.
- Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked unless the Zone 1 Outlet Type is an orifice plate or elliptical slot, in which case the existing inputs will remain unchanged.
- Elliptical slot axis ratio (Cell B20) will be hidden unless the Zone 1 Outlet Type is an elliptical slot, in which case it will remain visible.
- Vertical Orifice input values (Cells B33:B36) are not changed.
- Overflow Weir Front Edge Height (Cell B40) is set to the estimated stage of Zone 1 (Cell E7) unless the Zone 1 Outlet Type is also a weir and pipe, in which case the second input column is used to set the Overflow Weir Front Edge Height (Cell C40) to the estimated stage of Zone 1 (Cell E7).
- The remaining overflow weir input values (Cells B41:B45, or Cells C41:C45 if Zone 1 Outlet Type is also a weir and pipe) are cleared and left for the user to determine.
- The outlet pipe input values (Cells B49:B50, or Cells C49:C50 if Zone 1 Outlet Type is also a weir and pipe) are also cleared and left for the user to determine.
- Rectangular Orifice Height (Cell B51, or Cell C51 if Zone 1 Outlet Type is also a weir and pipe) is made visible since it applies to the rectangular orifice. The cell is left blank for the user to determine.

- **Overflow Weir (No Pipe) –** Always an available option for Zone 2. When selected, the program will set several defaults including:
  - Underdrain Orifice Invert Depth and Orifice Diameter (Cells B12:B13) will not be changed in case these values are being utilized for Zone 1.
  - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked unless the Zone 1 Outlet Type is an orifice plate or elliptical slot, in which case the existing inputs will remain unchanged.
  - Elliptical slot axis ratio (Cell B20) will be hidden unless the Zone 1 Outlet Type is an elliptical slot, in which case it will remain visible.
  - Vertical Orifice input values (Cells B33:B36) are not changed.
  - Overflow Weir Front Edge Height (Cell B40) is set to the estimated stage of Zone 1 (Cell E7) unless the Zone 1 Outlet Type is also a weir, in which case the second input column is used to set the Overflow Weir Front Edge Height (Cell C40) to the estimated stage of Zone 1 (Cell E7).
  - Overflow Weir Bottom Length and Overflow Weir Side Slopes (Cells B41:B42, or Cells C41:C42 if Zone 1 Outlet Type is also a weir) are cleared and left for the user to determine.
  - The remaining overflow weir input values (B43:B45, or Cells C43:C45 if Zone 1 Outlet Type is also a weir) are set to N/A since they don’t apply when there is no dropbox or overflow grate.

  - If the Zone 1 Outlet Type is also a Weir Only (No Pipe), Cell C46 is made visible and unlocked so that the user can decide whether the second overflow weir overlaps or is offset from the first overflow weir.
  - The outlet pipe input values (Cells B49:B50, or Cells C49:C50 if Zone 1 Outlet Type is also a weir) are also set to N/A since they don't apply when there is no dropbox, overflow grate, or outlet pipe.
  - The third outlet pipe input value (Cell B51, or Cell C51 if Zone 1 Outlet Type is also a weir) is hidden since it does not apply when no pipe is included.
  - Spillway position relative to Overflow Weir (Cell B58) is made visible and unlocked so that the user can decide whether the overflow weir overlaps or is offset from the emergency spillway.
- **Zone 2 Not Utilized –** Always an available option for Zone 2 regardless of the BMP type selected or the Zone volume selections. When selected, the program will set several defaults including:
  - Underdrain Orifice Invert Depth and Orifice Diameter (Cells B12:B13) will not be changed in case these values are being utilized for Zone 1.
  - Orifice plate input cells (B16:B19) and orifice stage/area tables (B24:I25 and B28:I29) are set to N/A and locked unless the Zone 1 Outlet Type is an orifice plate or elliptical slot, in which case the existing inputs will remain unchanged.
  - Elliptical slot axis ratio (Cell B20) will be hidden unless the Zone 1 Outlet Type is an elliptical slot, in which case it will remain visible.
  - Vertical Orifice input values (Cells B33:B36) are not changed.
  - Weir and Pipe input values (Cells B40:B45 and B49:B51) are not changed.

After the program fills in the appropriate default values based on the Outlet Type selected by the user, a check is performed to determine if certain cells need to be hidden. The first check is the position of Weir 2 relative to Weir 1 (Cell C46) which is only shown when both the Zone 1 and Zone 2 outlet types are overflow weirs without pipes. For any other scenario, Cell C46 is hidden. The second check is the position of the emergency spillway relative to an overflow weir without pipe (Cell B58). If neither Zone 1 or Zone 2 include an overflow weir without pipe, then Cell B58 is hidden.

3.2.3 Zone 3 Outlet Type

The following outlet types are potential options for the Zone 3 Outlet Type pulldown. Beside each option is a description of when the outlet type is available to be selected and which default values will be populated by the program. The Zone 3 Outlet Type cannot be selected until both the Zone 1 and Zone 2 Outlet Types are selected because there are some inherent dependencies.

- **Vertical Orifice (Circular) –** Always available as long as both the Zone 2 and Zone 3 Volumes are selected on the Basin worksheet. However, if the user already selected a Vertical Orifice for both the Zone 1 and Zone 2 Outlet Types, the program will reject the

Vertical Orifice for Zone 3 because the workbook can only evaluate two independent vertical orifices due to a limitation of the workbook setup. Similarly, if the user selected a Zone 2 Outlet Type of Overflow Weir (No Pipe), the program will reject the Vertical Orifice for Zone 3 because these options are not structurally compatible. Otherwise, when selected, the program will set several defaults depending on the outlet types for Zone 1 and Zone 2:

  - Invert of Vertical Orifice (Cell B33) is set to the estimated stage of Zone 2 (Cell E8) unless the Zone 1 or Zone 2 Outlet Type is also a vertical orifice, in which case the second input column is used to set the Invert of Vertical Orifice (Cell C33) to the estimated stage of Zone 2 (Cell E8).
  - Depth at top of Zone using Vertical Orifice (Cell B34) is set equal to the estimated stage for Zone 3 (Cell E9) unless the Zone 1 or Zone 2 Outlet Type is also a vertical orifice, in which case the second input column is used to set the Depth at top of Zone (Cell C34) to the estimated stage of Zone 3 (Cell E9).
  - Vertical Orifice Diameter (Cell B35, or Cell C35 if Zone 1 or Zone 2 Outlet Type is also a vertical orifice) is left blank for the user to determine.
  - Vertical Orifice Width (Cell B36, or Cell C36 if Zone 1 or Zone 2 Outlet Type is also a vertical orifice) is hidden since it does not apply to the circular orifice.

- **Vertical Orifice (Rectangular) –** Always available as long as both the Zone 2 and Zone 3 Volumes are selected on the Basin worksheet. However, if the user already selected a Vertical Orifice for both the Zone 1 and Zone 2 Outlet Types, the program will reject the Vertical Orifice for Zone 3 because the workbook can only evaluate two independent vertical orifices due to a limitation of the workbook setup. Similarly, if the user selected a Zone 2 Outlet Type of Overflow Weir (No Pipe), the program will reject the Vertical Orifice for Zone 3 because these options are not structurally compatible. Otherwise, when selected, the program will set several defaults depending on the outlet types for Zone 1 and Zone 2:
  - Invert of Vertical Orifice (Cell B33) is set to the estimated stage of Zone 2 (Cell E8) unless the Zone 1 or Zone 2 Outlet Type is also a vertical orifice, in which case the second input column is used to set the Invert of Vertical Orifice (Cell C33) to the estimated stage of Zone 2 (Cell E8).
  - Depth at top of Zone using Vertical Orifice (Cell B34) is set equal to the estimated stage for Zone 3 (Cell E9) unless the Zone 1 or Zone 2 Outlet Type is also a vertical orifice, in which case the second input column is used to set the Depth at top of Zone (Cell C34) to the estimated stage of Zone 3 (Cell E9).
  - Vertical Orifice Height (Cell B35, or Cell C35 if Zone 1 or Zone 2 Outlet Type is also a vertical orifice) is set to a default of 2 inches.
  - Vertical Orifice Width (Cell B36, or Cell C36 if Zone 1 or Zone 2 Outlet Type is also a vertical orifice) is made visible since it applies to the rectangular orifice. The cell is left blank for the user to determine.

- **Weir and Pipe (w/ Circular Orifice Plate) –** Always available except when neither the Zone 2 or Zone 3 Volumes are selected on the Basin worksheet. However, if the user already selected a Weir and Pipe for both the Zone 1 and Zone 2 Outlet Types, the program will reject the Weir and Pipe for Zone 3 because the workbook can only evaluate two independent weir and pipe configurations due to a limitation of the workbook setup. Similarly, if the user selected a Zone 2 Outlet Type of Overflow Weir (No Pipe), the program will reject the Weir and Pipe for Zone 3 because these options are not structurally compatible. Otherwise, when selected, the program will set several defaults depending on the outlet types for Zone 1 and Zone 2:
    - Overflow Weir Front Edge Height (Cell B40) is set to the estimated stage of Zone 2 (Cell E8) unless the Zone 1 or Zone 2 Outlet Type is also a weir and pipe, in which case the second input column is used to set the Overflow Weir Front Edge Height (Cell C40) to the estimated stage of Zone 2 (Cell E8).
    - The remaining overflow weir input values (Cells B41:B45, or Cells C41:C45 if the Zone 1 or Zone 2 Outlet Type is also a weir and pipe) are cleared and left for the user to determine.
    - The outlet pipe input values (Cells B49:B50, or Cells C49:C50 if the Zone 1 or Zone 2 Outlet Type is also a weir and pipe) are also cleared and left for the user to determine.
    - The third outlet pipe input value (Cell B51, or Cell C51 if the Zone 1 or Zone 2 Outlet Type is also a weir and pipe) is hidden since it does not apply to the circular orifice.
- **Weir and Pipe (w/ Restrictor Plate) –** Always available as long as both the Zone 2 and Zone 3 Volumes are selected on the Basin worksheet. However, if the user already selected a Weir and Pipe for both the Zone 1 and Zone 2 Outlet Types, the program will reject the Weir and Pipe for Zone 3 because the workbook can only evaluate two independent weir and pipe configurations due to a limitation of the workbook setup. Similarly, if the user selected a Zone 2 Outlet Type of Overflow Weir (No Pipe), the program will reject the Weir and Pipe for Zone 3 because these options are not structurally compatible. Otherwise, when selected, the program will set several defaults depending on the outlet types for Zone 1 and Zone 2:
    - Overflow Weir Front Edge Height (Cell B40) is set to the estimated stage of Zone 2 (Cell E8) unless the Zone 1 or Zone 2 Outlet Type is also a weir and pipe, in which case the second input column is used to set the Overflow Weir Front Edge Height (Cell C40) to the estimated stage of Zone 2 (Cell E8).
    - The remaining overflow weir input values (Cells B41:B45, or Cells C41:C45 if the Zone 1 or Zone 2 Outlet Type is also a weir and pipe) are cleared and left for the user to determine.

- The outlet pipe input values (Cells B49:B50, or Cells C49:C50 if the Zone 1 or Zone 2 Outlet Type is also a weir and pipe) are also cleared and left for the user to determine.
- Restrictor Plate Height Above Pipe Invert (Cell B51, or Cell C51 if the Zone 1 or Zone 2 Outlet Type is also a weir and pipe) is made visible since it applies to the circular pipe with restrictor plate. The cell is left blank for the user to determine

- **Weir and Pipe (w/ Rectangular Orifice Plate) –** Always available except when neither the Zone 2 or Zone 3 Volumes are selected on the Basin worksheet. However, if the user already selected a Weir and Pipe for both the Zone 1 and Zone 2 Outlet Types, the program will reject the Weir and Pipe for Zone 3 because the workbook can only evaluate two independent weir and pipe configurations due to a limitation of the workbook setup. Similarly, if the user selected a Zone 2 Outlet Type of Overflow Weir (No Pipe), the program will reject the Weir and Pipe for Zone 3 because these options are not structurally compatible. Otherwise, when selected, the program will set several defaults depending on the outlet types for Zone 1 and Zone 2:
  - Overflow Weir Front Edge Height (Cell B40) is set to the estimated stage of Zone 2 (Cell E8) unless the Zone 1 or Zone 2 Outlet Type is also a weir and pipe, in which case the second input column is used to set the Overflow Weir Front Edge Height (Cell C40) to the estimated stage of Zone 2 (Cell E8).
  - The remaining overflow weir input values (Cells B41:B45, or Cells C41:C45 if the Zone 1 or Zone 2 Outlet Type is also a weir and pipe) are cleared and left for the user to determine.
  - The outlet pipe input values (Cells B49:B50, or Cells C49:C50 if the Zone 1 or Zone 2 Outlet Type is also a weir and pipe) are also cleared and left for the user to determine.
  - Rectangular Orifice Height (Cell B51, or Cell C51 if the Zone 1 or Zone 2 Outlet Type is also a weir and pipe) is made visible since it applies to the rectangular orifice. The cell is left blank for the user to determine.
- **Overflow Weir (No Pipe) –** Always available except when neither the Zone 2 or Zone 3 Volumes are selected on the Basin worksheet. However, if the user already selected an Overflow Weir (with or without Outlet Pipe) for both the Zone 1 and Zone 2 Outlet Types, the program will reject the Overflow Weir (No Pipe) for Zone 3 because the workbook can only evaluate two independent weir configurations due to a limitation of the workbook setup. Otherwise, when selected, the program will set several defaults depending on the outlet types for Zone 1 and Zone 2:
  - Overflow Weir Front Edge Height (Cell B40) is set to the estimated stage of Zone 2 (Cell E8) unless the Zone 1 or Zone 2 Outlet Type is also a weir, in which case the second input column is used to set the Overflow Weir Front Edge Height (Cell C40) to the estimated stage of Zone 2 (Cell E8).

- Overflow Weir Bottom Length and Overflow Weir Side Slopes (Cells B41:B42, or Cells C41:C42 if the Zone 1 or Zone 2 Outlet Type is also a weir) are cleared and left for the user to determine.
- The remaining overflow weir input values (B43:B45, or Cells C43:C45 if the Zone 1 or Zone 2 Outlet Type is also a weir) are set to N/A since they don't apply when there is no dropbox or overflow grate.
- If the Zone 1 or Zone 2 Outlet Type is also a Weir Only (No Pipe), Cell C46 is made visible and unlocked so that the user can decide whether the second overflow weir overlaps or is offset from the first overflow weir.
- The outlet pipe input values (Cells B49:B50, or Cells C49:C50 if the Zone 1 or Zone 2 Outlet Type is also a weir) are also set to N/A since they don't apply when there is no dropbox, overflow grate, or outlet pipe.
- The third outlet pipe input value (Cell B51, or Cell C51 if the Zone 1 or Zone 2 Outlet Type is also a weir) is hidden since it does not apply when no pipe is included.
- Spillway position relative to Overflow Weir (Cell B58) is made visible and unlocked so that the user can decide whether the overflow weir overlaps or is offset from the emergency spillway.

* **Zone 3 Not Utilized –** Always an available option for Zone 3. There are no default input values set when this option is selected.

After the program has finished filling in the default input values based on the Zone 3 Outlet Type selected, the program will then proceed to fill in any remaining blank input values with N/A in an attempt to eliminate confusion regarding empty input cells. The program starts by checking to see if either of the vertical orifice outlets were selected. If one vertical orifice was selected, the program will set the second column (Cells C33:C35) to N/A and hide cell C36. If no vertical orifice was selected, the program will set both columns (Cells B33:C35) to N/A and hide Cells B36:C36. Next the program checks to see if either of the Weir and Pipe outlets were selected. If one weir and pipe configuration was selected, the program will set the second column of inputs (Cells C40:C45 and C49:C50) to N/A and hide Cells C46 and C51. If no Weir and Pipe configurations were selected, the program will set both columns of inputs (Cells B40:C45 and B49:C50) to N/A and hide Cells B46:C46 and B51:C51. It should be noted, that the user can still override many of the input cells populated with N/A to provide additional outlets beyond the three selected from the Outlet Type pulldown lists.

The program will also perform a check to determine if certain cells need to be hidden. The first check is the position of Weir 2 relative to Weir 1 (Cell C46) which is only shown when there are two zones drained by overflow weirs without pipes. For any other scenario, Cell C46 is hidden. The second check is the position of the emergency spillway relative to an overflow weir without pipe (Cell B58). If none of the zones include an overflow weir without pipe, then Cell B58 is hidden.

3.3 Orifice at Underdrain Outlet

The next section on the Outlet Structure worksheet provides user input cells for the Orifice at Underdrain Outlet which is typically used to drain the WQCV in a filtration BMP. This section is only applicable if a Sand Filter or Rain Garden (Bioretention) are selected as the BMP Type on the Basin worksheet. Figure 3.2 shows the section for the Underdrain Outlet.

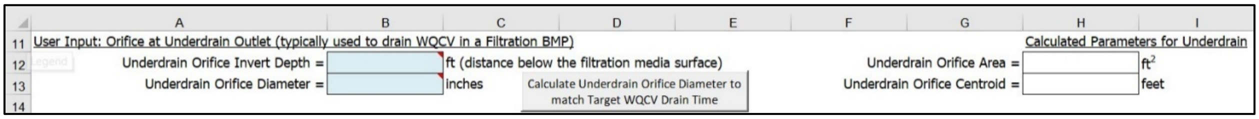


Figure 3.2 - Orifice at Underdrain Outlet

The user input values for the underdrain section include:

- **Underdrain Orifice Invert Depth (feet)** is the vertical distance from the filtration media surface to the invert of the underdrain outlet orifice. This must be a positive number.
- **Underdrain Orifice Diameter (inches)** is the diameter of the orifice restricting flow from the underdrain. This workbook assumes that orifice flow controls the release rate, not the infiltration rate through the filtration media. For full infiltration sections, the user can size the orifice based on the expected infiltration rate of the native soils.

Two calculated parameters for the underdrain outlet orifice are shown to the right of the input values. These values are required for use in other parts of the workbook discussed later. The calculated parameters include:

- **Underdrain Orifice Area (square feet)** is calculated as the area of a circle with the user input diameter (Cell B13). The result is then converted from square inches to square feet.
- **Underdrain Orifice Centroid (feet)** is half of the orifice diameter (Cell B13) converted from inches to feet.

The stage-discharge relationship for the Underdrain Outlet Orifice is calculated in Cells G87:G3087 using the orifice discharge equation shown below.

$$Q = C_d * A * \sqrt{2gh}$$

Where:

Q = orifice discharge (cubic feet per second)

C_d = discharge coefficient of 0.6

A = orifice area (square feet) from Cell H12

g = gravitational constant of 32.2 (feet per square second)

h = depth of water above the orifice centroid (feet)

The depth of water is calculated for each row as the stage for the current row (Column B) plus the underdrain orifice invert depth below the filtration media surface (Cell B12) minus the underdrain orifice centroid height (Cell H13).

As seen in Figure 3.2, there is a sizing button that is only visible when the Zone 1 Volume is WQCV and the Zone 1 Outlet Type is Filtration Media with Underdrain. In all other scenarios, this button is hidden from the user. When clicked the button will calculate the underdrain orifice diameter required to match the target WQCV drain time. In order for the sizing routine to begin, the user must at a minimum, provide the underdrain orifice invert depth (Cell B12). The sizing routine starts by checking to see if a starting orifice diameter (Cell B13) was provided, if not a value of 1-inch is used as a starting point in the sizing routine. Then, the program iteratively increases or decreases the orifice diameter (0.01-feet increments) as necessary until the calculated WQCV drain time (Cell B75) matches the target drain time (Cell B21 on the Basin worksheet). If a solution cannot be found, a message will pop up and suggest the user try to decrease the orifice invert depth (Cell B12). The calculated WQCV drain time (Cell B75) is determined using the Modified Puls routing method to determine the time interval when 99% of the WQCV brim full capacity has drained through the underdrain outlet orifice (storage volume at end of time interval is calculated in Cells J3091:J4531).

3.4 Orifice Plate

The next section on the Outlet Structure worksheet provides user input cells for the Orifice Plate or Elliptical Slot Weir, which are typically used to drain the WQCV (and/or EURV) in a sedimentation BMP. Figure 3.3 shows the orifice plate section of the worksheet (elliptical slot weir will be discussed in Section 3.5). The orifice plate section is available when Zone 1 is the WQCV or EURV-WQCV (for EDB, RP, or CWP) or when Zone 2 is the EURV-WQCV (for EDB, RP, CWP, SF, or RG).

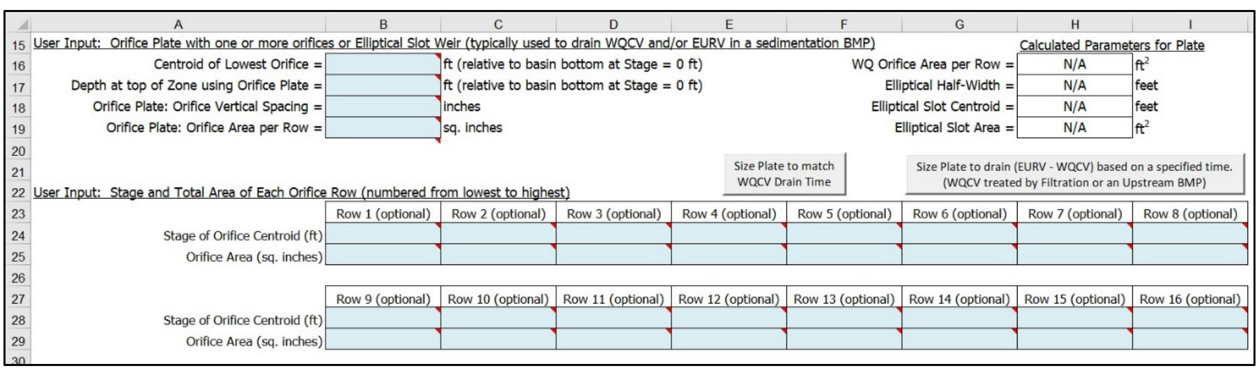


Figure 3.3 – Orifice Plate

The user input values for the orifice plate section include:

- **Centroid of Lowest Orifice (feet)** is measured relative to the basin bottom at a stage of zero feet. The default value selected by the program is zero for an EDB, RP, or CWP. The program will set the default value equal to the estimated stage for Zone 1 (Cell E7) for a SF or RG when Zone 1 drains through the filtration media and the orifice plate is only used to drain the EURV-WQCV in Zone 2. The user can override the default value but a negative value is not acceptable.
- **Depth at Top of Zone Using Orifice Plate (feet)** is also measured relative to the basin bottom at a stage of zero feet. When the orifice plate is used to only drain the Zone 1 volume (e.g., WQCV), the default depth selected by the program is the estimated stage for Zone 1 (Cell E7). If the orifice plate drains both Zones 1 and 2, (e.g., WQCV and EURV), the default depth selected by the program is the estimated stage for Zone 2 (Cell E8). The user can override the default value but a negative value is not acceptable.
- **Orifice Vertical Spacing (inches)** is the vertical distance between each orifice opening cut into the orifice plate, measured from centroid to centroid. The MHFD recommends a maximum of three orifices since several smaller orifices may result in more frequent clogging. The default vertical spacing set by the program is equal to the depth of the zone or zones draining through the orifice plate divided by three. The user can override the default vertical spacing in this cell which will automatically update the orifice centroid stage for each row in the table (Cells B24:I24 and Cells B28:I28). If the user changes stage values in the table and the spacing is not consistent between each row, Cell B18 will be set to N/A.
- **Orifice Area per Row (square inches)** represents the open area of a circular or rectangular orifice. To minimize clogging, the minimum acceptable area is 0.12 square inches. To avoid smaller orifices, increase the vertical spacing in Cell B18 or reduce the number of orifices. When the user enters a number in this cell, all rows in the orifice plate will have the same orifice area and the values in the table below (Cells B25:I25 and Cells B29:I29) will automatically be updated. The user can also override individual orifice areas for each row (including entering zero to block a row entirely) in the table below. If different rows have different areas, Cell B19 will be set to N/A.
- **Stage of Orifice Centroid (feet)** for each row of the orifice plate can be entered in Cells B24:I24 (rows 1-8) and Cells B28:I28 (rows 9-16). The program sets the default stage for Row 1 equal to the centroid of lowest orifice (Cell B16) which is typically zero. The default stage for Row 2 is then set equal to 1/3 of the zone depth being drained by the orifice plate. The default stage for Row 3 is set equal to 2/3 of the zone depth being drained by the orifice plate. The stage for all of the other rows is left blank since the MHFD recommends only 3 orifice openings in the plate. The user can override the stage values for each row but if the spacing between rows is not consistent, Cell B18 will be set to N/A.

- **Orifice Area (square inches)** for each row of the orifice plate can be entered in Cells B25:I25 (rows 1-8) and Cells B29:I29 (rows 9-16). The program sets the default area for each row equal to Cell B19. The user can override the orifice area for each row but if the areas are not all the same, Cell B19 will be set to N/A.

To the right of the input cells are four cells showing calculated parameters. The WQ Orifice Area per Row (square feet) in Cell H16 is the value in Cell B19 converted from square inches to square feet. The remaining three calculated parameters (Cells H17:H19) are set to N/A since they only apply to the elliptical slot weir.

The stage-discharge relationship for the Orifice Plate is calculated in Cells AB87:AR3087. Each row of the orifice plate is calculated in a separate column (e.g., Row 1 is calculated in Cells AB87:AB3087 and Row 2 is calculated in Cells AC87:AC3087). There are 16 columns representing the 16 separate orifice rows that can be input by the user. Each column (AB through AQ) is calculated using the orifice discharge equation shown below.

$$Q = C_d * A * \sqrt{2gh}$$

Where:

Q = orifice discharge (cubic feet per second)

C_d = discharge coefficient of 0.6

A = orifice area (square feet) from Cells B25:I25 and B29:I29 (converted from sq. in.)

g = gravitational constant of 32.2 (feet per square second)

h = depth of water above the orifice centroid (feet)

The depth of water is calculated for each row in the spreadsheet as the Stage for the current row (Column B) minus the corresponding orifice centroid stage (Cells B24:I24 and Cells B28:I28). Once the stage-discharge relationship is calculated for each row in the orifice plate (Cells AB87:AQ3087), the combined stage-discharge relationship is calculated in Cells AR87:AR3087 by summing the 16 columns. Since an orifice plate is being used, Cells H87:H3087 reference the values in Column AQ, as opposed to the elliptical slot weir values in Column BC.

As seen in Figure 3.3, there are two sizing buttons that may be visible depending on the zone volumes and outlet types selected by the user.

3.4.1 Size Plate to Match WQCV Drain Time

The *Size Plate to Match WQCV Drain Time* button is only visible when the Zone 1 Volume is WQCV and the Zone 1 Outlet Type is an Orifice Plate or Elliptical Slot Weir. In all other scenarios, this button is hidden from the user.

When the button is clicked for an orifice plate, the program will calculate the orifice area per row (Cell B19) required to match the target WQCV drain time. In order for the sizing routine to begin, the user must at a minimum, provide the centroid of lowest orifice (Cell B16), depth at top of zone using orifice plate (Cell B17), and the orifice vertical spacing (Cell B18). The sizing routine starts by checking to see if a starting orifice area per row (Cell B19) was provided. If not provided, a value of 0.12 square inches (minimum acceptable value) is used as a starting point in the sizing routine.

Then, the program iteratively increases the orifice area per row (0.01-feet increments) as necessary until the calculated WQCV drain time (Cell B75) matches the target drain time (Cell B21 on the Basin worksheet). If a solution cannot be found, a message will pop up and suggest the user try to manually change the orifice area and vertical spacing for each row or resize the basin geometry. The calculated WQCV drain time (Cell B75) is determined using the Modified Puls routing method to determine the time interval when 99% of the WQCV brim full capacity has drained through the orifice plate (storage volume at end of time interval is calculated in Cells J3091:J4531).

3.4.2 Size Plate to drain (EURV – WQCV) based on a Specified Time

The *Size Plate to drain (EURV – WQCV) based on a specified time* button is only visible for two scenarios. The first scenario is when the Zone 1 volume is WQCV and it drains through filtration media and the Zone 2 volume is EURV-WQCV and it drains through an orifice plate or elliptical slot weir. The second scenario is when the Zone 1 volume is EURV – WQCV and it drains through an orifice plate or elliptical slot weir. In the second scenario, the WQCV is provided in an upstream BMP and the user is required to provide their own inflow hydrographs for the current workbook to reflect the drain time of the WQCV from the upstream BMP. In all other scenarios, this button is hidden from the user.

When the button is clicked, the program will ask the user to select a target drain time for the EURV – WQCV. The program will provide an acceptable range with a minimum of 12 hours and a maximum based on the BMP Type selected and the associated WQCV drain time, so that the combined drain time does not exceed 72 hours. For example, an EDB has a WQCV drain time of 40 hours, so the acceptable range for the EURV-WQCV drain time would be 12 to 32 hours. Similarly, a SF has a WQCV drain time of 12 hours, so the acceptable range for the EURV-WQCV drain time would be 12 to 60 hours. For the first scenario described above (WQCV through filtration media), the target EURV drain time would be the sum of the WQCV drain time and the EURV – WQCV drain time (e.g., 52 to 72 hours for an EDB or 24 to 72 hours for a SF). However, in the second scenario (WQCV in upstream BMP) the target EURV drain time would be equal to the user-entered EURV-WQCV drain time.

The program will then calculate the orifice area per row (Cell B19) required to match the target EURV drain time. In order for the sizing routine to begin, the user must at a minimum, provide the centroid of lowest orifice (Cell B16), depth at top of zone using orifice plate (Cell B17), and the orifice vertical spacing (Cell B18). The sizing routine starts by checking to see if a starting orifice area per row (Cell B19) was provided. If not provided, a value of 0.12 square inches (minimum acceptable value) is used as a starting point in the sizing routine.

Then, the program iteratively increases the orifice area per row (0.01-feet increments) as necessary until the calculated EURV drain time (Cell C75) matches the target EURV drain time. If a solution cannot be found, a message will pop up and suggest the user try to manually change the orifice area and vertical spacing for each row or resize the basin geometry. The calculated EURV drain time (Cell C75) is determined using the Modified Puls routing method to determine the time interval when 99% of the EURV brim full capacity has drained through the orifice plate (storage volume at end of time interval is calculated in Cells AA3091:AA4531).

3.5 Elliptical Slot Weir

The Orifice Plate section on the Outlet Structure worksheet is also used for the less common Elliptical Slot Weir, which is similarly used to drain the WQCV (and/or EURV) in a sedimentation BMP. The elliptical slot weir serves as an alternative to the orifice plate for larger watersheds (generally larger than 60 acres). If the user selects Elliptical Slot for the Zone 1 and/or Zone 2 Outlet Type, the user input cell descriptions will be modified as shown in Figure 3.4. Documentation supporting the derivation of the elliptical slot weir equations based on physical model studies is located in *Detention Basin Alternative Outlet Design Study* provided in Appendix E.

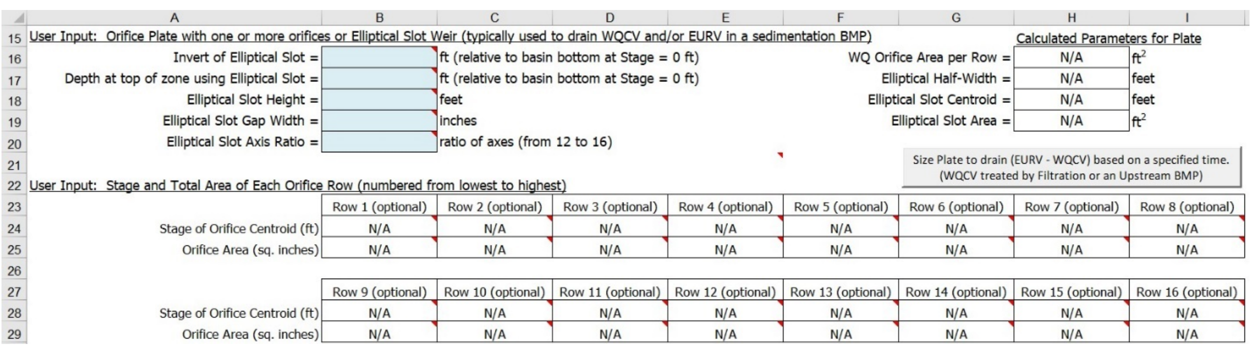


Figure 3.4 – Elliptical Slot Weir

The user input values for the elliptical slot weir section include:

- **Invert of Elliptical Slot (feet)** is measured relative to the basin bottom at a stage of zero feet. The default value selected by the program is zero for an EDB, RP, or CWP. The program will set the default value equal to the estimated stage for Zone 1 (Cell E7) for a SF or RG when Zone 1 drains through the filtration media and the elliptical slot weir is only used to drain the EURV-WQCV in Zone 2. The user can override the default value but a negative value is not acceptable.

- **Depth at top of zone using Elliptical Slot (feet)** is also measured relative to the basin bottom at a stage of zero feet. When the elliptical slot weir is used to only drain the Zone 1 volume (e.g., WQCV), the default depth selected by the program is the estimated stage for Zone 1 (Cell E7). If the elliptical slot weir drains both Zones 1 and 2, (e.g., WQCV and EURV), the default depth selected by the program is the estimated stage for Zone 2 (Cell E8). The user can override the default value but a negative value is not acceptable.
- **Elliptical Slot Height (feet)** is typically 3 to 4 inches less than the depth of the zone being drained. This allows 3 to 4 inches between the top of the slot and the top of the grate for the support structure. The program sets the default value at 4 inches below the top of the zone. The user can override this height but it must be greater than zero.
- **Elliptical Slot Gap Width (inches)** is the width at the bottom of the elliptical slot, shown in Figure 3.5. To minimize clogging, the minimum acceptable slot width is 0.375 inches. If this minimum slot width results in too slow a drain time, the elliptical slot is not appropriate for the application. The program does not set a default width, this value can be adjusted by the user to modify drain times.
- **Elliptical Slot Axis Ratio** is the ratio of the semi-major ellipse axis in the vertical direction (H in Figure 3.5) to the semi-minor ellipse axis in the horizontal direction (W in Figure 3.5). The MHFD tested elliptical slots having major-to-minor axis ratios from 12 to 16 and the flow equations shown in Figure 3.5 are calibrated within this range. The program sets the default axis ratio to 14, but this value can be changed by the user within the range of 12 to 16. This input value (Cell B20) is only visible for the elliptical slot weir and is hidden when the orifice plate is selected.

WQ Elliptical Slot Weir (Alternative to WQ Orifice Plate for Large Watersheds)

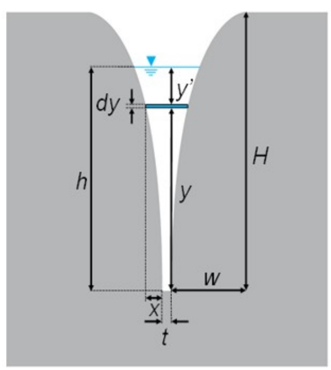
$$f(y)=\sqrt{2g(h-y)}\left[2\frac{H}{R}\left(1-1\sqrt{1-\frac{y^2}{H^2}}\right)+t\right]$$

$$Q_{app}=0.3015h\left[f(0)+f(0.603h)\right]+0.1415h\left[f(0.603h)+f(0.886h)\right]+0.0570h\left[f(0.886h)\right]$$

Orifice Equations

$$Q=C_dA_t\sqrt{2g(h-C_y)}$$

$$A_t=H(2W+t)-\frac{\pi HW}{2} \qquad C_y=\frac{\frac{H}{2}(2W+t)-\frac{2HW}{3}}{2W+t-\frac{\pi W}{2}}$$



- *dy* = elementary flow vertical distance [L];
- *h* = total flow depth [L];
- *H* = total weir height and semi-major ellipse axis [L];
- *t* = weir gap thickness [L];
- *W* = semi-minor ellipse axis [L];
- *x* = horizontal distance along the ellipse shape [L];
- *y* = vertical depth measured from the weir crest to the elementary flow strip [L]; and
- *y'* = vertical distance measured from the water surface to the elementary flow strip [L].

Figure 3.5 – Elliptical Slot Weir Equations

To the right of the input cells are four cells showing calculated parameters.

- **WQ Orifice Area per Row (square feet)** in Cell H16 only applies to the orifice plate and is not applicable for the elliptical slot weir.

- **Elliptical Half Width (feet)** in Cell H17 is the width of the elliptical slot at half of the slot height and is calculated as the elliptical slot height (Cell B18) divided by the elliptical slot axis ratio (Cell B20).
- **Elliptical Slot Centroid (feet)** is the centroid of the elliptical slot open area and is calculated using the C_y equation in the lower left corner of Figure 3.5.
- **Elliptical Slot Area (square feet)** is the open area of the elliptical slot and is calculated using the A_t equation in the lower left corner of Figure 3.5.

The stage-discharge relationship for the elliptical slot weir is calculated in Cells AS87:BC3087. The elliptical slot weir discharge equation is a theoretical rating equation integrated using the trapezoidal approximation method. The function equation f(y) for approximation of the discharge integral is shown in the top left corner of Figure 3.5 and described in the report provided in Appendix E. Through an optimization analysis comparing the implicit integral solution to the explicit trapezoidal approximation, the optimal intervals for the trapezoidal approximation were determined to be 0 to 0.603, 0.603 to 0.886, and 0.886 to 1.000 times the flow depth through the weir. The (y) values at each of these optimal intervals are calculated for each row in the stage-discharge table of the Outlet Structure worksheet in Cells AS87:AV3087. The calculated (y) values along with the elliptical slot height (Cell B18), gap width (Cell B19), and axis ratio (Cell B20) are plugged into the function equations f(y) for each row in the stage-discharge table (Cells AW87:AZ3087). The function equation f(y) values are then plugged into the simplified expression for the elliptical slot weir discharge equation (Q_{app}) as shown on the middle-left side of Figure 3.5. The elliptical slot weir equation (Cells BA87:BA3087) applies to all rows of the stage-discharge table where the water surface is below the top height of the weir. When the water rises above the top of the weir, the elliptical slot discharge is calculated using the orifice equation (Cells BB87:BB3087) as shown in the lower left corner of Figure 3.5. The final column in the stage-discharge table calculations (Cells BC87:BC3087) selects the appropriate discharge value from the weir or orifice column. Since an elliptical slot weir is being used, Cells H87:H3087 reference the values in Column BC, as opposed to the orifice plate values in Column AQ.

As seen in Figure 3.4, there are two sizing buttons that may be visible depending on the zone volumes and outlet types selected by the user.

3.5.1 Size Plate to Match WQCV Drain Time

The *Size Plate to Match WQCV Drain Time* button is only visible when the Zone 1 Volume is WQCV and the Zone 1 Outlet Type is an Orifice Plate or Elliptical Slot Weir. In all other scenarios, this button is hidden from the user.

When the button is clicked for an elliptical slot, the program will calculate the elliptical slot gap width (Cell B19) required to match the target WQCV drain time. In order for the sizing routine to begin, the user must at a minimum, provide the invert of elliptical slot (Cell B16), depth at top of zone using elliptical slot (Cell B17), elliptical slot height (Cell B18), and elliptical slot axis ratio (Cell B20). The sizing routine starts by checking to see if a starting elliptical slot gap width (Cell B19) was provided. If not provided, a value of 0.38 inches (minimum acceptable value) is used as a starting point in the sizing routine.

Then, the program iteratively increases the elliptical slot gap width (0.01-feet increments) as necessary until the calculated WQCV drain time (Cell B75) matches the target drain time (Cell B21 on the Basin worksheet). If a solution with a gap width greater than 0.38-inches cannot be found, a message will pop up notifying the user that the elliptical slot weir is not appropriate and the program will automatically switch the outlet type to an orifice plate and begin automatically sizing the orifice plate to match the drain time as described in Section 3.4. The calculated WQCV drain time (Cell B75) is determined using the Modified Puls routing method to determine the time interval when 99% of the WQCV brim full capacity has drained through the elliptical slot (storage volume at end of time interval is calculated in Cells J3091:J4531)

3.5.2 Size Plate to drain (EURV – WQCV) based on a Specified Time

The *Size Plate to drain (EURV – WQCV) based on a specified time* button is only visible for two scenarios. The first scenario is when the Zone 1 volume is WQCV and it drains through filtration media and the Zone 2 volume is EURV-WQCV and it drains through an orifice plate or elliptical slot weir. The second scenario is when the Zone 1 volume is EURV – WQCV and it drains through an orifice plate or elliptical slot weir. In the second scenario, the WQCV is provided in an upstream BMP and the user is required to provide their own inflow hydrographs for the current workbook to reflect the drain time of the WQCV from the upstream BMP. In all other scenarios, this button is hidden from the user.

When the button is clicked, the program will ask the user to select a target drain time for the EURV – WQCV. The program will provide an acceptable range with a minimum of 12 hours and a maximum based on the BMP Type selected and the associated WQCV drain time, so that the combined drain time does not exceed 72 hours. For example, an EDB has a WQCV drain time of 40 hours, so the acceptable range for the EURV-WQCV drain time would be 12 to 32 hours. Similarly, a SF has a WQCV drain time of 12 hours, so the acceptable range for the EURV-WQCV drain time would be 12 to 60 hours. For the first scenario described above (WQCV through filtration media), the target EURV drain time would be the sum of the WQCV drain time and the EURV – WQCV drain time (e.g., 52 to 72 hours for an EDB or 24 to 72 hours for a SF). However, in the second scenario (WQCV in upstream BMP) the target EURV drain time would be equal to the user-entered EURV-WQCV drain time.

The program will then calculate the elliptical slot gap width (Cell B19) required to match the target EURV drain time. In order for the sizing routine to begin, the user must at a minimum, provide the invert of elliptical slot (Cell B16), depth at top of zone using elliptical slot (Cell B17), elliptical slot height (Cell B18), and elliptical slot axis ratio (Cell B20). The sizing routine starts by checking to see if a starting elliptical slot gap width (Cell B19) was provided. If not provided, a value of 0.38 inches (minimum acceptable value) is used as a starting point in the sizing routine.

Then, the program iteratively increases the elliptical slot gap width (0.01-feet increments) as necessary until the calculated EURV drain time (Cell C75) matches the target EURV drain time. If a solution with a gap width greater than 0.38-inches cannot be found, a message will pop up notifying the user that the elliptical slot weir is not appropriate and the program will automatically switch the outlet type to an orifice plate and begin automatically sizing the orifice plate to match the drain time as described in Section 3.4. The calculated EURV drain time (Cell C75) is determined using the Modified Puls routing method to determine the time interval when 99% of the EURV brim full capacity has drained through the elliptical slot (storage volume at end of time interval is calculated in Cells AA3091:AA4531).

3.6 Vertical Orifice

The next section on the Outlet Structure worksheet provides user input cells for up to two separate vertical orifices which can be either circular or rectangular (Columns B:C). Figure 3.6 shows the vertical orifice section of the worksheet when a circular orifice is selected for Zone 1 and a rectangular orifice is selected for Zone 2. The vertical orifice section is almost always available for use, even when not selected as the outlet type for one of the three zones. For example, if the user selects filtration media for Zones 1 and 2 and an overflow weir and pipe for Zone 3, the user could still include a vertical orifice in the front wall of the overflow weir dropbox and the program would account for this additional discharge through the outlet structure.

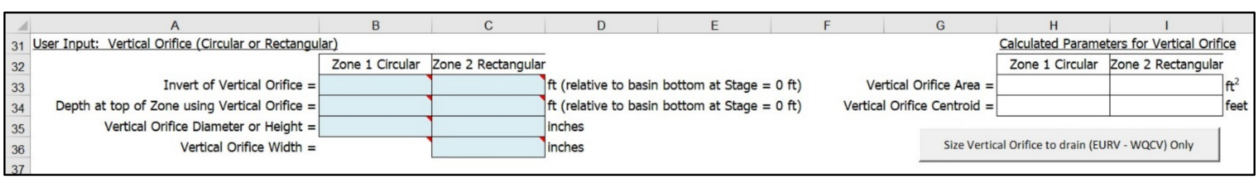


Figure 3.6 – Vertical Orifice

The user input values for the vertical orifice section include:

- **Invert of Vertical Orifice (feet)** is measured relative to the basin bottom at a stage of zero feet. If the user selects vertical orifice for one of the zone outlet types, the default invert value is set equal to the estimated top stage for the zone below (Cell E7 or E8) or set equal to zero for Zone 1. The user can override the default invert value but a negative value is not acceptable. If the outlet structure includes two separate vertical orifice openings, both columns (B:C) can be utilized. The invert for the second orifice (Cell C33) must be greater than the invert for the first orifice (Cell B33).
- **Depth at Top of Zone Using Vertical Orifice (feet)** is also measured relative to the basin bottom at a stage of zero feet. When the vertical orifice is used to drain the Zone 1

volume, the default depth selected by the program is the estimated stage for Zone 1 (Cell E7). Similarly, if the vertical orifice is used to drain Zone 2 or 3, the default depth selected by the program is the estimated stage for Zone 2 (Cell E8) or Zone 3 (Cell E9), respectively. The user can override the default value but a negative value is not acceptable. If the outlet structure includes two separate vertical orifice openings, both columns (B:C) can be utilized. The top depth for the second orifice (Cell C34) must be greater than the top depth for the first orifice (Cell B34).

- **Vertical Orifice Diameter or Height (inches)** is either the orifice diameter for a circular orifice or the orifice height for a rectangular orifice. The input cell description (Cell A35) will change depending on whether a circular orifice, rectangular orifice, or both are selected. If a circular orifice is selected by the user, the program will leave the input cell blank. The minimum recommended circular orifice diameter is 3/8-inch to prevent clogging, but the user can provide any value greater than zero. If a rectangular orifice is selected, the program will set a default orifice height of 2-inches. The user can override the default value but a value greater than zero must be entered. If the outlet structure includes two separate vertical orifice openings, both columns (B:C) can be utilized and the input values will reflect either the orifice diameter or height depending on whether circular or rectangular was selected for that column.
- **Vertical Orifice Width (inches)** is only visible when a rectangular orifice is selected. If the outlet structure includes two separate vertical orifice openings, and one or both of the columns (B:C) represent a circular orifice, the corresponding input value (Cell B36 and/or C36) will be hidden. If a rectangular orifice is selected by the user, the program will show the corresponding input cell, but will leave the orifice width blank and the user must enter a width greater than zero.

To the right of the vertical orifice input cells are four cells showing calculated parameters (two cells for each vertical orifice column). Cells H33:H34 correspond to the first vertical orifice input in Cells B33:B36. Cells I33:I34 correspond to the second vertical orifice input in Cells C33:C36.

- **Vertical Orifice Area (square feet)** is the calculated open area for the vertical orifice. For a circular orifice, the area is calculated using the equation $A = \frac{\pi D^2}{4}$ (where D is the orifice diameter in Cell B35 or C35). The resulting area is then converted from square inches to square feet by dividing by 144. For a rectangular orifice, the area is calculated by multiplying the orifice height (Cell B35 or C35) by the orifice width (Cell B36 or C36) and then converting from square inches to square feet by dividing by 144.
- **Vertical Orifice Centroid (feet)** is calculated as half of the orifice diameter or orifice height, depending on whether a circular or rectangular orifice was selected. The resulting centroid height is then converted to feet by dividing by 12.

The stage-discharge relationship for the first vertical orifice is calculated in Cells I87:I3087. The stage-discharge relationship for the second vertical orifice is calculated in the next column (Cells J87:J3087). The vertical orifice discharge equation varies depending on whether the current water surface elevation is above or below the top of the orifice opening.

When the water surface is above the top of the orifice opening, the standard orifice discharge equation is used as shown below.

$$Q = C_d * A * \sqrt{2gh}$$

Where:

> Q = orifice discharge (cubic feet per second)
>
> C_d = discharge coefficient of 0.6
>
> A = orifice area (square feet) from Cell H33 or Cell I33
>
> g = gravitational constant of 32.2 (feet per square second)
>
> h = ponding depth of water above the orifice centroid (feet)

The depth of water above the orifice centroid is calculated as the water stage for the current row (Column B) minus both the corresponding orifice invert stage (Cell B33 or Cell C33) and the corresponding centroid height (Cell H34 or Cell I34). Again, this equation only applies when the water surface is above the top (crown) of the orifice opening.

When the water surface falls below the top of the orifice opening, the standard orifice equation no longer applies and an empirical equation for estimating flow through a partially submerged vertical orifice is used. The derivation of the empirical equation is documented in a Technical Memorandum entitled *Estimating Flow through a Partially Submerged Vertical Orifice*, dated December 31, 2015 (Appendix F). This technical memorandum is based on a technical paper titled *Flow Through Partially Submerged Orifice* in the ASCE Journal of Irrigation and Drainage Engineering by Guo, Stitt and Mays, dated December 1, 2015 (Appendix G).

The empirical equation for flow through a partially submerged vertical orifice used in the workbook is based on the proportional relationship between depth and discharge for flow in partially full pipes. This relationship was then fit to the results in the technical paper by Guo and Stitt which was verified in the University of Colorado Hydraulics Lab. The resulting equation showing the proportional relationship between depth and discharge with a best fit exponent to the technical paper results is shown below.

$$Q = Q_{full}\left(\frac{y}{D}\right)^{1.81}$$

Where:

Q = partially submerged orifice discharge (cubic feet per second)

Q_{full} = orifice discharge when water depth equals top of orifice (cubic feet per second)

y = ponding depth ranging from orifice invert to top of orifice opening (feet)

D = diameter or height of orifice (feet)

The full flow orifice discharge (Q_{full}) is calculated using the standard orifice discharge equation shown previously, where the ponding depth (h) is set equal to the top of the orifice opening minus the centroid height, the discharge coefficient (C_d) equals 0.6, and the orifice area (A) is provided in Cell H33 or I33. This empirical equation allows the program to solve for orifice discharge from the top of the orifice all the way down to the invert, similar to the method used for partially full pipes.

As seen in Figure 3.6, there is a sizing button that may be visible depending on the zone volumes and outlet types selected by the user.

3.6.1 Size Vertical Orifice to drain (EURV – WQCV) Only

The *Size Vertical Orifice to drain (EURV – WQCV) Only* button is only visible for three different scenarios. The first scenario is when the Zone 1 volume is WQCV and it drains through filtration media and the Zone 2 volume is EURV-WQCV and it drains through a vertical orifice (circular or rectangular). The second scenario is when the Zone 1 volume is WQCV and it drains through an orifice plate or elliptical slot and the Zone 2 volume is EURV-WQCV and it drains through a vertical orifice (circular or rectangular). The third scenario is when the Zone 1 volume is EURV – WQCV and it drains through a vertical orifice (circular or rectangular). In the third scenario, the WQCV is provided in an upstream BMP and the user is required to provide their own inflow hydrographs for the current workbook to reflect the drain time of the WQCV from the upstream BMP. In all other scenarios, this button is hidden from the user.

When the button is clicked, the program will ask the user to select a target drain time for the EURV – WQCV. The program will provide an acceptable range with a minimum of 12 hours and a maximum based on the BMP Type selected and the associated WQCV drain time, so that the combined drain time does not exceed 72 hours. For example, an EDB has a WQCV drain time of 40 hours, so the acceptable range for the EURV-WQCV drain time would be 12 to 32 hours. Similarly, a SF has a WQCV drain time of 12 hours, so the acceptable range for the EURV-WQCV drain time would be 12 to 60 hours. For the first and second scenarios described above (WQCV through filtration media, orifice plate, or elliptical slot), the target EURV drain time would be the sum of the WQCV drain time and the EURV – WQCV drain time (e.g., 52 to 72 hours for an EDB or 24 to 72 hours for a SF). However, in the third scenario (WQCV in upstream BMP) the target EURV drain time would be equal to the user-entered EURV-WQCV drain time.

The program will then calculate either the vertical orifice diameter (Cell B35) for a circular orifice or the vertical orifice width (Cell B36) for a rectangular orifice to match the target EURV drain time. The calculated EURV drain time (Cell C75) is determined using the Modified Puls routing method to determine the time interval when 99% of the EURV brim full capacity has drained through the vertical orifice (storage volume at end of time interval is calculated in Cells AA3091:AA4531). In order for the sizing routine to begin, the user must at a minimum, provide the invert of the vertical orifice (Cell B33) and depth at top of zone using vertical orifice (Cell B34). When sizing a rectangular orifice, the user must also provide the vertical orifice height (Cell B35).

For a circular orifice, the sizing routine starts by checking to see if a starting vertical orifice diameter (Cell B35) was provided. If not provided, a value of 3/8-inch (minimum acceptable value) is used as a starting point in the sizing routine. Then, the program iteratively increases or decreases the vertical orifice diameter (0.01-feet increments) as necessary until the calculated EURV drain time (Cell C75) matches the target EURV drain time. If the program cannot find a solution with an orifice diameter greater than 3/8-inch, a message will notify the user to consider changing the orifice size manually or resizing the basin geometry. If the program determines that an orifice diameter greater than 144 inches is required, a message will notify the user to try switching to a rectangular orifice with a shallow height and wider width, or if utilizing filtration media or a water quality plate to drain the WQCV, try using that outlet component to help match the target EURV drain time.

For a rectangular orifice, the sizing routine starts by checking to see if a starting vertical orifice width (Cell B36) was provided. If not provided, a value of 2-inches is used as a starting point in the sizing routine. Then, the program iteratively increases or decreases the vertical orifice width (0.01-feet increments) as necessary until the calculated EURV drain time (Cell C75) matches the target EURV drain time. If the program cannot find a solution with an orifice width greater than 3/8-inch, a message will notify the user that a width smaller than 3/8-inch will easily clog, and the program will automatically switch to a circular orifice and start the sizing routine described above. On the other hand, if the program determines that an orifice width greater than 144 inches is required, a message will notify the user that a solution could not be found and that if utilizing filtration media or a water quality plate to drain the WQCV, to try using that outlet component to help match the target EURV drain time.

3.7 Overflow Weir

The next section on the Outlet Structure worksheet provides user input cells for up to two separate overflow weirs (Columns B:C). Figure 3.7 shows the overflow weir section of the worksheet with all of the potential input cells and the automated sizing button visible for purposes of describing the various scenarios available in the workbook. The overflow weir section is always available for use, even when not selected as the outlet type for one of the three zones. For example, if the user selects filtration media or an orifice plate for Zones 1 and 2 and a vertical orifice for Zone 3, the user can still include an overflow weir (with or without outlet pipe) above the vertical orifice and the program will account for this additional discharge as part of the combined outlet structure.

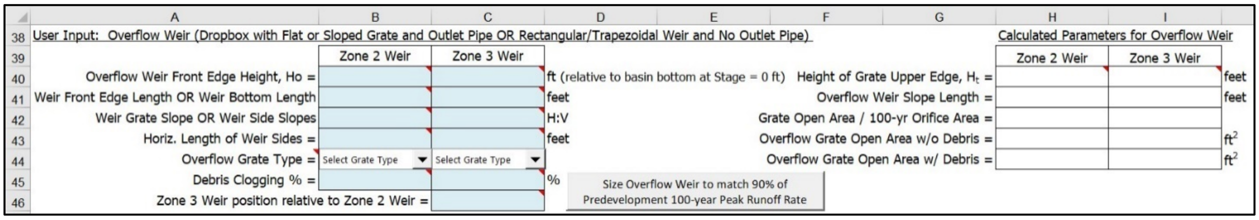


Figure 3.7 – Overflow Weir

Each of the input columns (B:C) can represent one of three different overflow weir configurations. The first overflow weir configuration is a dropbox structure with a flat or sloped grate on top that allows water to spill over the weir through the grate and down into the dropbox structure where it is discharged through an outlet pipe as shown in Figure 3.8. The second overflow weir option is a rectangular or trapezoidal weir where water spills directly into a dropbox structure without a grate and is discharged through an outlet pipe. The third option is a simple rectangular or trapezoidal overflow weir which acts as a spillway (no dropbox or outlet pipe are included in this option).

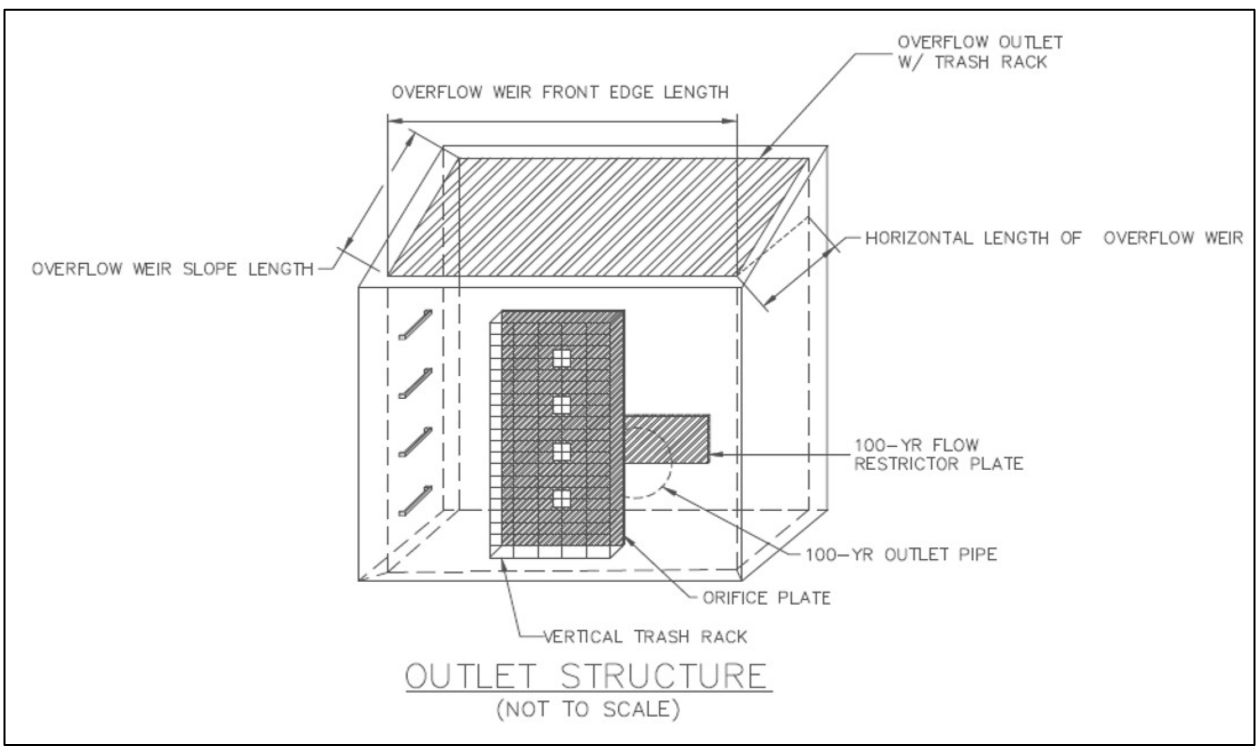


Figure 3.8 –Overflow Weir with Grate

The user input values for the overflow weir section include:

- **Overflow Weir Front Edge Height (feet)** is measured relative to the basin bottom at a stage of zero feet. If the user selects overflow weir for one of the three zone outlet types, the default invert value is set equal to the estimated top stage for the zone below (Cell E7 or E8) or set equal to zero when selected for Zone 1. The user can override the default invert value but a negative value is not acceptable. If the outlet structure includes two separate overflow weirs, both columns (B:C) can be utilized. The invert for the second weir (Cell C40) must be greater than the invert for the first weir (Cell B40).
- **Weir Front Edge Length OR Bottom Length (feet)** is dependent on the type of overflow weir being evaluated. The input cell description (Cell A41) will change depending on whether a dropbox with grate, rectangular/trapezoidal weir, or both are selected. If a dropbox with grate is being used, this input cell represents the front inside edge length of the dropbox opening as seen in Figure 3.8. If the overflow weir acts as a rectangular or trapezoidal weir, this input cell represents the bottom length of the overflow weir (perpendicular to the flow direction). For a dropbox with outlet pipe configuration, it is recommended that the user start with a large length in this cell to ensure that the outlet pipe discharge is not affected by choking of flow at the overflow weir. After properly sizing the outlet pipe, the user can come back to this cell and reduce the weir length as appropriate. The user can enter any value except a negative value which is not acceptable. If the outlet structure includes two separate overflow weirs, both columns (B:C) can be utilized and the input values will reflect either the front edge length or bottom length depending on what is being evaluated in that column.
- **Weir Grate Slope OR Weir Side Slopes (H:V)** is also dependent on the type of overflow weir being evaluated. The input cell description (Cell A42) will change depending on whether a dropbox with grate, rectangular/trapezoidal weir, or both are selected. If a dropbox with grate is used, this input cell represents the slope of the overflow grate expressed as horizontal length to vertical height (e.g., a 4H:1V slope is entered as 4). The weir grate slope must be greater than or equal to 3 based on physical model testing conducted by the US Bureau of Reclamation. The only other acceptable value is zero which represents a flat grate. If the overflow weir acts as a rectangular or trapezoidal weir, this input cell represents the side slopes of a trapezoidal weir (e.g., 4H:1V) or the vertical sides a rectangular weir (slope equals zero). If the outlet structure includes two separate overflow weirs, both columns (B:C) can be utilized and the input values will reflect either the grate slope or weir side slope depending on what is being evaluated in that column.
- **Horizontal Length of Weir Sides (feet)** is only applicable when a dropbox with outlet pipe is being evaluated. When the dropbox includes a flat or sloping grate, the horizontal length (not the length along the grate slope) is measured on the inside edge of the dropbox as seen in Figure 3.8. For example, if the grate covers a 4-ft by 4-ft dropbox opening, the horizontal length of the weir is 4-feet, regardless of the grate slope. If a side length of zero is entered by the user, the overflow weir is treated as a rectangular or

trapezoidal weir that spills directly into the dropbox without a grate. In both cases, water that enters the dropbox will be discharged downstream through an outlet pipe. If there is no dropbox or outlet pipe and the rectangular/trapezoidal weir is acting as a spillway, the side length will be set to "N/A" by the program. If the outlet structure includes two separate overflow weirs, both columns (B:C) can be utilized and the input values will reflect the type of weir being evaluated in that column.

- **Overflow Grate Type** is selected from a pulldown list which includes three options to describe the type of overflow grate installed on top of the dropbox. The grate type is not applicable to rectangular/trapezoidal weirs and should be set to "N/A" if not already done so by the program. Pictures and figures of the three grate types available are shown in Figure 3.9. These options are based on physical modeling conducted by the US Bureau of Reclamation and documented in a report titled *Physical Modeling of Overflow Outlets for Extended Detention Stormwater Basins* dated September 2014 (Appendix H). If "No Grate" is selected, the cell will be flagged due to public safety concerns. The effective open area for each grate type is as follows:
    - CDOT Type C Grate (Bar Grate) = 0.70
    - Close Mesh Grate = 0.79
    - No Grate (Open Grate) = 1.00

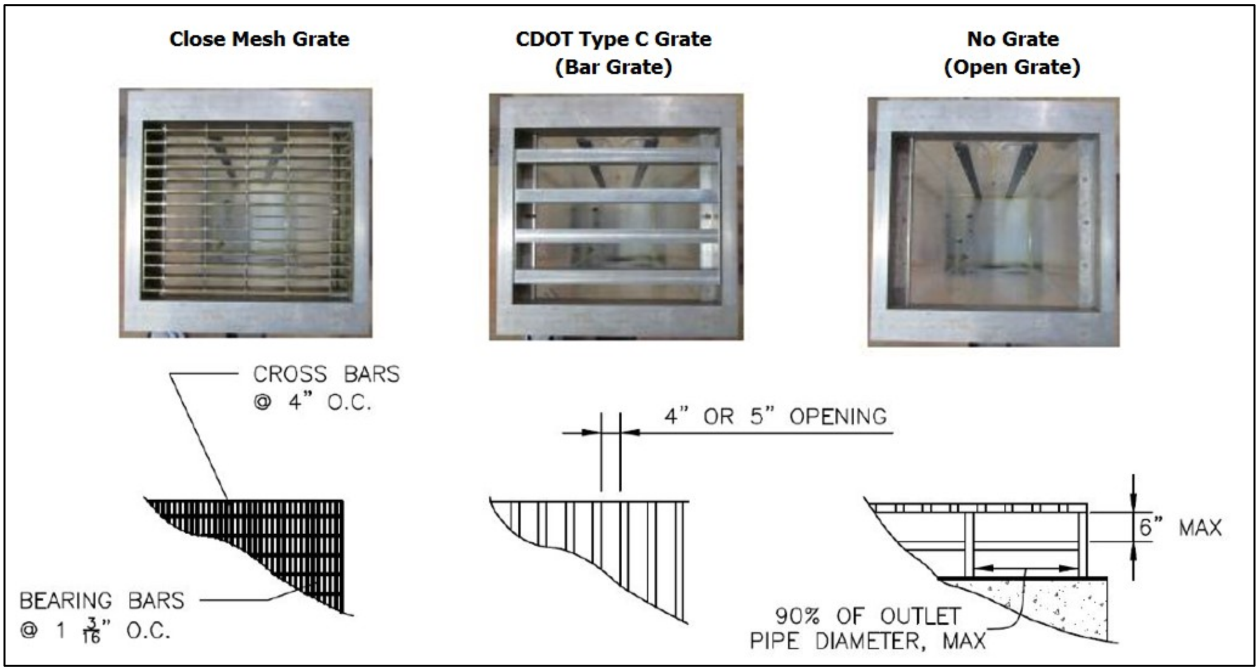


Figure 3.9 – Overflow Grate Types

- **Debris Clogging Percent (%)** allows the user to evaluate the impacts of debris clogging on the grate capacity. Increasing the debris clogging percentage reduces the effective open area of the grate. Debris clogging is not applicable to rectangular/trapezoidal weirs and should be set to "N/A" if not already done so by the program. The intent of this input value is to ensure that a clogged grate condition does not control storm events with a specific discharge target (e.g., 100-year allowable release rate). MHFD recommends using 0% debris clogging to size for targeted release rates, drain times, and preparing a stage-discharge table. MHFD also recommends testing the outlet structure design using 50% debris clogging to ensure there is still adequate capacity through the outlet structure in the clogged condition. The user can check row 71 of the Routed Hydrograph Results table to ensure clogged conditions are not controlling targeted release rates.

- **Upper Weir Position Relative to Lower Weir** in Cells B46:C46 is only visible when the user selects two separate overflow weirs without pipes. For all other outlet configurations, Cells B46:C46 are hidden and locked. In Figure 3.7, Zones 2 and 3 were both selected as Overflow Weir (No Pipe) and so the upper weir drains Zone 3 and the lower weir drains Zone 2. When visible, the user can select offset or overlapping from the pulldown list in Cell C46 to indicate the upper overflow weir position relative to the lower overflow weir. This is done in order to properly account for the flow through separate and offset weirs (versus flow through an overlapping two-stage weir). This ensures that the workbook does not double count flow through overlapping weir sections.

To the right of the overflow weir input cells are ten cells showing calculated parameters (five cells for each overflow weir column) that are later used in the program code or hidden table calculations. Cells H40:H44 correspond to the first set of overflow weir input values in Cells B40:B45. Cells I40:I44 correspond to the second set of overflow weir input values in Cells C40:C45.

- **Height of Grate Upper Edge (feet)** is calculated to determine the stage at the back of the overflow weir grate for sloping grates. If the grate slope is zero, the back edge height is equal to the weir front edge height (Cell B42 or C42). If the grate is sloped, the grate upper edge height is equal to the weir front edge height (Cell B40 or C40) plus the product of the grate slope (Cell B42 or C42) and the horizontal length of the side (Cell B43 or C43). If there is no grate associated with the overflow weir, this cell is set to N/A.
- **Overflow Weir Slope Length (feet)** is calculated to determine the length of the grate along the grate slope as shown in Figure 3.8. If the grate slope is zero, the slope length is equal to the horizontal length of the weir side (Cell B43 or C43). For slopes greater than zero, the slope length is calculated as a function of the grate slope and horizontal length using the following equation.

$$L_{slope} = L_{horizontal}/\cos\left(\tan^{-1}\left(\frac{1}{Slope}\right)\right)$$

Where:

L_{slope} = overflow weir slope length (feet)

$L_{horizontal}$ = horizontal length of weir sides (feet) in Cell B43 or C43

Slope = overflow weir grate slope (H:V) in Cell B42 or C42

- **Grate Open Area / 100-yr Orifice Area** compares the effective open area of the overflow weir grate relative to the open area of the outlet pipe orifice. The effective open area of the grate is explained in the next bullet point (Cell H43 or I43). The open area of the outlet pipe orifice is explained in the next section (Cell H49 or I49).
- **Overflow Grate Open Area without Debris (square feet)** is calculated to determine the effective open area of the grate by subtracting the area covered by the grate bars. The grate open area is calculated by multiplying the front edge length of the grate (Cell B41 or C41) by the overflow weir slope length (Cell H41 or I41) to get the total grate area, then multiplying this value by the effective open area for the selected grate type (Cell B44 or C44). The effective open area for the different grates ranges from 0.70 to 1.00.
- **Overflow Grate Open Area with Debris (square feet)** accounts for the user input debris clogging percentage to reduce the effective open area for the grate. It is calculated by multiplying the grate open area without debris (Cell H43 or I43) by the complementary debris clogging percentage (percentage not clogged with debris calculated as 100% minus Cell B45 or C45).

The stage-discharge relationship for the first overflow weir is calculated using four columns (Cells K87:N3087). The stage-discharge relationship for the second overflow weir is calculated in the next four columns (Cells O87:R3087). The four columns for each overflow weir are used to calculate the estimated discharge using different methods. The first column (K or O) estimates discharge using a weir equation for shallow water depths. The second column (L or P) estimates discharge using an orifice equation for deeper water depths. The third column (M or Q) estimates discharge using a mixed flow equation which reflects the transition zone between weir flow and orifice flow. The fourth column (N or R) then selects the minimum discharge estimate from the previous three columns as the controlling discharge for the given water depth in each row of the stage-discharge table. Each of the three discharge methods are discussed in more detail below.

The first method for estimating discharge through the overflow weir (Columns K and O) is based on a set of weir equations that depend on the type of overflow weir configuration and the depth of water relative to the weir height. Regardless of the overflow weir configuration, if the depth of water is below the weir front edge height (Cell B40 or C40), the discharge is zero.

For a dropbox with grate configuration, equations documented in the US Bureau of Reclamation report titled *Physical Modeling of Overflow Outlets for Extended Detention Stormwater Basins* dated September 2014 (Appendix H) are used to estimate weir discharge. These equations are summarized in Figure 3.10 for a flat weir, a sloped un-submerged weir, and a sloped submerged weir. The flat weir equation accounts for inflow from all four sides of the dropbox structure. The sloped weir equations only account for inflow from the front and sides of the dropbox since the back edge is typically installed flush with the embankment.

Flat Weir	$Q_{Flat} = \frac{2}{3} n C_d (2B + 2L)\sqrt{2g} H^{\frac{3}{2}}$
Sloped Un-Submerged Weir ($H < H_b$)	$Q_{WS} = \frac{4}{15} n C_d \sqrt{2g} \cot(\theta)\, H^{\frac{5}{2}}$ $Q_{WB} = \frac{2}{3} n C_d \sqrt{2g} B H^{\frac{3}{2}}$ $\boldsymbol{Q_W = 2Q_{WS} + Q_{WB}}$
Sloped Submerged Weir ($H \geq H_b$)	$Q_{WS} = \frac{4}{15} n C_d \sqrt{2g} L \cos(\theta) \left[\frac{H^{\frac{5}{2}} - (H - H_b)^{\frac{5}{2}}}{H_b} \right]$ $Q_{WB} = \frac{2}{3} n C_d \sqrt{2g} B H^{\frac{3}{2}}$ $\boldsymbol{Q_W = 2Q_{WS} + Q_{WB}}$

Figure 3.10 – Overflow Weir Discharge Equations

Where:

Q_{Flat} = total weir discharge for flat dropbox and grate (cubic feet per second)

Q_W = total weir discharge for sloped dropbox and grate (cubic feet per second)

Q_{WS} = weir discharge through side of sloped dropbox and grate (cubic feet per second)

Q_{WB} = weir discharge through front of sloped dropbox and grate (cubic feet per second)

B = weir front edge length (feet) in Cell B41 or C41

L = horizontal length of weir sides (feet) in Cell B43 or C43

θ = angle of inclined grate (radians) calculated from grate slope in Cell B42 or C42

H = headwater depth above weir front edge height (feet)

H_b = height of the top of upper edge of the grate above weir front edge height (feet)

g = gravitational constant of 32.2 (feet per square second)

n = open area ratio for grate based on grate type selected in Cell B44 or C44

C_d = weir discharge coefficient

The product of the grate open area ratio and weir discharge coefficient (n^*C_d) is calculated in the MHFD-Detention workbook using an empirical equation that was developed by MHFD to provide a best fit to the USBR physical model data discussed in Appendix H. The empirical equation accounts for both the grate slope and the effective open area of the grate. It should be noted that the coefficient values provided in Table 5 of Appendix H are no longer used and have been replaced with the following 2nd order equation where θ is the angle of the grate in radians and the coefficients a, b, and c were determined by regression and shown in Table 3.11.

$$n * C_d = a\theta^2 + b\theta + c$$

	Open Area, n	a	b	c
No Grate	**1.00**	-1.9748	0.7493	0.6319
Close Mesh Grate	**0.79**	-1.3509	0.3950	0.6210
Type C Grate	**0.70**	-1.9736	0.6689	0.6022

Figure 3.11 – Overflow Weir Discharge Equation Coefficients

The MHFD-Detention workbook also accounts for the user input debris clogging percentage (Cell B45 or C45) when calculating the weir discharge by multiplying the result from the equations above by the complementary debris clogging percentage.

When the overflow weir configuration is designed to function as a rectangular or trapezoidal weir (horizontal length of weir sides in Cell B43 or C43 equals zero or N/A), the above equations do not apply and a broad-crested weir equation is used to calculate discharge as discussed in Section 5.14.2 of the USDCM Storage Chapter. In order to calculate the total flow over a trapezoidal weir, the results from equation 12-8 (discharge through rectangular weir) are added to two times the result from equation 12-9 (discharge through sloping weir on one side). The resulting equation used in the MHFD-Detention workbook is shown below. If the side slopes are zero, the second term in the equation becomes zero and you are left with the rectangular weir equation.

$$Q = C_{BCW} L H^{1.5} + 2\left(\frac{2}{5}\right) C_{BCW} Z H^{2.5}$$

Where:

Q = total weir discharge (cubic feet per second)

C_{BCW} = broad-crested weir coefficient set equal to 3.0

L = weir bottom length (feet) in Cell B41 or C41

Z = weir side slopes (H:V) in Cell B42 or C42

H = headwater depth above weir front edge height (feet)

If the user selects two overflow weirs designed to function as rectangular or trapezoidal weirs (horizontal length of weir sides in Cell B43 equals N/A and in Cell C43 equals zero or N/A), then the user must indicate whether these weirs are offset or overlapping in Cell C46. If the two weirs are offset, then the discharge through each weir is calculated independently using the equations described above. However, if the two weirs overlap, then the program will subtract the discharge through the overlapping section from the second weir discharge (Cells O87:O3087) to avoid double counting the flow through this overlapping section.

The second method for estimating discharge through the overflow weir (Columns L and P) is based on a set of orifice equations that depend on the type of overflow weir configuration and the depth of water relative to the weir crest height. The orifice equations only apply to a dropbox with grate configuration and the discharge is set equal to zero if the depth of water is below the weir front edge height (Cell B40 or C40). The discharge also equals zero for the entire column if the overflow weir is designed to function as a rectangular or trapezoidal weir.

For a dropbox with grate configuration, equations documented in the US Bureau of Reclamation report titled *Physical Modeling of Overflow Outlets for Extended Detention Stormwater Basins* dated September 2014 (Appendix H) are used to estimate orifice discharge. These equations are summarized in Figure 3.12 for a flat orifice, a sloped un-submerged orifice, and a sloped submerged orifice.

Flat Orifice	$Q_O = \frac{2}{3} n C_d B L \sqrt{2gH}$
Sloped Un-Submerged Orifice ($H < H_b$)	$Q_O = \frac{2}{3} n C_d B H \cot(\theta) \sqrt{2gH}$
Sloped Submerged Orifice ($H \geq H_b$)	$Q_O = \frac{2}{3} n C_d B L \cos(\theta) \sqrt{2gH} \left[\frac{H^{\frac{3}{2}} - (H - H_b)^{\frac{3}{2}}}{H_b \sqrt{H}} \right]$

Figure 3.12 – Overflow Orifice Discharge Equations

Where:

Q_O = orifice discharge through dropbox and grate (cubic feet per second)

B = weir front edge length (feet) in Cell B41 or C41

L = horizontal length of weir sides (feet) in Cell B43 or C43

θ = angle of inclined grate (radians) calculated from grate slope in Cell B42 or C42

H = headwater depth above weir front edge height (feet)

H_b = height of the top of upper edge of the grate above weir front edge height (feet)

g = gravitational constant of 32.2 (feet per square second)

n = open area ratio for grate based on grate type selected in Cell B44 or C44

C_d = orifice discharge coefficient

The product of the grate open area ratio and orifice discharge coefficient ($n*C_d$) is calculated in the MHFD-Detention workbook using an empirical equation that was developed by MHFD to provide a best fit to the USBR physical model data discussed in Appendix H. The empirical equation accounts for both the grate slope and the effective open area of the grate. The orifice discharge coefficient is calculated using the following 2nd order equation where θ is the angle of the grate in radians and the coefficients a, b, and c were determined by regression and shown in Table 3.11.

$$n * C_d = a\theta^2 + b\theta + c$$

	Open Area, n	a	b	c
No Grate	**1.00**	6.9486	-2.8042	0.9710
Close Mesh Grate	**0.79**	4.6080	-2.4401	0.9560
Type C Grate	**0.70**	3.8611	-1.9835	0.7372

Figure 3.13 – Overflow Orifice Discharge Equation Coefficients

The MHFD-Detention workbook also accounts for the user input debris clogging percentage (Cell B45 or C45) when calculating the orifice discharge by multiplying the result from the equations above by the complementary debris clogging percentage.

The third method for estimating discharge through the overflow weir (Columns M and Q) is to evaluate the transition zone between weir flow at low headwater depths and orifice flow at high headwater depths. This transition zone at intermediate headwater depths is commonly referred to as mixed flow. The US Bureau of Reclamation report titled *Physical Modeling of Overflow Outlets for Extended Detention Stormwater Basins* dated September 2014 (Appendix H) discussed the mixed flow zone and how the physical model observations of stage became unstable and would fluctuate significantly with a constant inflow. Therefore, the MHFD-Detention workbook uses an empirical equation that was developed by MHFD to provide a best fit to the USBR physical model data discussed in Appendix H. Mixed flow is calculated using the following empirical equation.

$$Q_{Mixed} = Q_{Weir} + Q_{Orifice} - 1.11\sqrt{Q_{Weir} * Q_{Orifice}}$$

The controlling discharge through the overflow weir (Columns N and R) is then set equal to the minimum result from the three calculation methods described above for weir flow, orifice flow, and mixed flow.

As seen in Figure 3.7, there is a sizing button that may be visible depending on the zone volumes and outlet types selected by the user.

3.7.1 Size Overflow Weir to Predevelopment 100-year Peak Runoff Rate

The *Size Overflow Weir to match 90% of Predevelopment 100-year Peak Runoff Rate* button is only visible when the top storage zone is drained by a rectangular or trapezoidal overflow weir (no pipe or dropbox included). The top storage zone can be Zone 3 when Zone 1 drains through filter media or a water quality plate and Zone 2 drains through filter media, a water quality plate, or a vertical orifice. The top storage zone can be Zone 2 when Zone 1 drains through filter media, a water quality plate, or a vertical orifice plate. The top zone can also be Zone 1. In all other scenarios, this button is hidden from the user.

When the button is clicked, the program will calculate the overflow weir bottom length (Cell B41) required to control the 100-year peak outflow to 90% of the predevelopment 100-year peak runoff rate as calculated in Cell I70. The 100-year peak outflow (Cell I69) is determined using the Modified Puls routing method to route the 100-year inflow hydrograph through the basin and outlet structure (outflow at end of time interval is calculated in Cells DU3091:DU4531). In order for the sizing routine to begin, the user must provide the overflow weir front edge height (Cell B40), overflow weir bottom length (Cell B41) and overflow weir side slope (Cell B42). The horizontal length of weir sides (Cell B43), overflow grate type (Cell B44), and debris clogging percentage (Cell B45) must all be set to N/A.

The sizing routine starts by setting the emergency spillway invert at a depth of 999 feet to ensure it does not interfere with the overflow weir sizing. Then, the program iteratively increases or decreases the weir bottom length (0.01-feet increments) as necessary until the ratio of the 100-year peak outflow to predevelopment runoff rate (Cell I70) equals 0.90. If the program cannot find a solution with a bottom length greater than zero, a message will notify the user that the program can switch from a trapezoidal weir to a rectangular weir by setting the side slopes to zero. The program will then repeat the process of iteratively increasing or decreasing the bottom length to try and achieve a value of 0.90 in Cell I70. If the program still cannot find a solution with a bottom length greater than zero, a message will notify the user that they can try to manually adjust the input parameters to find the problem. However, if at either step the program does find a valid solution, a message will notify the user that the overflow weir has been sized to control the 100-year release rate. The program will then ask the user if they would like to have the overflow weir also function as the emergency spillway for larger events or if they would like to have a separate emergency spillway. If a separate emergency spillway is desired, the program will start sizing the emergency spillway to pass the developed 100-year peak runoff rate as discussed in a later section of this manual.

3.8 Outlet Pipe with Flow Restriction Plate

The next section on the Outlet Structure worksheet provides user input cells for up to two separate outlet pipes with flow restriction plates (Columns B:C). Figure 3.14 shows the outlet pipe section of the worksheet with potential input cells and the automated sizing buttons visible for purposes of describing the various scenarios available in the workbook. The outlet pipe section is always available for use, even when not selected as the outlet type for one of the three zones. For example, if the user selects filtration media or an orifice plate for Zones 1 and 2 and a vertical orifice for Zone 3, the user can still include an outlet pipe with flow restrictor plate downstream of the vertical orifice and the program will account for this discharge as part of the combined outlet structure. It is important to note that the MHFD-Detention workbook does not consider the downstream conveyance limitations of the outlet pipe such as inlet vs. outlet control of the pipe or downstream tailwater conditions. These limitations must be considered separately outside of the workbook.

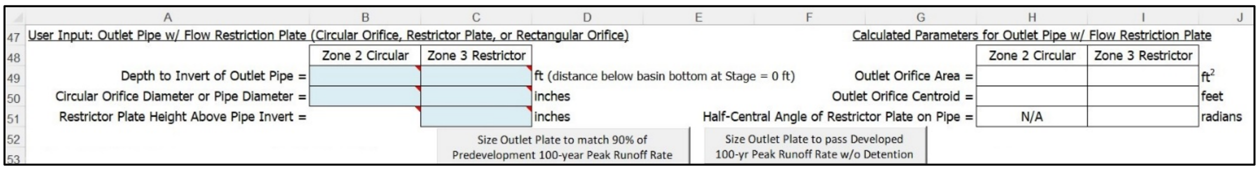


Figure 3.14 – Outlet Pipe with Flow Restriction Plate

Each of the input columns (B:C) can represent one of three different flow restriction plate configurations as shown in Figure 3.15. The first flow restriction plate configuration consists of a circular orifice opening cut into a metal plate covering the outlet pipe. The second flow restriction plate configuration consists of a rectangular orifice opening cut into a metal plate covering the outlet pipe. The third flow restriction plate configuration consists of a metal restrictor plate that covers the top part of a circular outlet pipe.

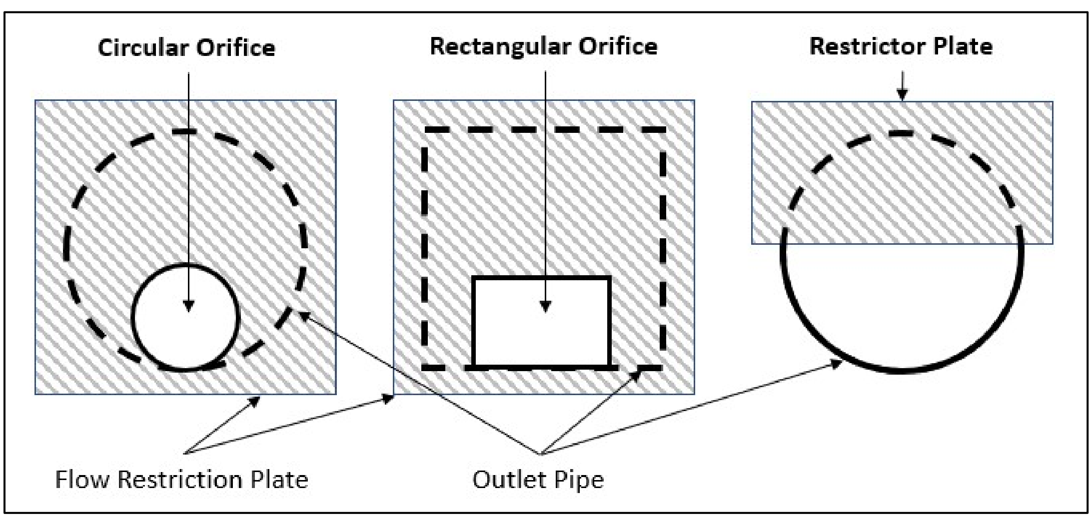


Figure 3.15 – Flow Restriction Plate Options

The user input values for the outlet pipe with flow restriction plate section include:

- **Depth to Invert of Outlet Pipe (feet)** is measured as the vertical distance below the basin bottom at a stage of zero feet. In other words, a value of zero sets the pipe invert equal to the basin bottom and a positive value sets the pipe invert below the basin bottom. A negative value is not accepted by the program because this would place the pipe invert above the basin bottom and prevent the basin from fully draining. For sand filters and rain gardens, the depth should be at least as deep as the underdrain outlet. For an EDB, a depth of 2.5 feet is often used since this is the minimum recommended micropool depth. Figure 3.15 shows the same invert elevations for the outlet pipe and the circular/rectangular orifice openings. However, if the orifice opening invert is above the outlet pipe invert (e.g., orifice opening centered in outlet pipe), the user should enter the orifice opening invert in Cell B49 or Cell C49 to make sure the headwater depths for calculating discharge are correct. If the outlet structure includes two separate outlet pipes with flow restriction plates, both columns (B:C) can be utilized. The invert for the second outlet pipe (Cell C49) must be greater (deeper) than the invert for the first outlet pipe (Cell B49).
- **Circular Orifice Diameter OR Rectangular Orifice Width OR Outlet Pipe Diameter (inches)** is dependent on the type of flow restriction plate being evaluated. The input cell description (Cell A50) will change depending on whether a circular orifice, rectangular orifice, circular pipe with restrictor plate, or a combination of two of them are selected. If a circular orifice is being used, the input cell represents the diameter of the circular orifice opening as seen in Figure 3.15. If a rectangular orifice is being used, the input cell represents the width of the rectangular orifice opening. It should be noted that for a circular or rectangular orifice opening, the outlet pipe dimensions are not needed or evaluated by the program, only the orifice dimensions. If a circular pipe with restrictor plate is being used, this input cell represents the diameter of the outlet pipe. Regardless of the flow restriction plate type selected, the user can enter any value greater than zero. If the outlet structure includes two separate outlet pipes with flow restriction plates, both columns (B:C) can be utilized and the input values will reflect the appropriate dimension depending on what is being evaluated in that column. The orifice area of the second opening must be larger than the first opening. This check is performed based on the calculated results in Cells H49:I49 discussed later.
- **Rectangular Orifice Height OR Restrictor Plate Height Above Invert (inches)** is only visible when a rectangular orifice or circular pipe with restrictor plate is selected. When a circular orifice is selected this input cell is hidden. The input cell description (Cell A51) will change depending on whether a rectangular orifice, circular pipe with restrictor plate, or both are selected. If a rectangular orifice is being used, this input cell represents the height of the rectangular orifice opening. If a circular pipe with restrictor plate is being used, this input cell represents the distance from the pipe invert to the bottom of the restrictor plate. Regardless of the flow restriction plate type selected, the user can enter

any value greater than zero. If the outlet structure includes two separate outlet pipes with flow restriction plates, both columns (B:C) can be utilized and the input values will reflect the appropriate dimension depending on what is being evaluated in that column. The orifice area of the second opening must be larger than the first opening. This check is performed based on the calculated results in Cells H49:I49 discussed later.

To the right of the outlet pipe with flow restriction plate input cells are six cells showing calculated parameters (three cells for each column) that are later used in the program code or hidden table calculations. Cells H49:H51 correspond to the first set of flow restriction plate input values in Cells B49:B51. Cells I49:I51 correspond to the second set of flow restriction plate input values in Cells C49:C51.

- **Outlet Orifice Area (square feet)** in Cell H49 or I49 is calculated to determine the opening area of the outlet orifice depending on the type of flow restriction plate selected. If a circular orifice is being used, the opening area is calculated using the equation $A = \frac{\pi D^2}{4}$ (where D is the circular orifice diameter in Cell B50 or C50). The resulting area is then converted from square inches to square feet by dividing by 144. If a rectangular orifice is being used, the opening area is calculated by multiplying the rectangular orifice width (Cell B50 or C50) by the rectangular orifice height (Cell B51 or C51) and then converting from square inches to square feet by dividing by 144. If a circular pipe with restrictor plate is being used, the geometry calculations are more complicated and it is necessary to determine the half-central angle of the restrictor plate on the circular pipe as shown below in Figure 3.16. Equations to solve for the area, centroid, and half-central angle are discussed in the ASCE technical paper titled *Flow Through Partially Submerged Orifice* (Appendix G). The outlet orifice area is calculated using the following equation.

$$A = \frac{D^2}{4}(\theta - \sin\theta\cos\theta)/144$$

  Where:

  A = outlet orifice area (square feet) calculated in Cell H49 or I49
  D = outlet pipe diameter (inches) from Cell B50 or C50
  θ = half-central angle of the restrictor plate on circular pipe (radians) calculated in Cell H51 or I51

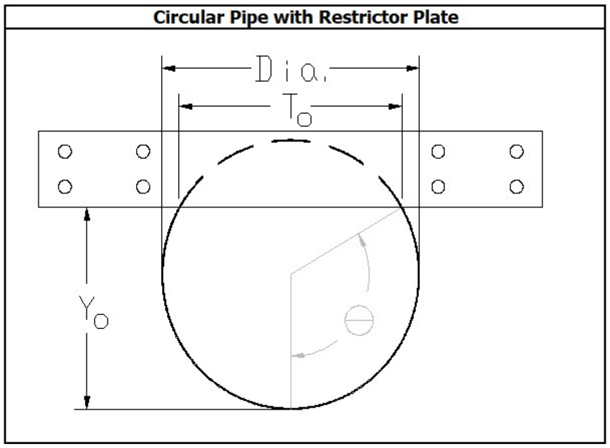


Figure 3.16 – Circular Pipe with Restrictor Plate Dimensions

- **Outlet Orifice Centroid (feet)** in Cell H50 or I50 is calculated to determine the height of the outlet orifice centroid depending on the type of flow restriction plate selected. If a circular orifice is being used, the centroid height is calculated as half of the circular orifice diameter in Cell B50 or C50 and then converted from inches to feet by dividing by 12. If a rectangular orifice is being used, the centroid height is calculated as half of the rectangular orifice height in Cell B51 or C51 and then converted from inches to feet by dividing by 12. If a circular pipe with restrictor plate is being used, the centroid height is dependent on the pipe diameter, restrictor plate height, and resulting half-central angle as discussed in Appendix G. The centroid height is calculated using the following equation.

$$Yc = \left[\frac{D}{2} - \frac{D * 2(\sin\theta)^3}{3(2\theta - \sin 2\theta)}\right] / 12$$

Where:

Y_c = outlet orifice centroid (feet) measured from the circular pipe invert and calculated in Cell H50 or I50

D = outlet pipe diameter (inches) from Cell B50 or C50

θ = half-central angle of the restrictor plate on circular pipe (radians) calculated in Cell H51 or I51

- **Half-Central Angle of Restrictor Plate on Pipe (radians)** in Cell H51 or I51 is only calculated when the circular pipe with flow restrictor plate is being evaluated. This cell will be populated with N/A if a circular orifice or rectangular orifice is being evaluated. The half-central angle is calculated as a function of the pipe diameter and the restrictor plate height as discussed in Appendix G and shown in the following equation.

$$\theta = \cos^{-1}\left(1 - \frac{2Y_o}{D}\right)$$

Where:

θ = half-central angle of the restrictor plate on circular pipe (radians) calculated in Cell H51 or I51

Y_o = restrictor plate height above pipe invert (inches) from Cell B51 or C51
D = outlet pipe diameter (inches) from Cell B50 or C50

The stage-discharge relationship for the first outlet pipe with flow restriction plate (input values in Cells B49:B51) is calculated using two columns in the stage-discharge table (Cells S87:T3087). The stage-discharge relationship for the second outlet pipe with flow restriction plate (input values in Cells C49:C51) is calculated in the next two columns of the stage-discharge table (Cells U87:V3087). The first column (S or U) estimates the discharge capacity of the outlet pipe with flow restriction plate based on the water surface elevation in each row of the table and disregards any upstream outlet structure component that may restrict the available flow. The second column (T or V) then compares the discharge capacity of the outlet pipe with flow restriction plate from the previous column against the available flow coming through the upstream outlet structure components (e.g., underdrain, water quality orifice plate, vertical orifice, and/or overflow weir) for each row in the table and selects the minimum discharge value as the controlling discharge. Each of these columns are discussed in more detail below.

The stage-discharge capacity calculations for the outlet pipe with flow restriction plate (Columns S and U) are based on the same orifice equations used for the vertical orifice openings discussed in Section 3.6 and depend on whether the current water surface elevation is above or below the top of the orifice opening.

When the water surface is above the top of the orifice opening (or restrictor plate height), the standard orifice discharge equation is used as shown below.

$$Q = C_d * A * \sqrt{2gh}$$

Where:

Q = discharge through flow restriction plate (cubic feet per second)

C_d = discharge Coefficient of 0.6

A = outlet orifice area (square feet) from Cell H49 or Cell I49

g = gravitational constant of 32.2 (feet per square second)

h = ponding depth of water above the outlet orifice centroid (feet)

The depth of water above the outlet orifice centroid is calculated as the water stage for the current row (Column B) plus the depth to the invert of the outlet pipe/orifice opening (Cell B49 or Cell C49) minus the corresponding outlet orifice centroid height (Cell H50 or Cell I50). Again, this equation only applies when the water surface is above the top (crown) of the outlet orifice opening (or above the height of the restrictor plate).

When the water surface falls below the top of the outlet orifice opening (or restrictor plate height), the standard orifice equation no longer applies and an empirical equation for estimating flow through a partially submerged orifice is used. The derivation of the empirical equation is documented in a Technical Memorandum entitled *Estimating Flow through a Partially Submerged Vertical Orifice*, dated December 31, 2015 (Appendix F). This technical memorandum is based on a technical paper titled *Flow Through Partially Submerged Orifice* in the ASCE Journal of Irrigation and Drainage Engineering by Guo, Stitt and Mays, dated December 1, 2015 (Appendix G).

The empirical equation for flow through a partially submerged orifice used in the workbook is based on the proportional relationship between depth and discharge for flow in partially full pipes. This relationship was fit to the results in the technical paper by Guo and Stitt which was verified in the University of Colorado Hydraulics Lab. The resulting equation showing the proportional relationship between depth and discharge with a best fit exponent to the technical paper results is shown below.

$$Q = Q_{full}\left(\frac{y}{D}\right)^{1.81}$$

Where:

Q = partially submerged outlet orifice discharge (cubic feet per second)

Q_{full} = orifice discharge when water depth equals top of orifice (cubic feet per second)

y = ponding depth ranging from orifice invert to top of orifice opening (feet)

D = diameter/height of orifice or height of restrictor plate on circular pipe (feet)

The full flow orifice discharge (Q_{full}) is calculated using the standard orifice discharge equation shown previously, where the ponding depth (h) is set equal to the top of the orifice opening minus the centroid height, the discharge coefficient (C_d) equals 0.6, and the outlet orifice area (A) is provided in Cell H49 or I49. The ponding depth (y) has a minimum value equal to the depth to the invert of the outlet pipe (Cell B49 or C49) since the pipe invert may be below the bottom of the basin. The ponding depth (y) has a maximum value equal to the top of the orifice opening when using this equation. For a circular orifice, the diameter (D) is set equal to the user input value for orifice diameter (Cell B50 or C50) divided by 12 to convert to feet. For a rectangular orifice or circular pipe with restrictor plate, the diameter (D) is set equal to the user input value for orifice height or plate height above pipe invert (Cell B51 or C51) divided by 12 to convert to feet.

It should be noted that the workbook includes a check to compare this calculated outlet orifice discharge capacity against the water quality orifice plate discharge (when included in the design) for each row in the stage-discharge table up to the top of the orifice opening, to make sure the empirical outlet orifice equation results do not constrict the flow coming through the water quality orifice plate. If the empirical equation results have a lower discharge, the water quality orifice plate discharge is used for that row based on the assumption that the standard orifice equation calculations used for the smaller water quality orifice plate are more accurate than the empirical equation for a larger, partially submerged orifice.

Once the outlet orifice discharge capacity is determined (Columns S or U), the actual discharge through the outlet orifice is determined (Columns T or V) by taking into account the upstream outlet structure components that may be limiting the available inflow to the outlet pipe with flow restriction plate. In order to check for the controlling discharge within the combined outlet structure components, the MHFD-Detention workbook assigns a rank (from 1 to 4) to any vertical orifice and/or overflow weir with dropbox included in the current design based on their respective invert elevations.

For example, consider a EDB design that includes two connected dropbox structures with a water quality orifice plate extending from the micropool to a depth of 3 feet, a vertical orifice for a 10-year storm at a stage of 4 feet, an overflow weir for the 25-year storm at a stage of 5 feet, a second vertical orifice for the 50-year storm at a stage of 6 feet, and a second overflow weir for the 100-year storm at a stage of 7 feet. The water quality orifice plate (or filter media with underdrain when used) is assigned a rank of zero and is always assumed to be the outlet with the lowest elevation. Of the remaining outlet structure components in this example, the first vertical orifice has the lowest invert elevation and discharges through the side of the first dropbox and would therefore be assigned a rank of 1. The lowest overflow weir also discharges into the first dropbox and has the next lowest invert elevation and would be assigned a rank of 2. The first dropbox is connected to the second dropbox by means of a circular orifice restriction plate at the bottom. The second vertical orifice is higher than the first overflow weir and discharges through the side of the second dropbox, therefore the second vertical orifice would be assigned a rank of 3. The second overflow weir also discharges into the second dropbox and with the highest invert elevation would be assigned a rank of 4. A circular outlet pipe with restrictor plate functions as the outlet for the second dropbox. While this may be an uncommon design, the workbook is setup to provide the user flexibility in evaluating complex outlet structures.

In this outlet configuration example, the workbook will determine the available inflow to the circular orifice restriction plate connecting the two dropbox structures for each row of the stage-discharge table by summing the discharge values of each outlet structure component up through rank 2. In other words, the total inflow reaching the circular outlet orifice for the first dropbox would be the sum of the water quality orifice plate (Column H), the first vertical orifice (Column I), and the first overflow weir (Column N). Based on this summed result, the controlling discharge for the circular orifice (Column T) is calculated as the minimum of the circular orifice discharge capacity (Column S) or the sum of the available inflows.

Similarly, the workbook will determine the available inflow to the circular pipe with flow restrictor plate (outlet for second dropbox) for each row of the stage-discharge table by summing the discharge values of each outlet structure component up through rank 4. In other words, the total inflow reaching the outlet pipe for the second dropbox would be the sum of the second vertical orifice (Column J), the second overflow weir (Column R), and the circular orifice connecting the two dropbox vaults (Column T), which as discussed above already accounts for the lower rank outlet components. Based on this summed result, the controlling discharge for the circular pipe with flow restrictor plate (Column V) is calculated as the minimum of the outlet pipe with flow restrictor plate discharge capacity (Column U) or the sum of the available inflows.

As seen in Figure 3.14, there are two buttons to assist with sizing the outlet pipe with flow restriction plate that may be visible depending on the zone volumes and outlet types selected by the user.

3.8.1 Size Outlet Plate to Match 90% of Predevelopment 100-year Peak Runoff Rate

The *Size Outlet Plate to match 90% of Predevelopment 100-year Peak Runoff Rate* button is only visible when the top storage zone is 100-year detention and it is drained by an overflow weir, dropbox, and outlet pipe with flow restriction plate. The top storage zone can be Zone 3 when Zone 1 drains through filter media or a water quality plate and Zone 2 drains through filter media, a water quality plate, or a vertical orifice. The top storage zone can be Zone 2 when Zone 1 drains through a water quality plate or a vertical orifice. The top storage zone can also be Zone 1 if designed for 100-year flood control only. In all other scenarios, this button is hidden from the user.

When the button is clicked, the program will calculate the flow restriction plate dimensions (Cell B50 and sometimes Cell B51) required to control the 100-year peak outflow to 90% of the predevelopment 100-year peak runoff rate as calculated in Cell I70. The 100-year peak outflow (Cell I69) is determined using the Modified Puls routing method to route the 100-year inflow hydrograph through the basin and outlet structure (outflow at each time interval is calculated in Cells DU3091:DU4531). In order for the automated sizing routine to begin, the program will check to make sure that the user has provided all of the necessary inputs for the other upstream outlet structure components included in the design (e.g., underdrain orifice, water quality orifice plate, elliptical slot, vertical orifice, and/or overflow weir with dropbox) as described for the three scenarios above. In addition, the user will need to provide the depth to the invert of the outlet pipe or orifice opening (Cell B49) prior to the program solving for the restriction plate geometry.

The first step in the sizing routine is to oversize the overflow weir front edge length (Cell B41) and horizontal length of weir sides (Cell B43) to 999 feet to make sure they don’t restrict the flow reaching the outlet pipe. The routine also sets the emergency spillway invert at a depth of 999 feet to ensure water does not exit through the spillway and impact the outlet pipe calculations. Then, depending on the type of flow restriction plate selected by the user, the program iteratively increases or decreases the orifice geometry as necessary until the ratio of the 100-year peak outflow to predevelopment runoff rate (Cell I70) equals 0.90.

For a circular orifice restriction plate, the program starts by calculating an initial estimate of the required orifice area to match 90% of the predevelopment runoff rate by rearranging the standard orifice equation and solving for orifice area as shown in the equation below.

$$A = \frac{0.90 * PreQ_{100}}{C_d * \sqrt{2gh}}$$

Where:

A = initial estimate of outlet orifice area (square feet)

$PreQ_{100}$ = 100-year predevelopment runoff rate (cubic feet per second) in Cell I65

C_d = discharge Coefficient of 0.6

g = gravitational constant of 32.2 (feet per square second)

h = ponding depth of water above the outlet orifice invert (feet)

The ponding depth of water above the outlet orifice invert is calculated as the estimated stage for the 100-year storage zone as shown in Cells E7:E9. Note that this initial estimate is based on the orifice invert and not the orifice centroid since the orifice diameter is still unknown. Regardless, this ponding depth provides a good starting point to help speed up the iterative program calculations. From the initial estimate of orifice area, the program calculates the corresponding orifice diameter as $D = \sqrt{4A/\pi}$ and plugs this value into Cell B50. The program then runs the Goal Seek function in Excel to solve for a 100-year peak outflow to predevelopment runoff rate ratio (Cell I70) equal to 0.90 by changing the initial estimate of the orifice diameter. This revised orifice diameter takes into account the actual 100-year ponding depth determined from the Modified Puls routing method. If the Goal Seek routine cannot find a solution with an orifice diameter greater than zero or less than the available height in the drop structure, a message will notify the user that they can try manually adjusting the orifice diameter or they can switch to a wide rectangular orifice.

For a rectangular orifice restriction plate, the program also starts by calculating an initial estimate of the required orifice area to match 90% of the predevelopment runoff rate by rearranging the standard orifice equation and solving for the orifice area as shown above for the circular orifice. However, instead of calculating the corresponding orifice diameter, the program calculates initial estimates of the orifice width and height. If the user has already provided an orifice height, the program will calculate the corresponding orifice width required to match the initial orifice area estimate. If the user does not provide a starting orifice height, the program will assume a square orifice and set the width and height both equal to the square root of the initial orifice area estimate. The calculated width and height values are then plugged into Cells B50:B51 and the Goal Seek function in Excel is run to solve for a 100-year peak outflow to predevelopment runoff rate ratio (Cell I70) equal to 0.90 by changing the initial estimate of the orifice width. This revised orifice width takes into account the actual 100-year ponding depth determined from the Modified Puls routing method. If the Goal Seek routine cannot find a solution with an orifice width greater than zero or less than 12 feet, or an orifice height less than the available height in the drop structure, a message will notify the user that they can try manually adjusting the orifice dimensions or they can increase the dropbox structure dimensions and try again.

For a <u>circular outlet pipe with restrictor plate</u>, the sizing approach is done differently than for the circular or rectangular orifice restriction plates discussed above. First the program over sizes the outlet pipe to have 120% of the required capacity for the target release rate (90% of the predevelopment runoff rate). This ensures that the outlet pipe has adequate capacity and is not controlling the release rate and that when the restrictor plate is installed, it will control the release rate instead. To size the outlet pipe, the program starts with a minimum diameter of 18-inches in Cell B50 and then increases the pipe diameter by 3-inch increments up to a 36" diameter, and then continues to increase the pipe diameter by 6-inch increments until the pipe is large enough to convey 120% of the target release rate. Each time the pipe diameter is increased, the Modified Puls routing method calculations are performed again to check the discharge rate through the outlet pipe. For these calculations, the restrictor plate height is assumed to be equal to the crown of the pipe so that it does not impact the discharge rate. If the program cannot find a solution with a pipe diameter less than 12 feet or less than the available height in the drop structure, a message will notify the user that they can try manually adjusting the pipe diameter or they can switch to a rectangular orifice plate. If the user already entered a pipe diameter in Cell B50 before clicking the sizing button, the program will compare the calculated pipe diameter to the user input diameter. Then a message will notify the user of any difference and allow the user to either keep their original pipe diameter or accept the program calculated pipe diameter. After the pipe diameter is determined, the program then moves on to fitting the restrictor plate height over the pipe. The restrictor plate height is determined by starting at the pipe crown and moving down to cover the pipe opening at increments of 0.1 feet until the 100-year peak outflow to predevelopment runoff rate ratio (Cell I70) is equal to 0.90. Each time the restrictor plate is lowered, the Modified Puls routing method calculations are performed again to check the discharge rate below the restrictor plate.

After the flow restriction plate on the outlet pipe is sized, the program then goes back and replaces the user input overflow weir front edge length (Cell B41) and horizontal length of weir sides (Cell B43) to their original values and checks to make sure the overflow weir doesn't limit the flow rate reaching the flow restriction plate on the outlet pipe. This control check is performed differently depending on the type of overflow weir used in the design. The three options include an overflow weir without grate acting as a spillway into the dropbox, an overflow weir with a flat grate, and an overflow weir with a sloped grate.

When the overflow weir is acting as a spillway with a bottom width but no grate or horizontal side lengths, the program starts the control check by restoring the user input weir bottom length (Cell B41) and setting the horizontal length of weir sides (Cell B43) to zero. Next the program checks to see if the 100-year peak outflow to predevelopment runoff rate ratio (Cell I70) is still equal to 0.90 and that the flow restriction plate is controlling the discharge rate. If not, the overflow weir may be restricting the discharge rate and the program will start to increase the weir bottom length until the discharge ratio (Cell I70) becomes 0.90 again and the flow restriction plate is controlling the discharge rate. If the program calculated bottom length is less than the original user input value then the program will notify the user that their input weir bottom length is larger than required and will offer to reduce it to the calculated bottom length. The user then has the choice to keep their input value or change to the shorter bottom length.

When the overflow weir includes a flat grate, the program starts the control check by calculating the minimum square grate dimensions that will ensure that the 100-year velocity through the grate does not exceed 2.0 feet per second (safety check based on pinning velocity) and that the total grate open area is at least four times the orifice area of the flow restriction plate (to limit potential clogging of flow restriction plate). Next the program checks to see if the 100-year peak outflow to predevelopment runoff rate ratio (Cell I70) is still equal to 0.90 and that the flow restriction plate is controlling the discharge rate. If not, the overflow weir with flat grate may be restricting the discharge rate and the program will start to increase the weir front length and side length until the discharge ratio (Cell I70) becomes 0.90 again and the flow restriction plate is controlling the discharge rate. If the program calculated weir front length and side length result in a grate area less than the original user input values then the program will notify the user that their input front length and side length are larger than required and will offer to reduce them to the calculated lengths. The user then has the choice to keep their input values or change to the shorter lengths.

When the overflow weir includes a sloped grate, the program starts the control check by calculating the minimum weir front edge length that will ensure that the 100-year velocity through the grate does not exceed 2.0 feet per second (safety check based on pinning velocity) and that the total grate open area is at least four times the orifice area of the flow restriction plate (to limit potential clogging of flow restriction plate). Next the program checks to see if the 100-year peak outflow to predevelopment runoff rate ratio (Cell I70) is still equal to 0.90 and that the flow restriction plate is controlling the discharge rate. If not, the overflow weir with sloped grate may be restricting the discharge rate and the program will start to increase the weir front length until the discharge ratio (Cell I70) becomes 0.90 again and the flow restriction plate is controlling the discharge rate. If the program calculated weir front length is less than the original user input weir front edge length then the program will notify the user that their input length is larger than required and will offer to reduce it to the calculated length. The user then has the choice to keep their input value or change to the shorter front edge length.

The final step of the automated sizing routine to match 90% of the predevelopment 100-year peak runoff rate is to reset the emergency spillway invert stage (Cell B54) to the original user input value. If the original user input spillway invert stage is less than the 100-year maximum ponding depth (Cell I76) or if the user did not provide a spillway invert stage, the program will notify the user that they need to set the spillway invert stage above the 100-year maximum ponding depth to ensure it does not interfere with the controlled release rate. Once a valid spillway invert stage is provided, the program will check to see if the other spillway input parameters were provided and if they are sufficient to pass the undetained 100-year peak inflow (Cell I68). An approximate spillway crest length (Cell B55) is back calculated using the broad crested rectangular weir equation $Q = C_{BCW}LH^{1.5}$ and assuming the 100-year peak inflow passes through the spillway with a flow depth (H) of one foot and a broad crested weir coefficient (C_{BCW}) of 3.0. If the user input spillway crest length is less than the calculated value or a value was never provided, the program will default to the approximate spillway crest length. If the user did not provide spillway end slopes (Cell B56) or the freeboard above the maximum water surface in the spillway (Cell B57), the program will default to an end slope of 4:1 (H:V) and 1.0 foot of freeboard. Otherwise, the end slopes and freeboard will remain as the user entered them. The program then finishes by calculating the spillway design flow depth (Cell H54) as discussed in Section 3.9 for the Emergency Spillway. That concludes the sizing button routine and the user will then have a chance to review the Routed Hydrograph Results table and check for design problems.

3.8.2 Size Outlet Plate to Pass Developed 100-year Peak Runoff Rate Without Detention

The *Size Outlet Plate to pass Developed 100-year Peak Runoff Rate without Detention* button is only visible when 100-year detention is not selected for one of the storage zone volumes and a circular or rectangular flow restriction plate is selected as one of the outlet types. In this type of scenario, the flow restriction plate will be sized to determine the minimum outlet pipe area required to pass the developed 100-year peak runoff rate without attenuation. In all other scenarios, this button is hidden from the user.

When the button is clicked, the program will calculate the flow restriction plate dimensions (Cell B50 and potentially Cell B51) required to pass the 100-year peak inflow (Cell I68) without providing detention. In order for the automated sizing routine to begin, the program will check to make sure that the user has provided all of the necessary inputs for the other upstream outlet structure components included in the design (e.g., underdrain orifice, water quality orifice plate, elliptical slot, vertical orifice, and/or overflow weir with dropbox). In addition, the user will need to provide the depth to the invert of the orifice opening (Cell B49) and starting orifice dimensions (Cell 50 and potentially Cell 51) prior to the program solving for the restriction plate geometry.

The first step in the sizing routine is to oversize the overflow weir front edge length (Cell B41) and horizontal length of weir sides (Cell B43) to 999 feet to make sure they don't restrict the flow rate reaching the outlet pipe. The routine also sets the emergency spillway invert at a depth of 999 feet to ensure water does not exit through the spillway and impact the outlet pipe calculations. Then, depending on the type of flow restriction plate selected by the user, the program iteratively increases or decreases the orifice geometry as necessary until the peak outflow (Cell I69) is equal to the peak inflow (Cell I68).

For a <u>circular orifice restriction plate</u>, the program starts by calculating an initial estimate of the orifice area required to match the peak outflow and inflow by rearranging the standard orifice equation and solving for orifice area as shown in the equation below.

$$A = \frac{Q_{100}}{C_d * \sqrt{2gh}}$$

Where:

A = initial estimate of outlet orifice area (square feet)

Q_{100} = 100-year peak inflow (cubic feet per second) in Cell I68

C_d = discharge Coefficient of 0.6

g = gravitational constant of 32.2 (feet per square second)

h = ponding depth of water above the outlet orifice invert (feet)

The ponding depth of water above the outlet orifice invert is calculated as the estimated stage for the 100-year maximum ponding depth (Cell I76). Note that this initial estimate is based on the orifice invert and not the orifice centroid since the orifice diameter is still unknown. Regardless, this ponding depth provides a good starting point to help speed up the iterative program calculations. From the initial estimate of orifice area, the program calculates the corresponding orifice diameter as $D = \sqrt{4A/\pi}$ and plugs this value into Cell B50. The program then runs the Goal Seek function in Excel to solve for a 100-year peak outflow that matches the 100-year peak inflow by changing the initial estimate of the orifice diameter. This revised orifice diameter takes into account the actual 100-year ponding depth determined from the Modified Puls routing method. If the Goal Seek routine cannot find a solution with an orifice diameter greater than zero or less than the available height in the drop structure, a message will notify the user that they can try manually adjusting the orifice diameter or they can switch to a wide rectangular orifice.

For a <u>rectangular orifice restriction plate</u>, the program also starts by calculating an initial estimate of the required orifice area to match the peak outflow and inflow by rearranging the standard orifice equation and solving for the orifice area as shown above for the circular orifice.

However, instead of calculating the corresponding orifice diameter, the program calculates initial estimates of the orifice width and height. If the user has already provided an orifice height, the program will calculate the corresponding orifice width required to match the initial orifice area estimate. If the user does not provide a starting orifice height, the program will assume a square orifice and set the width and height both equal to the square root of the initial orifice area estimate. The calculated width and height values are then plugged into Cells B50:B51 and the Goal Seek function in Excel is run to solve for a 100-year peak outflow that matches the 100-year peak inflow by changing the initial estimate of the orifice width. This revised orifice width takes into account the actual 100-year ponding depth determined from the Modified Puls routing method. If the Goal Seek routine cannot find a solution with an orifice width greater than zero or less than 12 feet, or an orifice height less than the available height in the drop structure, a message will notify the user that they can try manually adjusting the orifice dimensions or they can increase the dropbox structure dimensions and try again.

After the flow restriction plate is sized, the program then goes back and replaces the user input overflow weir front edge length (Cell B41) and horizontal length of weir sides (Cell B43) to their original values and checks to make sure the overflow weir doesn't limit the flow rate reaching the outlet pipe. This control check is performed differently depending on the type of overflow weir used in the design. The three options include an overflow weir without grate acting as a spillway into the dropbox, an overflow weir with a flat grate, and an overflow weir with a sloped grate.

When the overflow weir is acting as a spillway with a bottom width but no grate or horizontal side lengths, the program starts the control check by restoring the user input weir bottom length (Cell B41) and setting the horizontal length of weir sides (Cell B43) to zero. Next the program checks to see if the 100-year peak outflow matches the peak inflow and that the flow restriction plate is controlling the discharge rate. If not, the overflow weir may be restricting the discharge rate and the program will start to increase the weir bottom length until the peak outflow matches the peak inflow and the flow restriction plate is controlling the discharge rate. If the program calculated bottom length is less than the original user input value then the program will notify the user that their input weir bottom length is larger than required and will offer to reduce it to the calculated bottom length. The user then has the choice to keep their input value or change to the shorter bottom length.

When the overflow weir includes a flat grate, the program starts the control check by calculating the minimum square grate dimensions that will ensure that the 100-year velocity through the grate does not exceed 2.0 feet per second (safety check based on pinning velocity) and that the total grate open area is at least four times the orifice area of the flow restriction plate (to limit potential clogging of flow restriction plate). Next the program checks to see if the 100-year peak outflow matches the peak inflow rate and that the flow restriction plate is controlling the discharge rate. If not, the overflow weir with flat grate may be restricting the discharge rate and the program will start to increase the weir front length and side length until the peak outflow matches the peak inflow and the flow restriction plate is controlling the discharge rate. If the program calculated weir front length and side length result in a grate area less than the original user input values then the program will notify the user that their input front length and side length are larger than required and will offer to reduce them to the calculated lengths. The user then has the choice to keep their input values or change to the shorter lengths.

When the overflow weir includes a sloped grate, the program starts the control check by calculating the minimum weir front edge length that will ensure that the 100-year velocity through the grate does not exceed 2.0 feet per second (safety check based on pinning velocity) and that the total grate open area is at least four times the orifice area of the flow restriction plate (to limit potential clogging of flow restriction plate). Next the program checks to see if the 100-year peak outflow still equals the peak inflow and that the flow restriction plate is controlling the discharge rate. If not, the overflow weir with sloped grate may be restricting the discharge rate and the program will start to increase the weir front length until the peak outflow matches the peak inflow and the flow restriction plate is controlling the discharge rate. If the program calculated weir front length is less than the original user input weir front edge length then the program will notify the user that their input length is larger than required and will offer to reduce it to the calculated length. The user then has the choice to keep their input value or change to the shorter front edge length.

The final step of the automated sizing routine is to reset the emergency spillway invert stage (Cell B54) to the original user input value. If the original user input spillway invert stage is less than the 100-year maximum ponding depth (Cell I76) or if the user did not provide a spillway invert stage, the program will notify the user that they need to set the spillway invert stage above the 100-year maximum ponding depth to ensure it does not interfere with the controlled release rate. Once a valid spillway invert stage is provided, the program will check to see if the other spillway input parameters were provided and if they are sufficient to pass the undetained 100-year peak inflow (Cell I68). An approximate spillway crest length (Cell B55) is back calculated using the broad crested rectangular weir equation $Q = C_{BCW}LH^{1.5}$ and assuming the 100-year peak inflow passes through the spillway with a flow depth (H) of one foot and a broad crested weir coefficient (C_{BCW}) of 3.0. If the user input spillway crest length is less than the calculated value or a value was never provided, the program will default to the approximate spillway crest length. If the user did not provide spillway end slopes (Cell B56) or the freeboard above the maximum water surface in the spillway (Cell B57), the program will default to an end slope of 4:1 (H:V) and 1.0 foot of freeboard. Otherwise, the end slopes and freeboard will remain as the user entered them. The program then finishes by calculating the spillway design flow depth (Cell H54) as discussed in Section 3.9 for the Emergency Spillway. That concludes the sizing button routine and the user will then have a chance to review the Routed Hydrograph Results table and check for design problems.

3.9 Emergency Spillway

The next section on the Outlet Structure worksheet provides user input cells for an emergency spillway (Cells B54:B58). Figure 3.17 shows the emergency spillway section of the worksheet with all of the potential input cells and the automated sizing button visible for purposes of describing the various scenarios available in the workbook. The emergency spillway section is always available for use, regardless of the outlet types selected for the three zones. The emergency spillway is modeled as a broad crested weir (rectangular or trapezoidal) and is intended to pass any flood events that exceed the available storage capacity in the basin, including the 500-year storm and smaller storms if the outlet structure becomes clogged.

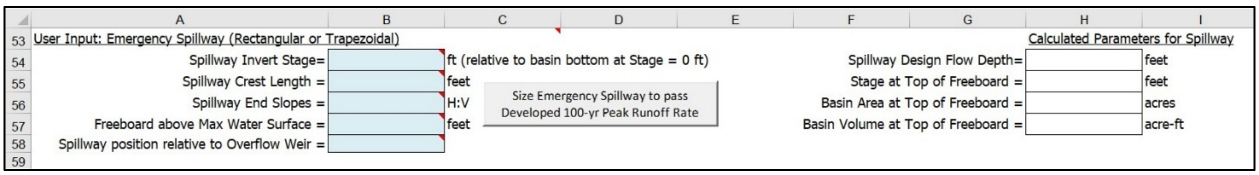


Figure 3.17 – Emergency Spillway

The user input values for the emergency spillway section include:

- **Spillway Invert Stage (feet)** is measured relative to the basin bottom at a stage of zero feet. The emergency spillway invert should be set at or above the maximum ponding depth of the selected target storage volume (e.g., 100-year max ponding depth in Cell I76). When using one of the automated sizing buttons in the outlet pipe with flow restriction plate section, the program will provide an emergency spillway invert elevation just above the maximum ponding depth. The user can override the recommended invert value but a negative value is not acceptable.
- **Spillway Crest Length (feet)** is the bottom length of the spillway and is measured perpendicular to the flow direction. The length of the crest will affect the depth of flow through the spillway. At a minimum, the user should size the spillway to pass the peak flow from the undetained design flood (e.g., 100-year) inflow hydrograph. Consider the limitations of the stage-storage-discharge table and increase the stage increment if necessary to ensure that the spillway invert stage plus the depth of flow does not exceed the maximum limit of the table. The user can enter any value except a negative value which is not acceptable. A value of zero results in a triangular weir.
- **Spillway End Slopes (H:V)** represent the side slopes of a trapezoidal weir (e.g., 4H:1V) or the vertical sides of a rectangular weir (slope equals zero). The user can enter any value except a negative value.
- **Freeboard Above Maximum Water Surface (feet)** is the desired stage for the top of the embankment and should be a minimum of one foot above the maximum water surface stage when the emergency spillway is conveying the maximum design flow. The user can enter any value except a negative value.

- **Spillway Position Relative to Overflow Weir** in Cell B58 is only visible when the user selects an overflow weir without pipe for one or more of the zone outlet types. For all other outlet configurations, Cells B58 is hidden and locked. When visible, the user can select offset or overlapping from the pulldown list in Cell B58 to indicate the emergency spillway position relative to the lower overflow weir. This is done in order to properly account for the flow through separate and offset weirs (versus flow through an overlapping two-stage weir). If the spillway is offset from the overflow weir, then the discharge through each is calculated independently. However, if the spillway overlaps the overflow weir, then the program will subtract the discharge through the overlapping section from the spillway to avoid double counting the flow through this overlapping section.

To the right of the overflow weir input cells are four cells showing calculated parameters (Cells H54:H57) that are later used in the program code or hidden table calculations.

- **Spillway Design Flow Depth (feet)** is calculated using an iterative code routine to determine the flow depth in spillway required to pass the undetained 100-year peak inflow (Cell I68) based on the user input spillway geometry. Any time the user modifies the spillway geometry, the program will automatically update the calculated flow depth. The broad-crested weir equations discussed in Section 5.14.2 of the USDCM Storage Chapter are used in the code routine. In order to calculate the total flow over a trapezoidal weir, the results from equation 12-8 (discharge through rectangular weir) are added to two times the result from equation 12-9 (discharge through sloping weir on one side). The combined equation used in the workbook code is shown below. If the side slopes are zero, the second term in the equation becomes zero and you are left with the rectangular weir equation. The program iteratively solves this equation by changing the headwater depth in increments of 0.1 feet until the calculated discharge matches the undetained 100-year peak inflow (Cell I68).

  $$Q = C_{BCW}LH^{1.5} + 2\left(\frac{2}{5}\right)C_{BCW}ZH^{2.5}$$

  Where:

  Q = total spillway discharge (cubic feet per second)

  C_{BCW} = broad-crested weir coefficient set equal to 3.0

  L = spillway crest length (feet) in Cell B55

  Z = spillway end slopes (H:V) in Cell B56

  H = headwater depth above spillway invert (feet)

- **Stage at Top of Freeboard (feet)** is calculated as the sum of the spillway invert stage (Cell B54), the spillway design flow depth (Cell H54), and the freeboard above max water surface depth (Cell B57). This provides the minimum stage for the top of the basin embankment in order to provide the desired freeboard.
- **Basin Area at Top of Freeboard (acres)** is determined by looking up the stage at top of freeboard (Cell H55) in the stage-area-volume table (Cells B87:D3087) and returning the area corresponding to this stage. The area is then divided by 43,560 to convert from square feet to acres.
- **Basin Volume at Top of Freeboard (acre-feet)** is determined by looking up the stage at top of freeboard (Cell H55) in the stage-area-volume table (Cells B87:D3087) and returning the volume corresponding to this stage. The volume is then divided by 43,560 to convert from cubic feet to acre feet.

The stage-discharge relationship for the emergency spillway is calculated in Cells W87:W3087 using the broad-crested weir equations discussed in Section 5.14.2 of the USDCM Storage Chapter. In order to calculate the total flow over a trapezoidal weir, the results from equation 12-8 (discharge through rectangular weir) are added to two times the result from equation 12-9 (discharge through sloping weir on one side). The resulting equation used in the MHFD-Detention workbook is shown below. If the side slopes are zero, the second term in the equation becomes zero and you are left with the rectangular weir equation.

$$Q = C_{BCW} L H^{1.5} + 2\left(\frac{2}{5}\right) C_{BCW} Z H^{2.5}$$

Where:

Q = total spillway discharge (cubic feet per second)

C_{BCW} = broad-crested weir coefficient set equal to 3.0

L = spillway crest length (feet) in Cell B55

Z = spillway end slopes (H:V) in Cell B56

H = headwater depth above spillway invert stage (feet)

This equation is solved for every row in the stage-discharge table and when the headwater depth is less than the spillway invert stage, the discharge for that row in the table is zero. If the user has also selected one or more overflow weirs without pipes for the zone outlet types, then the user must indicate whether the spillway is offset from or overlapping the overflow weir(s). If the spillway and overflow weir(s) are offset, then the discharge through the spillway and weir(s) is calculated independently using the equations described above. However, if the spillway and weir(s) overlap, then the program will subtract the discharge through the overlapping section from the spillway (Cells W87:W3087) to avoid double counting the flow through this overlapping section.

As seen in Figure 3.17, there is a sizing button to help size the emergency spillway to be able to pass the 100-year peak inflow rate in case the outlet structure becomes clogged.

3.9.1 Size Emergency Spillway to Pass Developed 100-year Peak Runoff Rate

The *Size Emergency Spillway to Pass Developed 100-year Peak Runoff Rate* button is always available to the user and is intended to ensure that the emergency spillway can pass the 100-year peak inflow rate in case the outlet structure becomes clogged with debris. The emergency spillway also serves to pass larger events such as the 500-year design storm.

When the button is clicked, the program will calculate a spillway crest length (Cell B55) sufficient to pass the undetained 100-year peak inflow rate and then calculate the resulting flow depth. In order for the sizing routine to begin, the user must provide at least one storage zone above which the spillway will be placed. The actual spillway input parameters (Cells B54:B58) can be left blank and the program will suggest a spillway invert stage above the maximum ponding depth of the desired storage volume. The user can then confirm the value provided by program or enter their own spillway invert stage. The program will also check to see if there is an overflow weir without pipe and if the user specified the spillway location as offset or overlapping with the overflow weir.

Once these input values are confirmed, the program determines an approximate spillway crest length (Cell B55) by rearranging the broad crested rectangular weir equation $Q = C_{BCW}LH^{1.5}$ to solve for length and assuming the 100-year peak inflow passes through the spillway with a flow depth (H) of one foot and a broad crested weir coefficient (C_{BCW}) of 3.0. If the user did not provide spillway end slopes (Cell B56) or the freeboard above the maximum water surface in the spillway (Cell B57), the program will default to an end slope of 4:1 (H:V) and 1.0 foot of freeboard. Otherwise, the end slopes and freeboard will remain as the user entered them.

The program then finishes by calculating the spillway design flow depth (Cell H54) using an iterative approach to determine the flow depth in spillway required to pass the undetained 100-year peak inflow (Cell I68). The same broad-crested weir equations discussed earlier in this section are used and the program iteratively solves the combined weir equation by changing the headwater depth in increments of 0.1 feet to match the design flow rate. That concludes the sizing button routine and the user can then review the Routed Hydrograph Results table and check for design problems.

3.10 Routed Hydrograph Results

The Routed Hydrograph Results table provides a summary of rainfall/runoff hydrology, outlet structure release rates, drain times, and maximum ponding depths, areas, and volumes as shown in Figure 3.18. Each column of the table represents the results for a different capture volume

(WQCV and EURV) or design storm (2-year through 500-year). The first seven rows of the table (Rows 62 through 68) summarize the rainfall and CUHP runoff results. The last 10 rows of the table (Rows 69 through 78) summarize the Modified Puls Routing Method results based on the inflow hydrographs, basin geometry and the outlet structure configuration.

	A	B	C	D	E	F	G	H	I	J
60	Routed Hydrograph Results	The user can override the default CUHP hydrographs and runoff volumes by entering new values in the Inflow Hydrographs table (Columns W through AF).								
61	Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
62	One-Hour Rainfall Depth (in) =	N/A	N/A	0.83	1.09	1.33	1.69	1.99	2.31	3.14
63	CUHP Runoff Volume (acre-ft) =									
64	Inflow Hydrograph Volume (acre-ft) =									
65	CUHP Predevelopment Peak Q (cfs) =									
66	OPTIONAL Override Predevelop Chart Area Q (cfs) =									
67	Predevelopment Unit Peak Flow, q (cfs/acre) =									
68	Peak Inflow Q (cfs) =									
69	Peak Outflow Q (cfs) =									
70	Ratio Peak Outflow to Predevelopment Q =									
71	Structure Controlling Flow =									
72	Max Velocity through Grate 1 (fps) =									
73	Max Velocity through Grate 2 (fps) =									
74	Time to Drain 97% of Inflow Volume (hours) =									
75	Time to Drain 99% of Inflow Volume (hours) =									
76	Maximum Ponding Depth (ft) =									
77	Area at Maximum Ponding Depth (acres) =									
78	Maximum Volume Stored (acre-ft) =									

Figure 3.18 – Routed Hydrograph Results Table

Each row of the summary results table is discussed in the bullet points below:

- **Design Storm Return Period** provides the column headers for the table. The results for the WQCV and EURV columns are based on a capture volume starting at the brim full capacity and draining out over time. The results for the 2-year through 500-year design storm columns are based on routing CUHP inflow hydrographs through the basin and outlet structure.
- **One-Hour Rainfall Depth (inches)** for the 2-year through 500-year design storms are copied over from the Basin worksheet and reflect the location selected from the pulldown list in Cell B22. If the user provided override rainfall depths (Cells D28:D34), then those values are copied over to this table instead. There are no rainfall depths associated with the WQCV or EURV.
- **CUHP Runoff Volume (acre-feet)** for the 2-year through 500-year design storms were calculated by the embedded CUHP code and complete results are stored on a hidden worksheet. The total post-development runoff volume for each design storm is copied to these cells. The WQCV and EURV values were calculated on the Basin worksheet (Cells B26:B27) using empirical equations and are copied over to this table.
- **Inflow Hydrograph Volume (acre-feet)** is by default equal to the CUHP runoff volume in the previous row. However, if the user overrides the CUHP inflow hydrographs (Cells X8:AF80) with their own inflow hydrographs, the volumes in this row will reflect the new inflow hydrograph volumes which are calculated by summing the flowrates for each incremental time step. When overridden, the cells will turn pink to notify reviewers that the CUHP inflow hydrographs are not being used in the design.
- **CUHP Predevelopment Peak Flow (cubic feet per second)** for the 2-year through 500-year design storms were calculated by the embedded CUHP code and complete predevelopment hydrographs are stored on a hidden worksheet. The peak flow rate from

each hydrograph is copied over to this row. The WQCV and EURV values in this row are always N/A.

- **OPTIONAL Override Predevelopment Peak Flow (cubic feet per second)** for the 2-year through 500-year design storms are user input cells (light blue color). This allows the user to override the default CUHP predevelopment peak flows with their own values for use in further calculations and for target release rates from the outlet structure. When the values are overridden, the cell turns pink to make it clear to reviewers that the CUHP values are not being used in the design. The WQCV and EURV values in this row are always N/A.
- **Predevelopment Unit Peak Flow (cubic feet per second per acre)** for the 2-year through 500-year design storms are calculated to determine the peak discharge rate per watershed acre by dividing the predevelopment peak flow (Cells D65:J65, or when overridden by the user Cells D66:J66) by the watershed area on the Basin worksheet (Cell B13). The WQCV and EURV values in this row are always N/A.
- **Peak Inflow (cubic feet per second)** for the 2-year through 500-year design storms is calculated by finding the maximum value for each inflow hydrograph (Inflow_1 Column) stored in the Modified Puls routing table (Cells B3091:EZ4531). The inflow hydrographs are either based on the CUHP calculated inflow hydrographs or user-override inflow hydrographs. The WQCV and EURV values in this row are typically N/A, unless the user provides override inflow hydrographs for these events (not recommended).
- **Peak Outflow (cubic feet per second)** is calculated by finding the maximum value for each outflow hydrograph (Outflow_2 Column) stored in the Modified Puls routing table (Cells B3091:EZ4531). The outflow hydrographs are calculated by routing the inflow hydrographs through the basin geometry to account for detention storage (stage-area-volume relationship) and determining the release rate through the combined outlet structure (stage-discharge relationship).
- **Ratio of Peak Outflow to Predevelopment Peak Flow** is calculated as the peak outflow (Row 69) divided by predevelopment peak flow (Row 65, or Row 66 when the user overrides the CUHP value). This is used to check whether the release rate is consistent with the predevelopment peak flow rate for full spectrum detention design. The WQCV and EURV values in this row are always N/A. The 2-year ratio is always N/A also because the predevelopment peak flow is typically so low that the ratio can become very large and is misleading.
- **Structure Controlling Flow** identifies which component of the outlet structure controls the release rate for each storm event being evaluated. The program first determines the maximum ponding depth for each storm event (Row 76 discussed below) and then checks to see which outlet structure component is the most restrictive and limiting the release rate at that ponding depth. The outlet structure component controlling the flow rate for

each row in the stage-discharge table is determined in Cells BD87:BL3087 which will be discussed in the next section.

- **Max Velocity Through Grate 1 (feet per second)** is calculated to ensure that the flow velocity through the grate does not exceed two feet per second which MHFD recommends as the maximum allowable velocity to prevent a person from being pinned to the grate during a flood event. At velocities lower than 2 feet per second, people should be able to climb to safety and avoid being trapped. The values in this row are only populated when the first overflow weir with grate (Cells B40:B45) is included in the design and the maximum ponding depth for the storm event exceeds the overflow weir front edge height (Cell B40). When these conditions are met, the velocity is calculated as the flowrate through the grate divided by the open area of the grate without debris clogging (Cell H43). In order to determine the flow rate through the grate, the program will check the rank of the overflow weir relative to other potential outlet structure components (e.g., underdrain orifice, water quality orifice plate, vertical orifice, etc.) and only account for the flow that passes through the grate. If the velocity exceeds two feet per second, the cell will turn pink to bring attention to the problem.
- **Max Velocity Through Grate 2 (feet per second)** is calculated for the same reasons described above for the first grate. The values in this row are only populated when the second overflow weir with grate (Cells C40:C45) is included in the design and the maximum ponding depth for the storm event exceeds the overflow weir front edge height (Cell C40). When these conditions are met, the velocity is calculated as the flowrate through the grate divided by the open area of the grate without debris clogging (Cell I43). In order to determine the flow rate through the grate, the program will check the rank of the second overflow weir relative to other potential outlet structure components (e.g., flow restriction plate from first dropbox structure and vertical orifice openings in the second dropbox) and only account for the flow that passes through the second grate. If the velocity exceeds two feet per second, the cell will turn pink to bring attention to the problem.
- **Time to Drain 97% of Inflow Volume (hours)** is calculated to determine how long it takes for 97% of the inflow volume to drain out through the outlet structure. In accordance with Colorado Revised Statute 37-92-602(8), the basin must continuously release or infiltrate at least 97% of the 5-year storm within 72 hours after the end of the event. The column "Find Time to Drain 97% of Inflow" is calculated for each storm event in the Modified Puls routing table (Cells B3091:EZ4531). The calculations first find the row in the table with the maximum ponding depth (column Stage @ O_1) and then for each subsequent row they start to check the storage volume at the end of each time increment (column $Storage_2$) as the basin drains down until the remaining storage volume is only 3% of the total inflow volume. Once the correct row is found, the corresponding time step in Column C is returned as the drain time in hours. If the WQCV, 2-year, or 5-year drain times exceed 72 hours, the cell will be highlighted pink to bring this to the

attention of the user or reviewer. If the drain time for any of the storm events exceeds 120 hours, the cell will be highlighted pink because the MHFD-Detention workbook is limited to 120 hours by the number of rows in the Modified Puls Routing table.

- **Time to Drain 99% of Inflow Volume (hours)** is calculated to determine how long it takes for 99% of the inflow volume to drain out through the outlet structure. In accordance with Colorado Revised Statute 37-92-602(8), the basin must continuously release or infiltrate at least 99% of the runoff from storms greater than a 5-year return period within 120 hours after the end of the event. The column "Find Time to Drain 99% of Inflow" is calculated for each storm event in the Modified Puls routing table (Cells B3091:EZ4531). The calculations first find the row in the table with the maximum ponding depth (column Stage @ O_1) and then for each subsequent row they start to check the storage volume at the end of each time increment (column $Storage_2$) as the basin drains down until the remaining storage volume is only 1% of the total inflow volume. Once the correct row is found, the corresponding time step in Column C is returned as the drain time in hours. If the drain time for any of the storm events exceeds 120 hours, the cell will be highlighted pink because of the State Statute and because the MHFD-Detention workbook is limited to 120 hours by the number of rows in the Modified Puls Routing table. Also, if the WQCV drain time does not match the target drain time (Cell B21 on Basin worksheet), the cell will be highlighted pink to bring this to the attention of the user or reviewer.
- **Maximum Ponding Depth (feet)** is calculated as the maximum value in the Modified Puls routing table in Column "Stage @ O_1" for each storm event. The program routes the inflow hydrograph through the basin and outlet structure and the maximum stage occurs after the peak of the inflow hydrograph has entered the basin and the release rate exceeds the remaining inflow rate.
- **Area at Maximum Ponding Depth (acres)** is calculated by looking up the maximum ponding depth from the previous row in the stage-area relationship (Cells B87:C3087) and returning the corresponding area (converted from square feet to acres).
- **Maximum Volume Stored (acre-feet)** is calculated by looking up the maximum ponding depth (Row 76) in the stage-area-volume relationship (Cells B87:D3087) and returning the corresponding volume (converted from cubic feet to acre-feet).

3.11 Hidden Stage-Storage-Discharge Table

The Stage-Storage-Discharge table is located in Cells B85:BL3087. Rows 84 through 3088 are initially hidden when a new workbook is opened but can be made visible by clicking on the *Show Stage-Storage-Discharge Table* button in Cell B82. The columns within the table are described in the bullet points below.

- **Stage (feet)** in Column B is provided in increments of 0.01-feet and is copied over from the hidden stage-area-volume table on the Basin worksheet (Cells F114:O3116). If the program calculated stage values are used, they are copied from Column G. If a user-override stage-area relationship was provided by the user, the values are copied from Column H and there may be empty rows at the bottom of the table if the total depth is less than 30 feet.

- **Area (square feet)** in Column C is copied over from the hidden stage-area-volume table on the Basin worksheet (Cells F114:O3116). If the program calculated area values are used, they are copied from Column K. If a user-override stage-area relationship was provided by the user, the area values are copied from Column L.
- **Volume (cubic feet)** in Column D is also copied over from the hidden stage-area-volume table on the Basin worksheet (Cells F114:O3116). The volume values are copied directly from Column N.
- **User Defined Discharge (cubic feet per second)** in Column E is optional but allows the user to provide a known stage-discharge relationship in the blue input cells. This relationship may be taken from a design report or different modeling software for purposes of evaluating the stage-storage-discharge relationship against MHFD recommendations.
- **2 * Volume (cubic feet)** in Column F is calculated as two times the Volume in Column D. This value is later used in Column Y of this table as part of the Modified Puls routing method calculations.
- **Filtration Media Orifice (cubic feet per second)** in Column G is the stage-discharge relationship for the underdrain orifice. The equations used in this column are explained in Section 3.3.
- **Orifice Plate (cubic feet per second)** in Column H is the stage-discharge relationship for the water quality orifice plate or elliptical slot, depending on which type was selected by the user. If a water quality orifice plate is being used, the total discharge through all orifice rows in the plate is copied from Column AR in the same table. If an elliptical slot is being used, the discharge through the elliptical slot is copied from Column BC in the same table. The equations used to calculate these discharges are explained in Sections 3.4 and 3.5, respectively.
- **Vertical Orifice #1 (cubic feet per second)** in Column I is the stage-discharge relationship for the first vertical orifice from Cells B33:B36. The equations used in this column are explained in Section 3.6.
- **Vertical Orifice #2 (cubic feet per second)** in Column J is the stage-discharge relationship for the second vertical orifice from Cells C33:C36. The equations used in this column are also explained in Section 3.6.
- **Overflow #1 Weir (cubic feet per second)** in Column K is the stage-discharge relationship for the first overflow weir from Cells B40:B45 when calculated as weir flow at shallow headwater depths. The equations used in this column are explained in Section 3.7.

- **Overflow #1 Orifice (cubic feet per second)** in Column L is the stage-discharge relationship for the first overflow weir from Cells B40:B45 when calculated as orifice flow for deep headwater depths. The equations used in this column are explained in Section 3.7.
- **Overflow #1 Mixed (cubic feet per second)** in Column M is the stage-discharge relationship for the first overflow weir from Cells B40:B45 when calculated as mixed flow for transitional depths between weir flow and orifice flow. The equations used in this column are explained in Section 3.7.
- **Overflow #1 Control (cubic feet per second)** in Column N is the controlling stage-discharge relationship for the first overflow weir from Cells B40:B45 which is calculated as the minimum discharge from the previous three columns (weir, orifice, and mixed). The equations used in this column are explained in Section 3.7.
- **Overflow #2 Weir (cubic feet per second)** in Column O is the stage-discharge relationship for the second overflow weir from Cells C40:C45 when calculated as weir flow at shallow headwater depths. The equations used in this column are explained in Section 3.7.
- **Overflow #2 Orifice (cubic feet per second)** in Column P is the stage-discharge relationship for the second overflow weir from Cells C40:C45 when calculated as orifice flow for deep headwater depths. The equations used in this column are explained in Section 3.7.
- **Overflow #2 Mixed (cubic feet per second)** in Column Q is the stage-discharge relationship for the second overflow weir from Cells C40:C45 when calculated as mixed flow for transitional depths between weir flow and orifice flow. The equations used in this column are explained in Section 3.7.
- **Overflow #2 Control (cubic feet per second)** in Column R is the controlling stage-discharge relationship for the second overflow weir from Cells C40:C45 which is calculated as the minimum discharge from the previous three columns (weir, orifice, and mixed). The equations used in this column are explained in Section 3.7.
- **Outlet Plate #1 Capacity (cubic feet per second)** in Column S is the stage-discharge relationship that represents the available capacity of the first outlet pipe with flow restriction plate (Cells B49:B51) assuming there are no upstream outlet structure components limiting the flow rate reaching the flow restriction plate. The equations used in this column are explained in Section 3.8.
- **Outlet Plate #1 Control (cubic feet per second)** in Column T is the stage-discharge relationship that represents the actual discharge through the first outlet pipe with flow restriction plate (Cells B49:B51) by accounting for the limited flow rate reaching the flow restriction plate due to upstream outlet structure components (e.g., water quality orifice plate, vertical orifice, and/or overflow weir). The equations used in this column are explained in Section 3.8.

- **Outlet Plate #2 Capacity (cubic feet per second)** in Column U is the stage-discharge relationship that represents the available capacity of the second outlet pipe with flow restriction plate (Cells C49:C51) assuming there are no upstream outlet structure components limiting the flow rate reaching the flow restriction plate. The equations used in this column are explained in Section 3.8.
- **Outlet Plate #2 Control (cubic feet per second)** in Column V is the stage-discharge relationship that represents the actual discharge through the second outlet pipe with flow restriction plate (Cells C49:C51) by accounting for the limited flow rate reaching the flow restriction plate due to upstream outlet structure components (e.g., first outlet plate in Column T, vertical orifice, and/or the second overflow weir). The equations used in this column are explained in Section 3.8.
- **Spillway (cubic feet per second)** in Column W is the stage-discharge relationship for the emergency spillway (Cells B54:B58). The equations used in this column are explained in Section 3.9.
- **Total Outflow (cubic feet per second)** in Column X is the combined stage-discharge relationship for the entire outlet structure. If the user provided an override stage-discharge relationship in Column E, then those values are copied to this column for subsequent calculations. Otherwise, the program determines which outlet structure components are included in the design and sums the appropriate columns to get the combined stage-discharge relationship.
- **O*dt + 2S (cubic feet)** in Column Y is used in the Modified Puls routing method. This column along with the next two columns Z and AA, are used to develop a nonlinear relationship called the storage-outflow function or routing relationship, which relates O*dt + 2S to outflow from the basin. Use of the storage-outflow function requires that characteristics of both the basin and outlet structure are known, which is why this column is included in the stage-storage-discharge table and not the Modified Puls routing table. Each row in this column is calculated by multiplying the total outflow (Column X) by the time interval (Cell C3089, default value is 5 minutes, multiplied by 60 to convert to seconds) and then adding two times the storage volume (Column F). The next section discussing the Modified Puls routing table will explain how this value is used to solve the continuity equation.
- **Outflow Slope** in Column Z is also used in the Modified Puls routing method to help interpolate the storage-outflow function described above. Each row of this column is calculated as the ratio of the change in outflow (Column X) between stage increments divided by the change in the storage-outflow function (Column Y) between stage increments. The equation is written as shown below.

$$Outflow\ Slope\ = \frac{O_j - O_{j-1}}{R_j - R_{j-1}}$$

Where:

O = outflow (cfs) in Column X

R = Storage-Outflow function in Column Y
j = current row value
j-1 = previous row value

- **Stage Slope** in Column AA is also used in the Modified Puls routing method to help interpolate the storage-outflow function described above. Each row of this column is calculated as the ratio of the change in stage (Column B) between stage increments divided by the change in the storage-outflow function (Column Y) between stage increments. The equation is written as shown below.

$$Stage\ Slope\ = \frac{H_j - H_{j-1}}{R_j - R_{j-1}}$$

Where:
H = stage (feet) in Column B
R = Storage-Outflow function in Column Y
j = current row value
j-1 = previous row value

- **Water Quality Orifice Plate, Rows 1-16 (cubic feet per second)** in Columns AB:AQ are used to calculate the stage-discharge relationship for each of the 16 potential rows in the water quality orifice plate. The equations used to calculate each orifice discharge are explained in Section 3.4.
- **Sum of Orifice Flow (cubic feet per second)** in Column AR is calculated for each row in the table as the sum of the values for each orifice row in Columns AB:AQ. The resulting value is then copied back to Column H when a water quality orifice plate is used in the outlet structure design.
- **Elliptical Slot Weir (cubic feet per second)** in Columns AS:BC are used to calculate the stage discharge relationship for the elliptical slot weir. A description of the equations used in each of these columns is explained in Section 3.5. The final discharge value is shown in Column BC and when an elliptical slot is used in the outlet structure design, these values are copied back to Column H instead of the orifice plate discharge values.
- **Flow Controlling Outlet** in Columns BD:BL are used to determine which component of the outlet structure is controlling the release rate for each stage increment in the table. In other words, at each stage elevation which component is the most restrictive and limiting the release rate. The calculations in the table result in binary values of 0 or 1, where cells with a 1 in them indicate the component controlling the release rate. The calculations for each component column are discussed below.
    - **Filtration Media Orifice** in Column BD returns a value of 1 when the total outflow (Column X) equals the filtration media orifice discharge (Column G).

- **Orifice Plate** in Column BE returns a value of 1 when the total outflow (Column X) minus the filtration media orifice discharge (Column G) equals the orifice plate discharge (Column H).
- **Vertical Orifice #1** in Column BF returns a value of 1 when the vertical orifice controls the release rate. In order to determine this though, the program needs to know where the vertical orifice ranks (1 through 3) relative to other outlet components. If the rank is one, then a value of 1 is returned when the total outflow (Column X) minus the filtration media orifice discharge (Column G) minus the orifice plate discharge (Column H) equals the vertical orifice discharge (Column I). If the rank is two, then a value of 1 is returned when the total outflow (Column X) minus the flow restriction plate discharge from the first dropbox (Column T) equals the vertical orifice discharge (Column I). If the rank is three, then a value of 1 is returned when the total outflow (Column X) minus the flow restriction plate discharge from the second dropbox (Column V) equals the vertical orifice discharge (Column I).
- **Vertical Orifice #2** in Column BG returns a value of 1 when the second vertical orifice controls the release rate. In order to determine this though, the program needs to know where the vertical orifice ranks (2 through 4) relative to other outlet components. If the rank is two, then a value of 1 is returned when the total outflow (Column X) minus the first vertical orifice discharge (Column I) minus the orifice plate discharge (Column H) minus the filtration media orifice discharge (Column G) equals the second vertical orifice discharge (Column J). If the rank is three and the first vertical orifice is rank one, then a value of 1 is returned when the total outflow (Column X) minus the flow restriction plate discharge from the first dropbox (Column T) equals the second vertical orifice discharge (Column J). However, if the rank is three but the first vertical orifice is rank two, then a value of 1 is returned when the total outflow (Column X) minus the flow restriction plate discharge from the first dropbox (Column T) minus the first vertical orifice discharge (Column I) equals the second vertical orifice discharge (Column J). If the rank is four and the first vertical orifice is rank one or two, then a value of 1 is returned when the total outflow (Column X) minus the flow restriction plate discharge from the first dropbox (Column T) equals the second vertical orifice discharge (Column J). However, if the rank is four but the first vertical orifice is rank three, then a value of 1 is returned when the total outflow (Column X) minus the flow restriction plate discharge from the first dropbox (Column T) minus the first vertical orifice discharge (Column I) equals the second vertical orifice discharge (Column J).
- **Overflow #1 Control** in Column BH returns a value of 1 when the first overflow weir controls the release rate. In order to determine this though, the program needs to know where the overflow weir ranks (1 through 3) relative to other outlet

components. If the rank is one, then a value of 1 is returned when the total outflow (Column X) minus the filtration media orifice discharge (Column G) minus the orifice plate discharge (Column H) equals the overflow weir discharge (Column N). If the rank is two, then a value of 1 is returned when the total outflow (Column X) minus the filtration media orifice discharge (Column G) minus the orifice plate discharge (Column H) minus the first vertical orifice discharge (Column I) equals the overflow weir discharge (Column N). If the rank is three, then a value of 1 is returned when the total outflow (Column X) minus the filtration media orifice discharge (Column G) minus the orifice plate discharge (Column H) minus the first vertical orifice discharge (Column I) minus the second vertical orifice discharge (Column J) equals the overflow weir discharge (Column N).

- **Overflow #2 Control** in Column BI returns a value of 1 when the second overflow weir controls the release rate. In order to determine this though, the program needs to know whether the second weir overlaps the first weir and where the second overflow weir ranks (2 through 4) relative to other outlet components. If the two overflow weirs overlap each other, then the rank is not important and a value of 1 is returned when the total outflow (Column X) minus the filtration media orifice discharge (Column G) minus the orifice plate discharge (Column H) minus both vertical orifice discharges (Columns I and J) minus the first overflow weir discharge (Column N) equals the second overflow weir discharge (Column R). If the two overflow weirs are offset though, then the rank is important. If the rank is two, then a value of 1 is returned when the total outflow (Column X) minus the flow restriction plate discharge from the first dropbox (Column T) equals the second overflow weir discharge (Column R). If the rank is three and the first overflow weir is rank two, then a value of 1 is returned when the total outflow (Column X) minus the flow restriction plate discharge from the first dropbox (Column T) equals the second overflow weir discharge (Column R). However, if the rank is three but the first overflow weir is rank one, then a value of 1 is returned when the total outflow (Column X) minus the flow restriction plate discharge from the first dropbox (Column T) minus the first vertical orifice discharge (Column I) equals the second overflow weir discharge (Column R). If the rank is four, and the first overflow weir is rank three, then a value of 1 is returned when the total outflow (Column X) minus the flow restriction plate discharge from the first dropbox (Column T) equals the second overflow weir discharge (Column R). If the rank is four, but the first overflow weir is rank two, then a value of 1 is returned when the total outflow (Column X) minus the flow restriction plate discharge from the first dropbox (Column T) minus the first vertical orifice discharge (Column I) equals the second overflow weir discharge (Column R). If the rank is four, but the first overflow weir is rank one, then a value of 1 is returned when the total outflow (Column X) minus the flow restriction plate discharge from the first dropbox (Column T) minus the first vertical orifice discharge (Column I) minus the second vertical orifice discharge (Column J) equals the second overflow weir discharge (Column R).

- **Outlet Pipe #1 Control** in Column BJ returns a value of 1 when the total outflow (Column X) equals the flow restriction plate discharge capacity (Column S).
- **Outlet Pipe #2 Control** in Column BK returns a value of 1 when the total outflow (Column X) equals the second flow restriction plate discharge capacity (Column U).
- **Spillway** in Column BL returns a value of 1 when the emergency spillway discharge (Column W) is greater than zero.

3.12 Hidden Modified Puls Routing Table

The Modified Puls (or Level Pool) routing table is located in Cells B3089:EZ4532. There is also a table to store the 6-hour inflow hydrographs generated by CUHP in Cells A4535:J4609. Rows 3089 through 4610 are initially hidden when a new workbook is opened but can be made visible by clicking on the *Show Routing Table* button in Cell D82.

Prior to discussing each column in the hidden routing table, an overview of the Modified Puls routing method will be provided. Modified Puls routing is a flow routing procedure used to determine the outflow hydrograph from a reservoir (e.g., detention basin) given an inflow hydrograph and the storage-outflow characteristics of the reservoir. In other words, Modified Puls routing relates the inflow, outflow, and storage of a reservoir over time. The relationship between the inflow, outflow, and storage of the reservoir is based on the continuity equation:

$$\frac{dS}{dt} = I(t) - O(t)$$

Where:

S = storage volume (cubic feet)
I = inflow rate (cubic feet per second)
O = outflow rate (cubic feet per second)
t = time (seconds)

The finite differences form, or stepwise form, of this equation can then be expressed as:

$$\frac{S_j - S_{j-1}}{\Delta t} = \frac{I_j + I_{j-1}}{2} - \frac{O_j + O_{j-1}}{2}$$

Where:

S = storage volume (cubic feet)
I = inflow rate (cubic feet per second)
O = outflow rate (cubic feet per second)

Δt = time interval (seconds)
j = current row value
j-1 = previous row value

However, although the inflow is typically known, the storage and outflow are both unknown. The finite differences form of the continuity equation may be rearranged so that the unknown variables are on the left side of the equation and the known variables are on the right side of the equation. This is based on the assumption that the variables for the previous time step, S_{j-1} and O_{j-1}, are known but that the variables for the current time step, S_j and O_j, are not. The rearranged equation takes the following form.

$$O_j \Delta t + 2S_j = \left(I_j + I_{j-1} - O_{j-1}\right)\Delta t + 2S_{j-1}$$

The storage and outflow are related through a nonlinear relationship called the storage-outflow function or routing relationship, which relates $O_j\,\Delta t + 2S_j$ to outflow. Use of the storage-outflow function requires that the characteristics of both the reservoir and outlet structure are known. The stage-storage-discharge table in Rows 87:3087 provides the necessary information and the storage-outflow function (also referred to as the R value) is calculated in Column Y of that table as discussed in the previous section. Since the left side of this equation equals the R value, the right side of the equation must also equal the R value. Once the storage-outflow function has been developed, it may be used along with the inflow hydrograph and initial conditions to determine the outflow hydrograph and storage volume of the reservoir over time. The hidden Modified Puls routing table in the MHFD-Detention workbook performs all of the calculations discussed above and the steps to do so are outlined below.

The first two columns of the routing table (B:C) provide a time series for the routing calculations. The default time interval is equal to five minutes as seen in Cell C3089, but can be overridden by the user in Cell V8. At the default time interval of five minutes, Column B will show the time series in hours from 0 to 120 hours and Column C will show the time series in minutes from 0 to 7,200 minutes.

The remainder of the routing table is split into groups of columns corresponding to each of the nine storm events. Each storm event has a total of 17 columns to perform the Modified Puls routing calculations. For example, routing of the WQCV event is calculated in Columns D through T, routing of the EURV event is calculated in Columns U through AK, and so on until you reach the routing calculations for the last storm event (500-year) in Columns EJ through EZ. The calculations within each storm event group are identical except that the first column of each group (inflow hydrograph, I_1) is unique to that particular storm event and the initial starting condition is different for the WQCV and EURV (brim full capacity) relative to the other design storms. Therefore, the description of the variables and calculations for each column discussed in the bullet points below apply to all nine storm event groups. The bullet numbers correspond with the 17 columns in each storm event group.

1. **Inflow Hydrograph, I_1 (cubic feet per second)** is the inflow rate at the beginning of the time interval and directly references the corresponding storm event inflow hydrograph provided in the inflow hydrograph table (Cells X8:AF80). By default, the WQCV and EURV inflow hydrographs are all zeros since these calculations assume that the initial volume starts at the brim full capacity. However, if the user provides an override inflow hydrograph for the WQCV or EURV, then the inflow hydrographs will be routed like the other design storm events. Regardless of the storm event, only 72 rows are available to be copied from the inflow hydrograph table to this column in Rows 3091:3163. The remaining rows in this column (Rows 3164:4531) are always equal to zero. At the bottom of the column in Row 4532, the total inflow volume (acre-feet) is calculated by summing the incremental flow rates in the column and multiplying by the time interval (in seconds) and then dividing by 43,560 to convert to acre-feet.
2. **Inflow, I_2 (cubic feet per second)** is the inflow rate at the end of the time interval and is set equal to the I_1 value from the next row. The very last cell in this column is set to zero since there is no I_1 value in the next row to reference.
3. **Outflow, O_1 (cubic feet per second)** is the outflow rate at the beginning of the time interval. The value in the first row is always set equal to zero based on the assumption that the initial condition has zero discharge. For all other rows, this value is set equal to the O_2 value in the previous row.
4. **Outflow, O_2 (cubic feet per second)** is the outflow rate at the end of the time interval and is calculated using the storage-outflow function (Cells Y87:Y3087) and outflow slope (Cells Z87:Z3087) developed in the stage-storage-discharge table. The outflow slope is used to help interpolate the storage-outflow function between stage increments and is calculated as the ratio of the change in outflow (ΔO) divided by the change in the storage-outflow function (ΔR) between each stage increment as shown in the equation below.

$$OS = \frac{O_j - O_{j-1}}{R_j - R_{j-1}}$$

   Where:

   OS = outflow slope (1/second)
   O = outflow (cubic feet per second)
   R = storage-outflow function (cubic feet)
   j = current row value
   j-1 = previous row value

   This equation can be rearranged to solve for the unknown outflow by using the calculated storage-outflow function R value (described in #8 below) and linear interpolation using index values from the stage-storage-discharge table corresponding to the calculated R value. The rearranged equation is shown below.

$$O_2 = O_{index} + OS_{index}(R_{calc} - R_{index})$$

Where:

O_2 = outflow at end of time interval (cubic feet per second)

R_{calc} = calculated storage-outflow function R value determined in #8 below

OS_{index} = index outflow slope determined in #10 below (1/second)

O_{index} = index outflow determined in #12 below (cubic feet per second)

R_{index} = index R value determined in #13 below (cubic feet)

The index values are determined by finding the row (#11 below) in the stage-storage-discharge table that most closely matches the calculated R value and then returning the corresponding outflow, outflow slope, and R values from that row. At the bottom of the column in Row 4532, the maximum outflow value in the column is calculated.

5. **Storage, S_1 (cubic feet)** is the storage volume at the beginning of the time interval. The value in the first row of this column represents the initial storage volume in the detention basin and is assumed to be zero when an inflow hydrograph is being evaluated. However, for the WQCV and EURV (Cells H3091 and Y3091, respectively), the first row is set equal to the brim full storage volume for these events. For the remaining rows in this column, the S_1 value is set equal to the S_2 value in the previous row.
6. **Storage, S_2 (cubic feet)** is calculated by solving the left-hand side of the rearranged finite differences form of the continuity equation discussed above and shown below.

$$O_2 \Delta t + 2S_2 = R$$

Where:

R = storage-outflow function value (cubic feet), #8 below

O_2 = outflow at end of current time step (cubic feet per second), #4 above

S_2 = unknown storage volume at end of current time step (cubic feet)

Δt = time interval (seconds)

The equation above can be rearranged to solve for S_2 using the calculated values of O_2 and R discussed in #4 and #8, respectively. At the bottom of the column in Row 4532, the maximum storage value in the column is calculated.

7. **Storage, S_2 (acre-feet)** is calculated by dividing the value in the previous column (#6) by 43,560 to convert from cubic feet to acre-feet. At the bottom of the column in Row 4532, the maximum storage value in the column is calculated.
8. **Storage-Outflow Function, R (cubic feet)** is calculated by solving the right-hand side of the rearranged finite differences form of the continuity equation discussed above. The right-hand side of the equation shown below includes known values already calculated in the current row of the table.

$$R = (I_2 + I_1 - O_1)\Delta t + 2S_1$$

Where:

R = storage-outflow function value (cubic feet)

I_1 = inflow at beginning of current time step (cubic feet per second), #1 above

I_2 = inflow at beginning of next time step (cubic feet per second), #2 above
O_1 = outflow at end of previous time step (cubic feet per second), #3 above
S_1 = storage volume at end of previous time step (cubic feet), #5 above
Δt = time interval (seconds)

If the result of this equation is negative, the program will revert to a value of zero.

9. **Stage at O_1 (feet)** is the stage of the water surface elevation at the beginning of the time interval. The first row in this column is typically equal to the stage value at the top of the stage-storage-discharge table in Cell B87. However, if evaluating the WQCV or EURV, the initial stage is set equal to the water surface elevation when the detention basin is at brim full capacity. For the remaining rows in the routing table, the stage is calculated using the storage-outflow function (Cells Y87:Y3087) and stage slope (Cells AA87:AA3087) developed in the stage-storage-discharge table. The stage slope is used to help interpolate the storage-outflow function between stage increments and is calculated as the ratio of the change in stage (ΔH) divided by the change in the storage-outflow function (ΔR) between each stage increment as shown in the equation below.

$$SS = \frac{H_j - H_{j-1}}{R_j - R_{j-1}}$$

Where:

SS = stage slope (1/square feet)
H = stage (feet)
R = storage-outflow function (cubic feet)
j = current row value
j-1 = previous row value

This equation can be rearranged to solve for the unknown stage by using the calculated storage-outflow function R value (described in #8 below) and linear interpolation using index values from the stage-storage-discharge table corresponding to the calculated R value. The rearranged equation is shown below.

$$H_2 = H_{index} + SS_{index}(R_{calc} - R_{index})$$

Where:

H_2 = stage at end of time interval (feet)
R_{calc} = calculated storage-outflow function R value determined in #8 below
R_{index} = index R value determined in #13 below (cubic feet)
H_{index} = index stage determined in #14 below (cubic feet per second)
SS_{index} = index stage slope determined in #15 below (1/square feet)

The index values are determined by finding the row (#11 below) in the stage-storage-discharge table that most closely matches the calculated R value and then returning the corresponding stage, stage slope, and R values from that row. At the bottom of the column in Row 4532, the maximum ponding depth in the column is calculated.

10. **Index Outflow Slope** returns the outflow slope from the stage-storage-discharge table (Cells Z87:Z3087) using the index function and the row number determined in the column labeled Match Stage Row (#11) discussed below.
11. **Match Stage Row** determines the row in the stage-storage-discharge table that corresponds to the calculated storage-outflow function R value. In other words, the calculated R value for each row of the routing table (#8 above) is matched with the closest value found in the storage-outflow function (Cells Y87:Y3087) located in the stage-storage-discharge table. The resulting row number is then used to find index values for Outflow Slope (#10), Outflow (#12), R (#13), Stage (#14), and Stage Slope (#15).
12. **Index Outflow** returns the outflow from the stage-storage-discharge table (Cells X87:X3087) using the index function and the row number determined in the column labeled Match Stage Row (#11) discussed above.
13. **Index R** returns the storage-outflow function value R from the stage-storage-discharge table (Cells Y87:Y3087) using the index function and the row number determined in the column labeled Match Stage Row (#11) discussed above.
14. **Index Stage** returns the stage from the stage-storage-discharge table (Cells B87:B3087) using the index function and the row number determined in the column labeled Match Stage Row (#11) discussed above.
15. **Index Stage Slope** returns the stage slope from the stage-storage-discharge table (Cells AA87:AA3087) using the index function and the row number determined in the column labeled Match Stage Row (#11) discussed above.
16. **Find Time to Drain 97% of Inflow (hours)** is used to calculate the time increment when 97% of the total inflow volume has drained out of the detention basin. The calculations first find the row in the routing table with the maximum ponding depth by checking the column labeled Stage at O_1 (described in #9 above). Then the program checks each subsequent row in the table to compare the storage volume (acre-feet) at the end of each time increment (S_2 as described in #7 above) as the basin drains until the remaining storage volume is only 3% of the total inflow volume. Once the correct row where the volume drops below 3% is found, the corresponding time step from Column C is returned as the drain time in hours.
17. **Find Time to Drain 99% of Inflow (hours)** is used to calculate the time increment when 99% of the total inflow volume has drained out of the detention basin. The calculations first find the row in the routing table with the maximum ponding depth by checking the column labeled Stage at O_1 (described in #9 above). Then the program checks each subsequent row in the table to compare the storage volume (acre-feet) at the end of each time increment (S_2 as described in #7 above) as the basin drains until the remaining storage volume is only 1% of the total inflow volume. Once the correct row where the volume drops below 1% is found, the corresponding time step from Column C is returned as the drain time in hours.

Below the Modified Puls routing table, there is another table (Cells A4535:J4609) to store the 6-hour inflow hydrographs generated by CUHP. This hidden table allows the program to compare the program generated CUHP hydrographs against user input override hydrographs in the inflow hydrograph table (Cells X8:AF80). It also allows the program to quickly repopulate the inflow hydrograph table if the user wants to revert back to the original CUHP hydrographs by clicking the Reset Hydrographs button in Cell W2.

3.13 Figures

The second printed page of the Outlet Structure worksheet (Columns K:U) includes three charts.

The first chart plots the inflow and outflow hydrographs for each storm event with time (hours) on the X-axis and flow (cfs) on the Y-axis. Data for the plot is obtained from the hidden Modified Puls routing table (Cells B3089:EZ4532). Time (hours) on the X-axis is pulled from Column B. The inflow hydrographs for each storm event are pulled from the Inflow_1 column (#1 in Section 3.12). The outflow hydrographs for each storm event are pulled from the Outflow_2 column (#4 in Section 3.12). This chart allows the user to visualize the attenuation of each storm event as it passes through the basin.

The second chart plots the ponding depth over time for each storm event with time (hours) on the X-axis and ponding depth (feet) on the Y-axis. Data for the plot is obtained from the hidden Modified Puls routing table (Cells B3089:EZ4532). Time (hours) on the X-axis is pulled from Column B. The ponding depths for each storm event are pulled from the Stage @ O_1 column (#9 in Section 3.12). This chart allows the user to visualize the maximum ponding depth and total drain time for each storm event. The WQCV and EURV ponding depths start at a maximum value (brim full capacity) and drain down over time, whereas the design storm ponding depth starts at zero and increases as the inflow hydrograph enters the basin and then decreases again as the basin drains out.

The third chart plots the stage-area-volume-discharge relationship for the basin design. The ponding depth or stage (feet) in the basin is on the X-axis. There are two Y-axis scales to allow plotting of different variables with different units of measurement. The left Y-axis represents both area (square feet, green) and storage volume (cubic feet, red). The right Y-axis represents the outflow (cubic feet per second, blue) through the combined outlet structure. The chart can plot up to seven different time series, depending on the input provided by the user.

The three primary time series on the third chart are shown as solid lines and include interpolated area, volume, and outflow. Data for these time series are obtained from the hidden stage-storage-discharge table (Cells B85:BL3087). Stage (feet) on the X-axis is pulled from Column B. Interpolated area (square feet) on the left Y-axis is pulled from Column C. Volume (cubic feet) on the left Y-axis is pulled from Column D. Outflow (cubic feet per second) on the right Y-axis is pulled from Column X which is the total combined outflow for all components included in the outlet structure.

The remaining four time series on the third chart are only included when the user provides additional information. The user area (square feet) is plotted using hollow green circles and represents user override stage-area pairs entered on the Basin Worksheet in Cells H11:H109 and Cells L11:L109, respectively. The summary area, summary volume, and summary outflow values are plotted using solid circles connected by dashed lines and represent the user input stage values provided in the summary stage-area-volume-discharge relationship table on the Outlet Structure worksheet in Cells AH8:AO80. Summary stage (feet) on the X-axis is pulled from Column AJ. Summary area (square feet) on the left Y-axis is pulled from Column AK. Summary volume (cubic feet) on the left Y-axis is pulled from Column AM. Summary outflow (cubic feet per second) on the right Y-axis is pulled from Column AO which is the total combined outflow for all components included in the outlet structure.

The third chart allows the user to visualize the stage-area-volume-discharge relationships and compare the summary table values against the full table values to make sure that all transitions or changes in slope are adequately accounted for in the summary table values. The user also has the ability to zoom in on specific parts of the third chart by changing the minimum and maximum bounds plotted on the three axes in Cells M80:O81 and clicking the *Update S-A-V-D Chart* button. This allows the user to customize the chart for printing purposes or to evaluate slope transitions in more detail.

3.14 Inflow Hydrographs

The third printed page of the Outlet Structure worksheet (Columns V:AF) is where the inflow hydrographs for each of the nine storm events are shown. By default, the source for each of the inflow hydrographs is the CUHP-Lite results generated on the Basin worksheet and discussed in Section 2.2. The default time interval in Cell V8 is set equal to five minutes based on the CUHP results. By default, the WQCV and EURV inflow hydrographs are all zeros since the program routes these events starting at brim full capacity. The 2-year through 500-year storm events all default to the CUHP inflow hydrographs. The user has the ability to override any of these inflow hydrographs by copying and pasting new values into these cells. When inflow hydrograph values are overridden, the source at the top of the column (Row 6) will change to USER and the cell will be highlighted pink to make it clear the values have been overridden. Similarly, if the user changes the time interval in Cell V8, this cell will also be highlighted pink. If the user has made changes to the inflow hydrographs but would like to revert back to the CUHP inflow hydrographs, the *Reset hydrographs to default values from CUHP* button can be clicked and the original values stored in Cells A4535:J4609 below the hidden Modified Puls routing table will be copied back to this table.

The MHFD-Detention workbook also has the ability to export the calculated outflow hydrographs from the Modified Puls routing table to a separate Excel workbook for later use. In order to utilize this function, the user must provide the filename and file path of the new Excel workbook in Cells AB2:AF2. This can be done by clicking the three-dot button in Cell AF2 which will bring up a File Explorer window where the user can specify the filename and location. If the box in Cell AA3 is checked, the file path will only include the file position relative to the current workbook. If the box is not checked, the full file path will be included. Once a valid filename and location are provided, the user can click on the button labeled *Export Outflow Hydrographs to a blank workbook for later use in a downstream MHFD-Detention Workbook* and the program will copy the $Outflow_2$ hydrographs (#4 in Section 3.12) from the Modified Puls routing table for each storm event to a new workbook.

3.15 Summary Stage-Area-Volume-Discharge Relationships

The fourth printed page of the Outlet Structure worksheet (Columns AG:AQ) allows the user to create a summary stage-area-volume-discharge relationship at stage increments of their choice. This table is not necessary for the rest of the workbook to run but is useful for generating report tables and for focusing on important elevations such as slope transitions or a change in the controlling outlet structure component. The Stage-Storage Description in Columns AH:AI is an optional input but does help to explain important transition points or key design storm elevations. The Stage (feet) in Column AJ is the required input column. When stage values are entered, the corresponding area, volume, and outflow values are automatically populated in Columns AK:AO using a lookup function to find the values in the stage-storage-table (Columns C, D, and X, respectively). For best results, the user should make sure to include stage values associated with grade slope changes (e.g., ISV and basin floor) and the invert elevations of outlet structure components (e.g., orifice plate, vertical orifice, overflow grate, and emergency spillway). The user should then graphically compare the summary table values to the full stage-area-volume-discharge table values by viewing the third chart discussed in Section 3.14 to confirm all key transition points were captured.

4 Reference Worksheet

The Reference worksheet is intended to provide quick access to figures and equations so that the user does not always have to refer to this manual or the Urban Storm Drainage Criteria Manual. Below is a brief description of the various figures and equations included on the Reference worksheet.

- **Circular Pipe with Restrictor Plate –** This figure provides a diagram of the circular outlet pipe with flow restrictor plate and shows how to measure the different variables used in the Outlet Structure worksheet. These variables include the outlet pipe diameter, Dia (Cell B50), restrictor plate height above pipe invert, Y_O (Cell B51), half-central angle of restrictor plate on pipe, θ (Cell H51), and top width of flow, T_O. This information is described in more detail in Section 3.8 of this manual.
- **WQ Elliptical Slot Weir –** This figure provides a diagram of the elliptical slot weir with descriptions of the labeled variables and several of the equations used in the Outlet Structure worksheet. These variables include the elliptical slot height, H (Cell B18),

elliptical slot gap width, t (Cell B19), and elliptical slot axis ratio, H/W (Cell B20). This information is described in more detail in Section 3.5 of this manual.

- **Outlet Structure –** This figure (located below the elliptical slot weir figure) shows a typical outlet structure design recommended by the MHFD for use on the Outlet Structure worksheet. This figure shows a concrete dropbox vault with a vertical trash rack and WQCV/EURV orifice plate covering an opening in the front wall, a sloped overflow weir with trash rack on top, and a 100-year outlet pipe with flow restrictor plate in the back wall. Several variations of this design can be implemented in the workbook, but this figure helps to visualize the various components.
- **WQCV and EURV Equations –** This box provides the MHFD equations for calculating WQCV and EURV. These are the equations are used on the Basin worksheet in Cells B26 and B27, respectively. These equations are discussed in more detail in Section 2.3, Appendix A, and Appendix B of this manual.
- **Approximate Storage Volume Equations –** This box provides the empirical equations used to estimate approximate detention volumes based on the watershed input parameters provided by the user. These are the equations used on the Basin worksheet in Cells B35:B40. These equations are discussed in more detail in Section 2.3 and Appendix C of this manual.
- **Basin Volume Calculations –** This box and adjacent figures provide the equations used to calculate the basin geometry for an extended detention basin including the initial surcharge volume, trickle channel, basin floor volume, and main basin volume. These are the equations used on the Basin worksheet in Cells B55:B68. These equations are discussed in more detail in Section 2.6 and Appendix D of this manual.
- **Default Horton's Equation Parameters and Default Depression Storage –** These two boxes provide the default parameters used by the workbook when generating hydrology using the CUHP-Lite code. These parameters are discussed in more detail in Section 2.2.1 of this manual.
- **Overflow Grate Types –** The figures at the bottom of the worksheet show the three types of overflow grates that can be selected on the Outlet Structure worksheet in Cell B44. The grate types include a close mesh grate, a CDOT Type C grate (bar grate), and an open grate (no grate included). These grate types are discussed in more detail in Section 3.7 and Appendix H of this manual.

5 User Tips and Tools Worksheet

The User Tips and Tools worksheet provides supporting information to assist with using the MHFD-Detention workbook. At the top of the worksheet is a link to the MHFD YouTube Channel where several instructional videos are provided. These videos range from basic overviews of the workbook to detailed examples on specific topics such as hydrology overrides, stage-area overrides, underground detention, etc. The next section of this worksheet provides user tips specific to the Basin worksheet and the Outlet Structure worksheet. Many of these tips are discussed in more detail in the YouTube instructional videos.

At the bottom of this worksheet (Cells B39:J92) is a tool that allows the user to back calculate an approximate stage-area relationship from a known stage-volume relationship. This tool is useful because the MHFD-Detention workbook does not allow for direct entry of a stage-volume relationship on the Basin worksheet. The workbook calculates volume using the conic approximation method because it better represents the conical shape of most detention basins (as opposed to the average end area method). The conic approximation method calculates the incremental volume between two horizontal slices through the basin; the two areas being added along with the square root of their product and multiplied by a third of the distance between the slices to determine the incremental volume between the slices, as expressed in the following equation.

$$V_2 = V_1 + \left(\frac{H_2 - H_1}{3}\right) * \left(A_1 + A_2 + \sqrt{A_1 * A_2}\right)$$

Where:

V_2 = Volume at top of slice (cubic feet)
V_1 = Volume at bottom of slice (cubic feet)
H_2 = Stage at top of slice (feet)
H_1 = Stage at bottom of slice (feet)
A_2 = Area at top of slice (square feet)
A_1 = Area at bottom of slice (square feet)

Unfortunately, if you only have the stage-volume relationship, it is very difficult to rearrange the equation and solve for area because there are two unknowns, A_1 and A_2. Therefore, a sizing tool was developed to allow the program to use an iterative procedure to solve for the area values by using a code routine to minimize the sum of squared differences (SSD) between sequential area values in order to create a smooth stage-area relationship that closely matches the program calculated volumes to the known volumes entered by the user. The chart to the right of the table shows the resulting stage-area and stage-volume relationships, and helps to identify large oscillations in the calculated area values. This method will typically not produce a perfect fit due to rounding errors in the known stage-volume relationship, but is usually good enough for use on the Basin worksheet.

The sizing tool is organized in a table format with each column of the table explained in the bullet points below.

- **Stage-Storage Description** in Column B is optional and allows the user to provide a brief description of important stage elevations. The first row is always assumed to be the basin bottom.

- **User Stage (feet)** in Column C is for user input stage values and is required for the sizing tool to run. The first stage value is always set to zero. There must be an equal number of stage and volume values for the program to run.
- **User Volume (acre-feet)** in Column D is for user input volume values and is required for the sizing tool to run. The first volume value is always set to zero. There must be an equal number of stage and volume values for the program to run.
- **User Volume (cubic feet)** in Column E just converts the values in Column D to acre-feet by multiplying by 43,560.
- **Calculated Volume (cubic feet)** in Column F is calculated using the conic approximation method equation shown above where the stage values are pulled from Column C and the area values are iteratively calculated by the program in Column G.
- **Calculated Area (square feet)** in Column G is iteratively calculated by the sizing routine to minimize the sum of squared differences between sequential area values in order to avoid oscillations and create a smooth stage-area relationship that matches the target volumes at each stage. The first row in this column (Cell G42) can be manually entered by the user to provide a known starting point in the sizing routine. However, it can also be left blank and the program will run through several iterations in an attempt to find a bottom area that provides the best fit to the volume curve.
- **Calculated Area (acre)** in Column H just converts the values in Column G to acres by dividing by 43,560.
- **Volume % Difference (percent)** in Column I calculates the difference between the known volume (Column E) and the calculated volume (Column F) and then divides by the known volume to get the percent difference.
- **Square Difference in Area** in Column J calculates the difference between the calculated area (Column G) in the current row and the calculated area in the previous row and then squares the difference. The first row in this column cannot be calculated and is always blank. The sum of the squared differences is calculated at the top of the column (Cell J38) by adding up all the squared differences in Cells J43:J92.

In order to use the sizing tool, the user must provide stage-volume pairs in Columns C and D. If the user provides the bottom area (and there are an equal number of stage and volume values), the program will automatically run the sizing routine to determine the area at each stage increment. If the user clicks the *Minimize SSD* button, the program will iteratively solve for the bottom area that provides the best overall fit to the known volume values. When the program is finished running, the user should check the chart to the right of the table to make sure the stage-area relationship doesn't have large oscillations which may indicate a potential problem in the known stage-volume pairs. It is also possible that the program may not be able to find a solution. In either case, check the user input values for stage and volume to identify any potential input mistakes or unreasonable values that may be causing problems. Next, the user can try manually changing the bottom area to see if a better fit can be found.

Once a good fit has been found and a reasonable stage-area relationship has been developed, the user has the option to click the *Copy Stage-Area Results to Basin Worksheet* button. When this button is clicked, the program will copy the Stage-Storage Description (Column B), User Stage (Column C), and Calculated Area (Column G) from the User Tips and Tools worksheet and then copy these values to the stage-area-volume summary table on the Basin worksheet in Columns F, H and L, respectively.

6 BMP Zone Images Worksheet

The last worksheet in the MHFD-Detention workbook shows various configurations of different BMP types with respect to the three storage zones. These figures help to visualize potential designs that may include WQCV only, WQCV and EURV, full spectrum detention, or flood control only. For a more detailed discussion of the different zones, please refer to Section 3.4 of the Storage chapter in the USDCM.

Appendix A – Sizing a Capture Volume for Stormwater Quality Enhancement

This article first appeared in the Urban Drainage and Flood Control District's 1989 edition of Flood Hazard News. Except for minor editorial corrections and enhancement of the figures, it has not been revised. It has served as one of the technical documents for the sizing of the water quality capture volume recommendations in Volume 3 of the USDM.

SIZING A CAPTURE VOLUME FOR STORMWATER QUALITY ENHANCEMENT

by

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INTRODUCTION

Urban stormwater management is rapidly changing from a focus only on the control of damages resulting from storm runoff to now include water quality. Two basic issues are influencing this change. First is a fundamental heightening of environmental awareness and concern by the public. It is documented that urban stormwater, along with non-point runoff from non-urban sources, contributes pollutants to the receiving waters and efforts to do something about it are slowly picking up support and momentum.

The second factor causing a shift toward urban stormwater quality is the Water Quality Act of 1987 (WQA), which amended the Federal Water Pollution Control Act. How this WQA may impact the citizens, communities, local governments, industry, consultants and the water quality across the United States is yet to be seen. Nevertheless, local governments and industries are mandated by Congress to control pollutants in urban runoff to the "maximum extent practicable" (MEP). This hopefully means that Congress expects solutions to be practical, pragmatic, and economical.

In order to be practical and effective it is important that technologies for dealing with urban stormwater runoff be available. Several simple technologies are emerging (Urbonas and Roesner, 1986), (Roesner, Urbonas and Sonnen, 1989), which include detention and retention basins, infiltration and percolation at the source of runoff, wetlands, sand filters, and combinations of these techniques. It is clear from the references cited above that stormwater quality facilities first need to capture a certain volume of runoff in order to treat it. As a result, the size of runoff event to be captured is critical in the design of stormwater quality facilities. For example, if the design runoff event is too small, the effectiveness will be reduced because too many storms will exceed the capacity of the facility. On the other hand, if the design event is too large, the smaller runoff events will tend to empty faster than desired for adequate treatment to take place. We know that large detention basins designed to control peaks from larger storms will not provide the needed retention time for the smaller events, which are much more numerous than the larger storms.

A balance between the storage size and water quality treatment effectiveness is needed. Grizzard et. al. (1986) reported results from a field study of basins with extended detention times in the Washington, D. C. area. Based on their observations they suggested that these basins provide good levels of treatment when they are sized to have an average drain time for all runoff events of 24 hours. This equates to a 40 hour drain time for a brim-full basin. Beyond that, there remains little rationale for the sizing of the capture volume that results in reasonable pollutant load removal while providing reasonably sized facilities.

This article first appeared in the Urban Drainage and Flood Control District's 1989 edition of Flood Hazard News. Except for minor editorial corrections and enhancement of the figures, it has not been revised. It has served as one of the technical documents for the sizing of the water quality capture volume recommendations in Volume 3 of the USDM.

This paper will discuss one possible method to find a point of diminishing returns for the sizing of water quality capture volume. It utilizes rainstorm records as its base instead of synthesized design storms. An example based on the National Weather Service long term precipitation record in Denver will illustrate this methodology.

FINDING A POINT OF DIMINISHING RETURNS

In 1976 Von den Herik (1976) suggested in Holland a rainfall-databased method for estimating runoff volumes which he called Rain Point Diagram (RPD). This was later modified to a Runoff Volume Point Diagram (RVPD) method, which approximates continuous modeling without setting up a continuous model. The method requires combining individual recorded hourly or 15-minute rainfall increments in a given period of record into separate storm depths. Individual storms are defined by the time during which no rainfall occurs. Very small storms are purged from the record. Storm totals are converted to runoff volumes by multiplying each storm's depth by the watershed's runoff coefficient (C).

The use of the RVPD is illustrated in Figure 1, where the individual storm runoff depth is plotted against storm duration. The runoff capture envelope consists of the "brim-full" volume of the detention facility, plus the average release rate times its emptying time. In this figure the runoff capture envelope is based on a detention basin that has a brim-full capacity of 0.3 watershed inches which can be emptied in 12 hours. All the points above the capture envelope line represent individual storms that have sufficient runoff to exceed the available storage volume (i.e., brim-full volume) of the detention facility. A software package was developed to perform this analysis and to report the results after testing a variety of capture volumes.

For the storm events in a given record there is a capture volume that will intercept all runoff within the record. For practical reasons this maximum pond volume, P_m, was defined to be equal to the 99.5 percent probability runoff event volume for the period of record. For the Denver rain gauge period of record studied (1944-1984), P_m is equal to the runoff from 3.04 inches of precipitation, or 6.9 times the precipitation of an average runoff producing storm for this same period of record. This value of P_m was then used to normalize all pond sizes being tested using the following equation:

$$P_r = P / P_m \qquad (1)$$

in which, P_r = relative pond size normalized to P_m

P = pond size being tested

P_m = maximum runoff volume (i.e., 99.5% probability).

The search for the point of diminishing returns, sometimes called "maximized point," incrementally increases the relative (i.e., normalized) pond size and calculates runoff volume and the number of events. Figure 2 illustrates an example of the results of such an analysis for the following conditions: storm separation criteria is 6-hours, emptying time for the brim-full basin is 12-hours, and the runoff coefficient for the watershed is C = 0.5.

The maximized pond size occurs where the 1:1 slope is tangent to the runoff capture rate function. Before this point is reached the capture rate increases faster than the relative capture volume size. After this point is reached the increases in the capture rate become less than corresponding increases in relative capture volume size. In other words, when the point of maximization is passed, diminishing returns are experienced if the capture volume is increased any further. Or the example illustrated in Figure 2, the maximized point occurs when the relative capture volume is equal to 0.18, which converts to 0.27 watershed inches.

A statistical summary of rainfall characteristics for all storms that exceeded a total of 0.1 inch at the Denver Rain Gauge is given in Table 1. A 0.1-inch "filter" was used to eliminate from the record the very small storms, which are not likely to produce runoff (see "Incipient Runoff Value of Rainfall in the Denver Region" in this issue of *Flood Hazard News* concerning the point of incipient runoff in the Denver area).

You can see from this summary that the rainfall exhibits a skewed statistical distribution. More than two-thirds of the storms have less precipitation than the average storm. Apparently in the Denver area the average runoff producing rainstorm depth is a relatively large event.

Once the precipitation and runoff probabilities were understood, an attempt was made to find a simple yet reasonably accurate relationship for approximating the maximized capture volume for water quality facilities. The final result for the Denver rain gauge data is illustrated in Figure 3. This figure relates the maximized capture volume to the watershed's runoff coefficient. Separate relationships are shown for the brim-full storage volume emptying time of 12-, 24- and 40-hours.

The capture volume found using these curves will result in 86 percent of all runoff events being totally captured and processed by the facility. It is the frequency of the shock loads that has the greatest negative effect on the aquatic life in the receiving streams. On the other hand, the very few large storms in the record are responsible for all of the flooding damages. Even during these larger events some degree of capture and treatment occurs, even though it may be at somewhat reduced efficiency.

SENSITIVITY OF PROCEDURE

An attempt was made to test the sensitivity of the capture volume as a surcharge above a permanent pool level on the removal rates of total suspended solids. For lack of local data on sediment settling velocities, the data given by EPA (1986) were use for several capture volume sizes. Estimates were made of the dynamic removals that occur while the runoff event is occurring and for the quiescent removals in the pond that occur between runoff events.

When theses estimates were made using a capture volume equal to 70% of the maximized volume, the annual removal of TSS was estimated at 86%. This compares to an estimated rate of 88% for the maximized capture volume and 90% for a volume that is twice as large as the maximized volume. In other words, the removal efficiencies appear to be very insensitive to an increasing the capture volume beyond the capture of the runoff from the 70th percentile event.

It thus appears possible to use a lesser capture volume above the wet detention pond water surface than the maximized volume and have virtually no effect on the TSS removal efficiency. Currently we suggest that the design volume could be based on the capture of an 80th percentile runoff event instead of the maximized volume. Obviously this suggestion needs further testing. In the meantime, Figure 4 may be used to size the surcharge capture volume for ponds with permanent pools of water and Figure 5 may be used to size a capture volume for a detention facility that drains completely.

On the other hand, if the removal of dissolved nutrients, such as phosphorous or nitrates, is desired, the designer has to consider using wet ponds or a marsh. Biologic activity is responsible for the removal of dissolved constituents. The effectiveness of these processes is primarily the function of residence time within the permanent water pool. Increasing the capture volume above this pool will have little effect on the removal efficiencies of dissolved compounds. On the other hand, dry ponds have little effect on the removal of dissolved materials, since their primary removal mechanism is sedimentation (Grizzard, et. al., 1986; Schueler, 1987; Roesner, et. al., 1988; Stahre and Urbonas, 1988) the quiescent removals in the pond at 86 percent. This compares to an between storms estimated rate of 88 percent annual. Using a capture volume equal to 70 removal of TSS when using the percent of the maximized volume, the maximized capture volume, and to a annual removal of TSS was estimated 90 percent removal rate when using

DETERMINATION OF RUNOFF COEFFICIENT

In 1982 EPA published data as part of the NURP study on rainfall depth vs. runoff volume. Although EPA did acknowledge some regional differences, much of the United States was found to be well represented by the data plotted in Figure 6. The curve in this figure is a third order regressed polynomial with the regression coefficient $R^2 = 0.79$. This value of R^2 implies a reasonably strong correlation between the watershed imperviousness, I, in percent and the runoff coefficient, *C*, for the range of data collected by EPA. Since the NURP study covered only two-year period, in our opinion this relationship is justified for 2-year recurrence probability and smaller storms.

CONCLUSIONS

An investigation of sizing stormwater quality facilities for maximized capture of stormwater runoff events and their performance in removing settleable pollutants revealed that simplified design guidelines arc possible. These guidelines can be developed using local or regional rain gauge records. Preliminary suggestions for such guidelines are illustrated in Figures 4 and 5 for the Denver area and areas having similar rain and snowstorm patterns.

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This article is a condensation of a paper presented at an Engineering Foundation Conference in Davos, Switzerland in October, 1989. The full paper will be printed in conference proceedings to be published by the American Society of Civil Engineers in 1990.

TABLE 1. Denver Rain Gauge Hourly Data Summary For Storms Larger Than 0.1 Inches In Depth

Separation Basis For New Storm (Hours)	Number of Storms	Average *Depth* (Inches)	Average Storm Duration (Hours)	Average Time Between Storms (Hours)	Percent of Storms Smaller Than Average
1	1131	0.39	7	267	70.9
3	1091	0.42	9	275	71.7
6	1084	0.44	11	275	70.7
12	1056	0.46	14	280	70.8
24	983	0.51	23	293	69.8
48	876	0.58	43	310	70.0

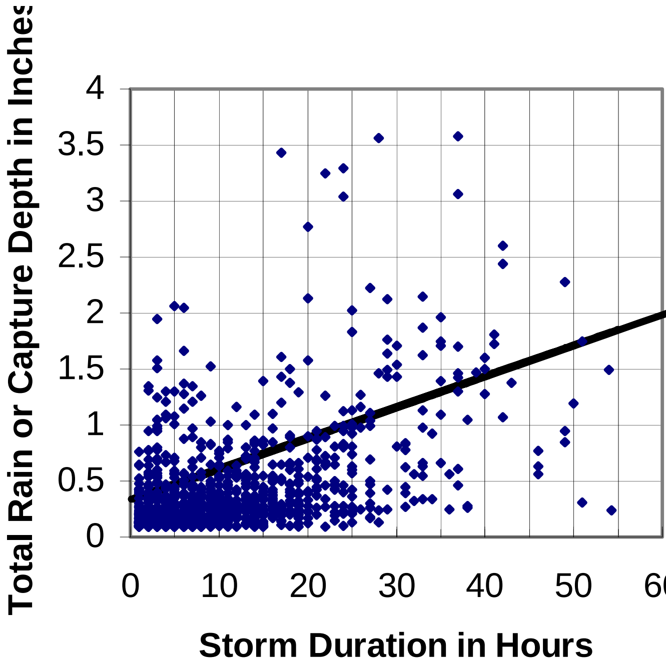

Figure 1. Example of a Capture Volume Envelope for a 0.33-inch Basin Volume and an Outlet Sized to Drain This Volume in 12-hours.

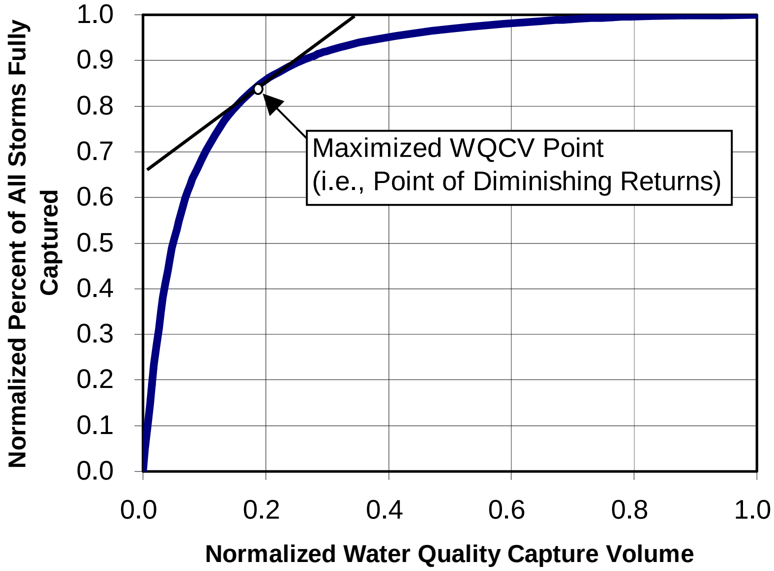

Figure 2. Point of Maximization of WQCV - Denver Example

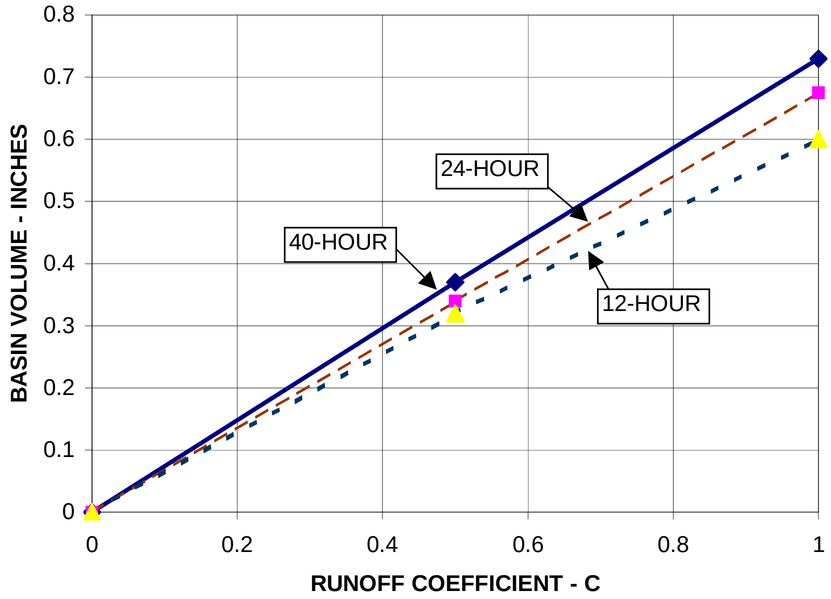

Figure 3. Maximized Capture Volume, Denver Raingage 1944-84 Period

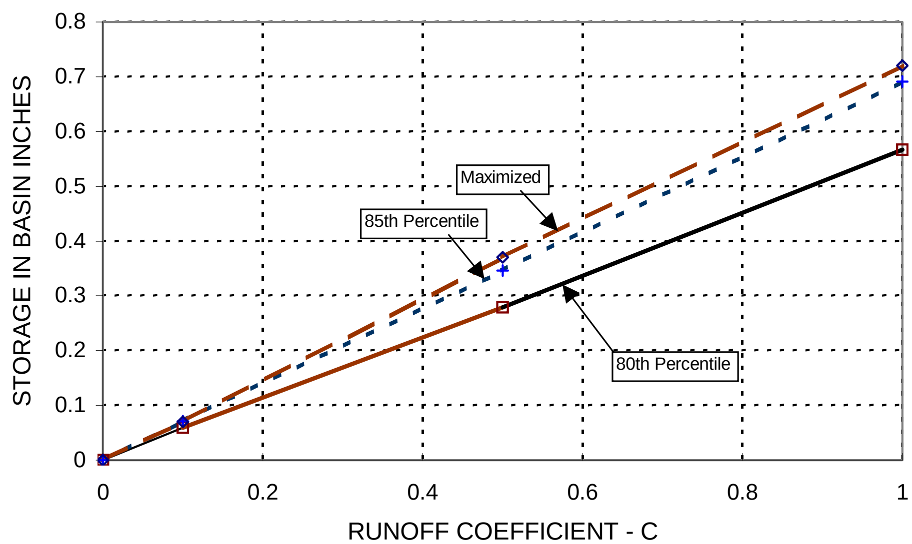

Figure 4. Capture Volume Using Denver Raingage for 40-Hour Emptying Time

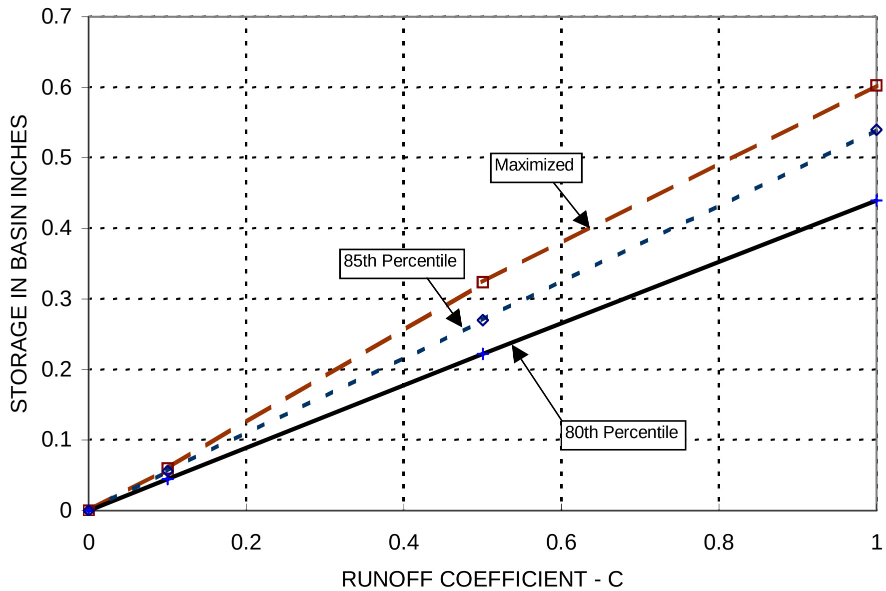

Figure 5. Capture Volume Using Denver Raingage for 12-Hour Emptying Time

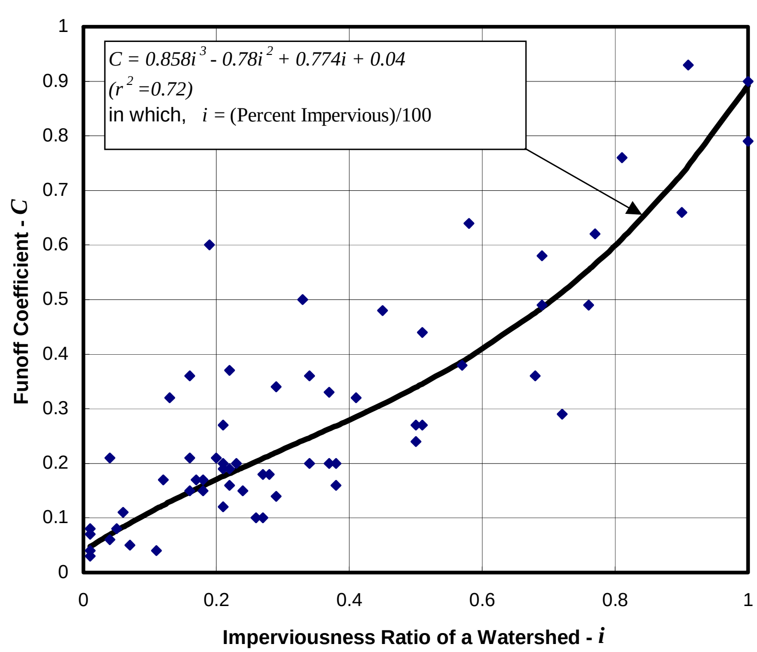

Figure 6. Runoff Coefficient Relationship Based on NURP Data. Applicable to 2-year and Smaller Design Storms.

Appendix B – Determination of the EURV for Full Spectrum Detention Design

URBAN DRAINAGE AND FLOOD CONTROL DISTRICT

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TECHNICAL MEMORANDUM

FROM: Ken A. MacKenzie, P.E., CFM, UDFCD Master Planning Program Manager
Derek N. Rapp, P.E., CFM, Peak Stormwater Engineering

SUBJECT: Determination of the Excess Urban Runoff Volume (EURV) for Full Spectrum Detention Design

DATE: Revised December 22, 2016 (March 23, 2015)

The purpose of this memorandum is to document the process used to develop new equations to estimate runoff volumes and the Excess Urban Runoff Volume (EURV) as the basis for full spectrum detention design. Simply put, the EURV is the difference in runoff volume between the developed condition and the undeveloped (i.e., natural) condition. The concept of full spectrum detention is described in the Storage chapter of the *Urban Storm Drainage Criteria Manual* and in other technical papers available for download at www.udfcd.org.

All of the equations developed in this memorandum were based on Colorado Urban Hydrograph Procedure (CUHP 2005, v2.0.0) modeling and one-hour rainfall depths from NOAA Atlas 14 at the Capitol Building in Denver.

The runoff volume equations are only valid for one-hour rainfall depths between 0.83 and 3.14 inches as shown in Table 1. These one-hour rainfall depths were temporally distributed over a two-hour period to create design storms consistent with CUHP protocol for the 2-, 5-, 10-, 25-, 50-, 100, and 500-year return periods.

Table 1: Average one-hour rainfall depth in the Denver region, as a function of probability of occurrence.

Recurrence Interval (Years)	Probability of Occurrence	Rainfall Depth (Inch)
2	0.50	0.83
5	0.20	1.09
10	0.10	1.33
25	0.04	1.69
50	0.02	1.99
100	0.01	2.31
500	0.002	3.14

CUHP was used to evaluate 2,020 subcatchments from recent UDFCD master planning studies. Subcatchments having a width/length ratio, slope, or centroid length outside one standard deviation of the mean of the data set were discarded in order to limit data scatter, leaving 1,203 subcatchments for further evaluation. The CUHP model was run for all 1,203 subcatchments and return periods with the hydrologic parameters listed in Table 2. Watershed characteristics (e.g., size, shape, slope, location of centroid, and imperviousness) were taken directly from the master planning studies. Various combinations of Soil Type (A, B, and C/D) were evaluated for each subcatchment.

Table 2: hydrologic parameters used in the CUHP modeling.

Soil Group	Historic Impervious Percentage (%)	Pervious Depression Storage (inch)	Impervious Depression Storage (inch)	Initial Infiltration Rate (in/hr)	Horton's Decay Coefficient (second^{-1})	Final Infiltration Rate (in/hr)
HSG A	2	0.35	0.1	5.0	0.0070	1.0
HSG B	2	0.35	0.1	4.5	0.0018	0.6
HSG C	2	0.35	0.1	3.0	0.0018	0.5

By performing a multiple regression analysis on the results for the 1,203 CUHP subcatchments, equations were developed for the 2-, 5-, 10-, 25-, 50-, 100- and 500-yr return periods for each hydrologic soil group and combined to provide the following watershed runoff equations:

$$V_{Runoff\_2yr} = P_1A[(0.082I^{1.311})A\% + (0.082I^{1.179})B\% + (0.082I^{1.132})CD\%] \quad (1)$$

$$V_{Runoff\_5yr} = P_1A[(0.084I^{1.285})A\% + (0.084I^{1.098})B\% + (0.082I + 0.003)CD\%] \quad (2)$$

$$V_{Runoff\_10yr} = P_1A[(0.086I^{1.241})A\% + (0.081I + 0.005)B\% + (0.073I + 0.012)CD\%] \quad (3)$$

$$V_{\_Runoff\_25yr} = P_1A[(0.087I^{1.133})A\% + (0.063I + 0.024)B\% + (0.056I + 0.030)CD\%] \quad (4)$$

$$V_{Runoff\_50yr} = P_1A[(0.084I + 0.002)A\% + (0.054I + 0.032)B\% + (0.048I + 0.038)CD\%] \quad (5)$$

$$V_{Runoff\_100yr} = P_1A[(0.077I + 0.010)A\% + (0.046I + 0.041)B\% + (0.040I + 0.047)CD\%] \quad (6)$$

$$V_{Runoff\_500yr} = P_1A[(0.064I + 0.024)A\% + (0.036I + 0.052)B\% + (0.031I + 0.057)CD\%] \quad (7)$$

Where $V_{\#yr}$ is the runoff volume for the given return period (acre-feet), P_1 is the one-hour rainfall depth (inches), A is the contributing watershed area (acres), I is the percentage imperviousness (expressed as a decimal), and *A%*, *B%*, and *CD%* are the percent of each hydraulic soil group (also expressed as a decimal). The CUHP Excel™ workbooks and multiple regression analysis files were saved in an archival folder named "*CUHP_Runoff_Volume_Equations.zip*" in the master planning reference library.

Runoff volume equations 1 through 7 were then used to calculate the EURV as the difference between developed condition runoff volume and historic runoff volume. The developed condition runoff volume was calculated for varying levels of imperviousness from 10 percent up to 100 percent. The historic condition runoff volume was estimated by setting the watershed imperviousness to 2%. The calculated EURV results in watershed inches versus imperviousness for each NRCS hydrologic soil group are shown in Figures 1 through 3.

A power curve was fit to the data set for each return period to develop an equation for the EURV. From Figures 1 through 3 it can be seen that the EURV is not always directly proportional in magnitude to the return period, that is to say that the 100-year EURV is not necessarily greater than the 50-year, the 25-year, or the 10-year EURV. The 10-year EURV was chosen as the representative EURV not because it is the largest, but because it is the most consistent among the three hydrologic soil groups.

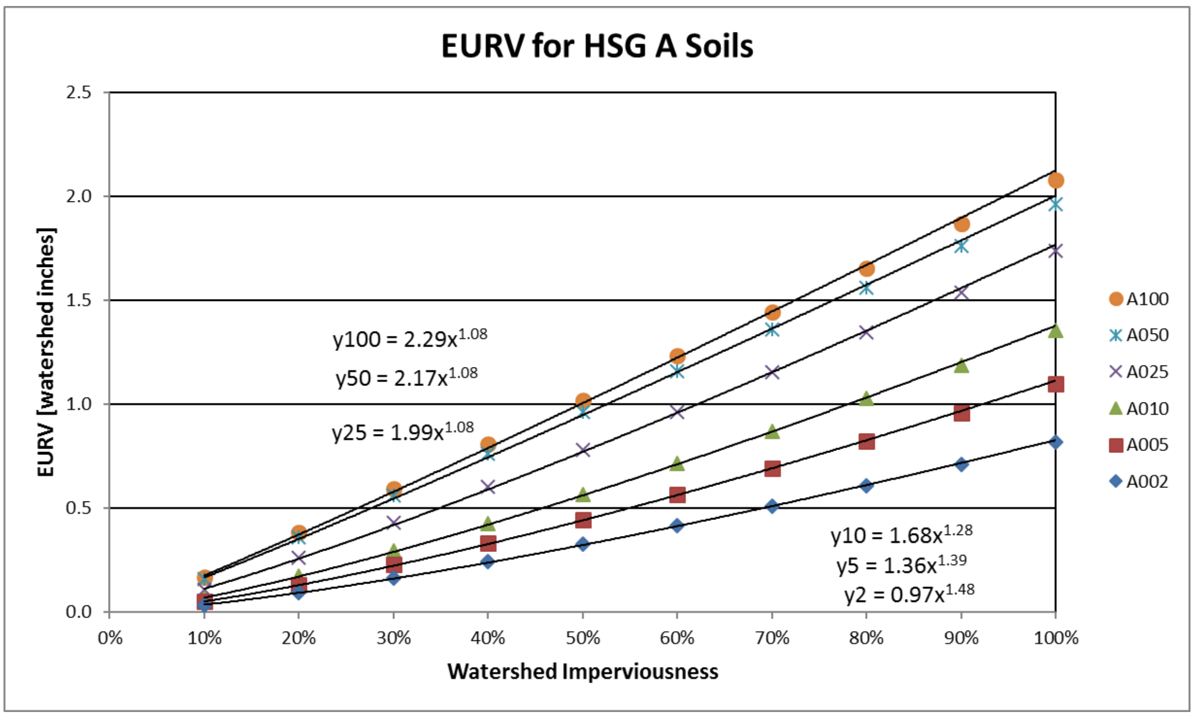

Figure 1: Hydrologic Soil Group A. Plot of EURV represented as the difference between developed condition and historic condition, in units of runoff depth per impervious area.

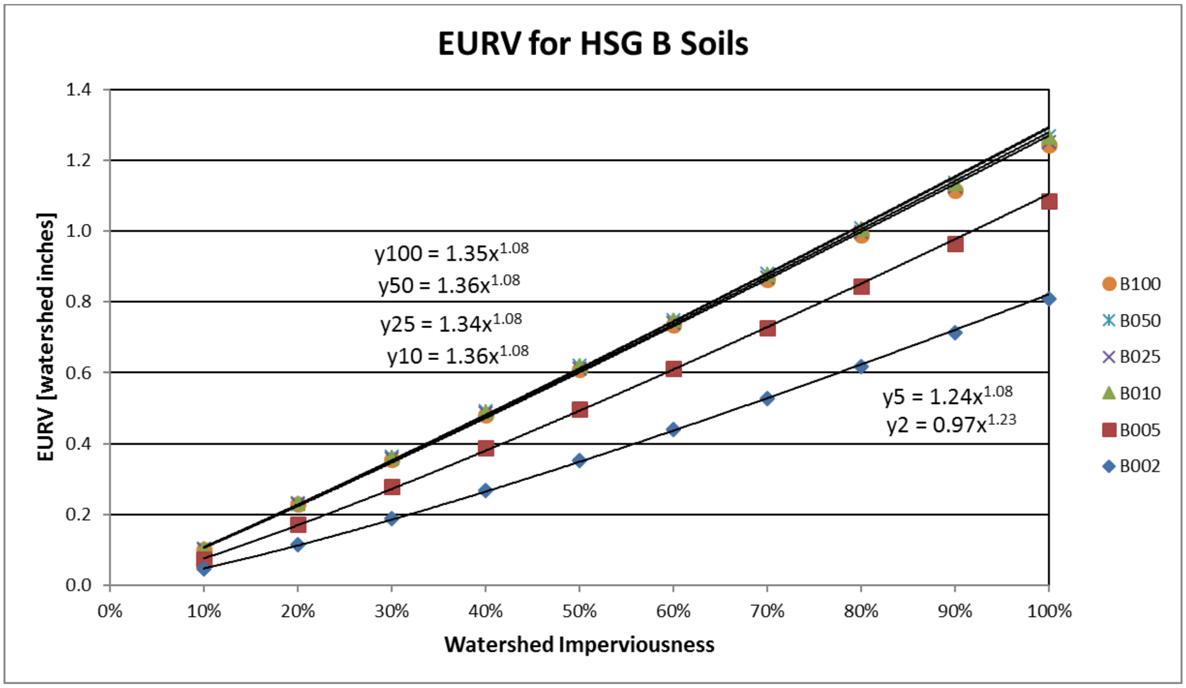

Figure 2: Hydrologic Soil Group B. Plot of EURV represented as the difference between developed condition and historic condition, in units of runoff depth per impervious area.

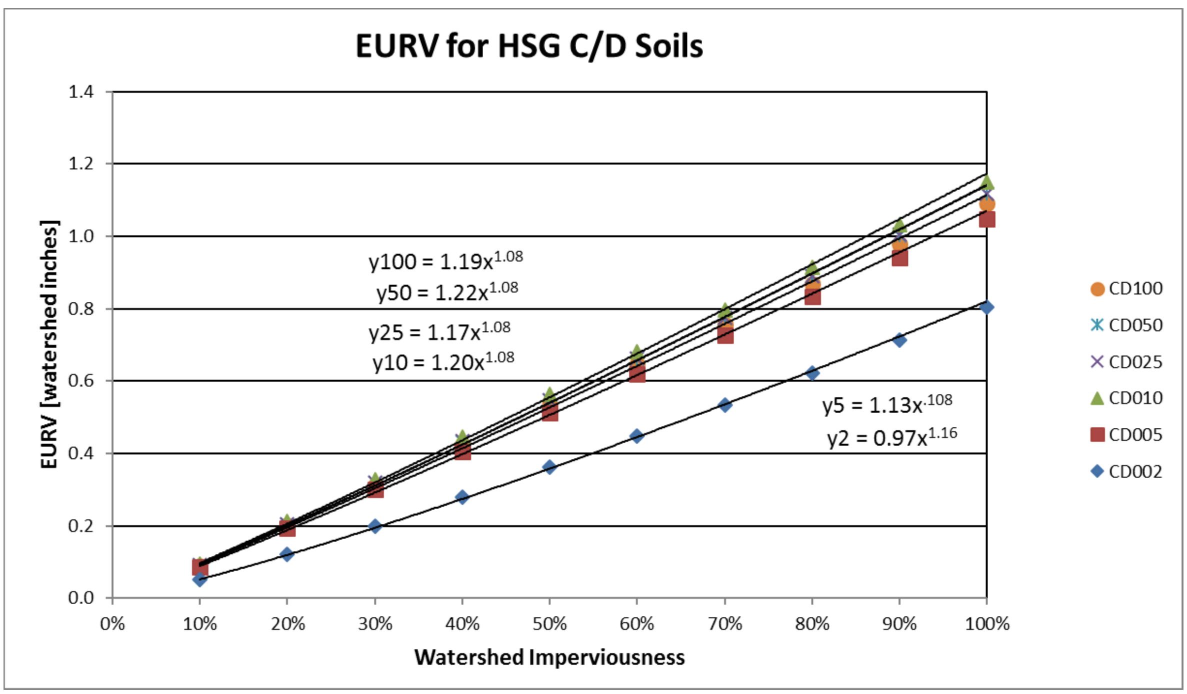

Figure 3: Hydrologic Soil Groups C and D. Plot of EURV represented as the difference between developed condition and historic condition, in units of runoff depth per impervious area.

To calculate the EURV in terms of runoff watershed inches, the resulting three EURV equations for the three representative hydrologic soil groups are:

$$EURV_{HSG\ A} = 1.68(IMP)^{1.28} \quad (8)$$

$$EURV_{HSG\ B} = 1.36(IMP)^{1.08} \quad (9)$$

$$EURV_{HSG\ C/D} = 1.20(IMP)^{1.08} \quad (10)$$

In which *EURV* is the excess urban runoff volume, ***in watershed inches***, and *IMP* is the developed condition imperviousness of the watershed, expressed as a ratio less than 1.

To calculate the EURV in units of acre feet, equations 8 through 10 are written as:

$$EURV_{HSG\ A} = Area * 0.140(IMP)^{1.28} \quad (11)$$

$$EURV_{HSG\ B} = Area * 0.113(IMP)^{1.08} \quad (12)$$

$$EURV_{HSG\ C/D} = Area * 0.100(IMP)^{1.08} \quad (13)$$

In which *EURV* is the excess urban runoff volume, ***in acre feet***, and *IMP* is the developed condition imperviousness of the watershed, expressed as a ratio less than 1. Equations 11 through 13 can then be combined to form equation 14.

$$EURV = Area * [0.140(IMP)^{1.28} * A\% + 0.113(IMP)^{1.08} * B\% + 0.100(IMP)^{1.08}] \quad (14)$$

Example Problem: A 50-acre Denver area watershed is situated on 44% HSG B and 56% HSG C. That portion of the watershed on HSG B soils is 35% impervious, the rest is 45% impervious.

Determine the excess urban runoff volume for this watershed.

Analysis:

$EURV_{HSG\ B} = 50 * 0.44 * 0.113(0.35)^{1.08}$ = 0.80 acre feet;

$EURV_{HSG\ C/D} = 50 * 0.56 * 0.10(0.45)^{1.08}$ = 1.18 acre feet

The excess urban runoff volume is 0.80 + 1.18 acre feet = **1.98 acre feet**.

This could also be calculated as:

$EURV_{HSG\ B} = 1.36(0.35)^{1.08}$ = 0.4377 inches per HSG B watershed area.

$EURV_{HSG\ C/D} = 1.20(0.45)^{1.08}$= 0.5066 inches per HSG C/D watershed area.

$$\left[(EURV_{HSG\ B})(HSG\ B\%) + \left(EURV_{HSG\ C/D}\right)(HSG\ C/D\%)\right]\left(\frac{AREA}{12}\right)$$

(0.4377*0.44 + 0.5066*0.56)(50/12) = **1.98 acre feet**.

Appendix C - Estimation of Runoff and Storage Volumes for Use with Full Spectrum Detention

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TECHNICAL MEMORANDUM

FROM: Ken A. MacKenzie, P.E., CFM, UDFCD Master Planning Program Manager
Derek N. Rapp, P.E., CFM, Peak Stormwater Engineering

SUBJECT: Estimation of Runoff and Storage Volumes for Use with Full Spectrum Detention

DATE: Revised January 5, 2017 (March 26, 2015)

The purpose of this memorandum is to document the process used to develop new equations to estimate the runoff volumes and required storage volumes for use with full spectrum detention design. The concept of full spectrum detention is described in the Storage chapter of the *Urban Storm Drainage Criteria Manual* (USDCM) and other technical papers available for download at www.udfcd.org. The USDCM allows the use of simplified equations for determining full spectrum detention design volumes for watersheds less than 10 acres.

The runoff volume equations developed in this memorandum were based on Colorado Urban Hydrograph Procedure (CUHP 2005, v2.0.0) modeling and one-hour rainfall depths from NOAA Atlas 14 at the Capitol Building in Denver.

The runoff volume equations are only valid for one-hour rainfall depths between 0.83 and 3.14 inches as shown in Table 1. These one-hour rainfall depths were temporally distributed over a two-hour period to create design storms consistent with CUHP protocol for the 2-, 5-, 10-, 25-, 50-, 100, and 500-year return periods.

Table 1: Average one-hour rainfall depth in the Denver region, as a function of probability of occurrence.

Recurrence Interval (Years)	Probability of Occurrence	Rainfall Depth (Inch)
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CUHP was used to evaluate 2,020 subcatchments from recent UDFCD master planning studies. Subcatchments having a width/length ratio, slope, or centroid length outside one standard deviation of the mean of the data set were discarded in order to limit data scatter, leaving 1,203 subcatchments for further evaluation. The CUHP model was run for all 1,203 subcatchments and return periods with the hydrologic parameters listed in Table 2. Watershed characteristics (e.g., size, shape, slope, location of centroid, and imperviousness) were taken directly from the master planning studies. Various combinations of Soil Type (A, B, and C/D) were evaluated for each subcatchment.

Table 2: hydrologic parameters used in the CUHP modeling.

Soil Group	**Historic Impervious Percentage (%)**	**Pervious Depression Storage (inch)**	**Impervious Depression Storage (inch)**	**Initial Infiltration Rate (in/hr)**	**Horton's Decay Coefficient (second^{-1})**	**Final Infiltration Rate (in/hr)**
HSG A	2	0.35	0.1	5.0	0.0070	1.0
HSG B	2	0.35	0.1	4.5	0.0018	0.6
HSG C	2	0.35	0.1	3.0	0.0018	0.5

By performing a multiple regression analysis on those remaining CUHP subcatchments, equations were developed for the 2-, 5-, 10-, 25-, 50-, 100- and 500-yr return periods for each hydrologic soil group and combined to provide the following watershed runoff equations:

$$V_{Runoff\_2yr} = P_1 A[(0.082I^{1.311})A\% + (0.082I^{1.179})B\% + (0.082I^{1.132})CD\%] \quad (1)$$

$$V_{Runoff\_5yr} = P_1 A[(0.084I^{1.285})A\% + (0.084I^{1.098})B\% + (0.082I + 0.003)CD\%] \quad (2)$$

$$V_{Runoff\_10yr} = P_1 A[(0.086I^{1.241})A\% + (0.081I + 0.005)B\% + (0.073I + 0.012)CD\%] \quad (3)$$

$$V_{\_Runoff\_25yr} = P_1 A[(0.087I^{1.133})A\% + (0.063I + 0.024)B\% + (0.056I + 0.030)CD\%] \quad (4)$$

$$V_{Runoff\_50yr} = P_1 A[(0.084I + 0.002)A\% + (0.054I + 0.032)B\% + (0.048I + 0.038)CD\%] \quad (5)$$

$$V_{Runoff\_100yr} = P_1 A[(0.077I + 0.010)A\% + (0.046I + 0.041)B\% + (0.040I + 0.047)CD\%] \quad (6)$$

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Where $V_{Runoff\_\#yr}$ is the runoff volume for the given return period (acre-feet), P_1 is the one-hour rainfall depth (inches), A is the contributing watershed area (acres), I is the percentage imperviousness (expressed as a decimal), and $A\%$, $B\%$, and $CD\%$ are the percent of each hydraulic soil group (also expressed as a decimal). It should be noted that these equations are a mix of linear and power functions. The CUHP Excel™ workbooks and multiple regression analysis files were saved in an archival folder named "*CUHP_Runoff_Volume_Equations_Dec2016.zip*" in the master planning reference library.

In order to develop estimated storage volume equations, the UD-Detention workbook was used to model full spectrum detention basins. UD-Detention was updated to v3.07 to include the runoff volume equations described above. UD-Detention v3.07 was run for watershed areas of 5-, 10-, 20-, 40-, 60-, and 100-acres at 33%, 67%, and 100% imperviousness. Design storms included the 2-, 5-, 10-, 25-, 50-, and 100-year return period. Hydrologic soil groups A, B, and C/D were evaluated separately. WQCV drain times of 40 hours, 24 hours, and 12 hours were also evaluated resulting in a total of 972 model runs). The resulting maximum required storage volumes were divided by the corresponding runoff hydrograph volume and those ratios are shown in Tables 3 through 5 for the 40 hour WQCV drain time scenarios.

Table 3: UD-Detention Storage Volume Reduction Factors for HSG A Soils (40 hour drain time).

UD-Detention Model Results for HSG A Soils (40-hr drain time): V_{STORED} / V_{INFLOW}							
Watershed Area =	**5.00**	**10.00**	**20.00**	**40.00**	**60.00**	**100.00**	**AVG**
Watershed Imperviousness =				**33%**			
2-Year Ratio (A soils) =	92.5%	93.0%	92.8%	93.0%	92.6%	92.9%	**93%**
5-Year Ratio (A soils) =	93.4%	93.1%	93.1%	93.3%	93.3%	93.4%	**93%**
10-Year Ratio (A soils) =	94.0%	93.9%	94.1%	93.8%	94.0%	93.8%	**94%**
25-Year Ratio (A soils) =	89.1%	90.0%	89.8%	90.4%	90.5%	90.2%	**90%**
50-Year Ratio (A soils) =	72.7%	75.0%	75.3%	76.6%	76.9%	76.7%	**76%**
100-Year Ratio (A soils) =	66.9%	67.6%	67.8%	68.3%	68.4%	68.1%	**68%**
Watershed Imperviousness =				**67%**			
2-Year Ratio (A soils) =	94.0%	94.2%	94.5%	94.4%	94.4%	94.4%	**94%**
5-Year Ratio (A soils) =	94.6%	94.5%	94.9%	94.8%	94.9%	94.6%	**95%**
10-Year Ratio (A soils) =	94.9%	95.1%	95.2%	95.4%	95.2%	95.1%	**95%**
25-Year Ratio (A soils) =	94.5%	95.0%	95.0%	95.1%	95.1%	94.9%	**95%**
50-Year Ratio (A soils) =	85.9%	87.2%	87.7%	88.5%	88.7%	88.6%	**88%**
100-Year Ratio (A soils) =	80.3%	81.2%	81.4%	82.1%	82.2%	82.2%	**82%**
Watershed Imperviousness =				**100%**			
2-Year Ratio (A soils) =	94.0%	93.8%	93.9%	94.1%	93.9%	94.2%	**94%**
5-Year Ratio (A soils) =	94.2%	94.4%	94.6%	94.7%	94.7%	94.6%	**95%**
10-Year Ratio (A soils) =	94.7%	94.7%	94.7%	94.8%	95.0%	95.0%	**95%**
25-Year Ratio (A soils) =	95.1%	95.3%	95.2%	95.5%	95.4%	95.3%	**95%**
50-Year Ratio (A soils) =	91.8%	91.6%	92.3%	92.3%	92.2%	92.1%	**92%**
100-Year Ratio (A soils) =	86.1%	86.2%	86.7%	87.1%	87.0%	86.9%	**87%**

Table 4: UD-Detention Storage Volume Reduction Factors for HSG B Soils (40 hour drain time).

UD-Detention Model Results for HSG B Soils (40-hr drain time): V_{STORED} / V_{INFLOW}							
Watershed Area =	**5.00**	**10.00**	**20.00**	**40.00**	**60.00**	**100.00**	**AVG**
Watershed Imperviousness =	**33%**						
2-Year Ratio (B soils) =	93.5%	93.2%	93.3%	93.2%	93.5%	93.4%	**93%**
5-Year Ratio (B soils) =	94.3%	94.1%	94.0%	94.0%	94.1%	93.9%	**94%**
10-Year Ratio (B soils) =	89.6%	89.1%	89.7%	89.4%	89.4%	88.8%	**89%**
25-Year Ratio (B soils) =	62.9%	62.7%	64.5%	65.0%	64.8%	63.5%	**64%**
50-Year Ratio (B soils) =	52.3%	51.9%	53.9%	54.9%	54.8%	53.7%	**54%**
100-Year Ratio (B soils) =	49.5%	48.8%	49.5%	49.5%	49.5%	49.0%	**49%**
Watershed Imperviousness =	**67%**						
2-Year Ratio (B soils) =	94.3%	94.1%	94.4%	94.6%	94.5%	94.5%	**94%**
5-Year Ratio (B soils) =	94.9%	94.9%	94.9%	95.0%	95.0%	94.7%	**95%**
10-Year Ratio (B soils) =	95.1%	95.2%	95.2%	95.2%	95.2%	95.0%	**95%**
25-Year Ratio (B soils) =	81.5%	81.8%	82.7%	82.5%	82.0%	82.0%	**82%**
50-Year Ratio (B soils) =	73.2%	73.3%	74.8%	74.7%	74.2%	74.2%	**74%**
100-Year Ratio (B soils) =	65.9%	65.7%	66.9%	66.8%	66.4%	66.5%	**66%**
Watershed Imperviousness =	**100%**						
2-Year Ratio (B soils) =	93.7%	93.7%	93.5%	93.9%	93.7%	93.6%	**94%**
5-Year Ratio (B soils) =	94.3%	94.0%	94.1%	94.2%	94.2%	94.1%	**94%**
10-Year Ratio (B soils) =	94.5%	94.7%	94.7%	94.7%	94.8%	94.7%	**95%**
25-Year Ratio (B soils) =	87.6%	87.6%	87.3%	87.2%	87.2%	86.8%	**87%**
50-Year Ratio (B soils) =	81.3%	81.3%	81.0%	80.8%	80.8%	80.3%	**81%**
100-Year Ratio (B soils) =	74.1%	74.0%	74.0%	73.9%	73.9%	73.3%	**74%**

Table 5: UD-Detention Storage Volume Reduction Factors for HSG C&D Soils (40 hour drain time).

UD-Detention Model Results for HSG C&D Soils (40-hr draintime): V_{STORED} / V_{INFLOW}							
Watershed Area =	**5.00**	**10.00**	**20.00**	**40.00**	**60.00**	**100.00**	**AVG**
Watershed Imperviousness =	**33%**						
2-Year Ratio (C/D soils) =	93.4%	93.2%	92.9%	93.1%	93.5%	93.4%	**93%**
5-Year Ratio (C/D soils) =	94.3%	94.3%	94.0%	94.1%	94.1%	94.0%	**94%**
10-Year Ratio (C/D soils) =	77.9%	77.8%	77.8%	78.3%	78.1%	76.9%	**78%**
25-Year Ratio (C/D soils) =	54.7%	54.4%	55.2%	56.3%	56.4%	55.1%	**55%**
50-Year Ratio (C/D soils) =	46.2%	46.0%	46.5%	47.6%	47.8%	46.6%	**47%**
100-Year Ratio (C/D soils) =	46.0%	45.9%	45.9%	46.1%	46.0%	45.5%	**46%**
Watershed Imperviousness =	**67%**						
2-Year Ratio (C/D soils) =	94.2%	94.3%	94.2%	94.2%	94.2%	94.0%	**94%**
5-Year Ratio (C/D soils) =	95.0%	94.9%	95.0%	94.9%	95.0%	94.9%	**95%**
10-Year Ratio (C/D soils) =	90.0%	89.6%	89.7%	89.8%	89.9%	89.3%	**90%**
25-Year Ratio (C/D soils) =	75.0%	75.3%	75.5%	76.2%	76.4%	75.5%	**76%**
50-Year Ratio (C/D soils) =	66.6%	66.2%	67.2%	68.2%	68.5%	67.5%	**67%**
100-Year Ratio (C/D soils) =	60.7%	60.7%	60.8%	61.6%	61.6%	61.2%	**61%**
Watershed Imperviousness =	**100%**						
2-Year Ratio (C/D soils) =	93.1%	93.1%	93.5%	93.4%	93.3%	93.3%	**93%**
5-Year Ratio (C/D soils) =	93.9%	94.0%	94.1%	94.0%	94.2%	94.1%	**94%**
10-Year Ratio (C/D soils) =	93.5%	93.6%	93.1%	93.0%	93.1%	93.0%	**93%**
25-Year Ratio (C/D soils) =	84.0%	84.5%	83.9%	83.5%	83.4%	83.1%	**84%**
50-Year Ratio (C/D soils) =	77.6%	77.6%	77.1%	76.9%	76.8%	76.3%	**77%**
100-Year Ratio (C/D soils) =	71.2%	70.9%	70.5%	70.4%	70.3%	69.8%	**70%**

For each return period, the average storage/runoff ratio for all six areas was calculated as shown in the last column of Tables 3 through 5. The average storage/runoff ratio was plotted vs. imperviousness for each of the three hydrologic soil groups and a power regression was applied as shown in Figure 1 for the 100-year return period. Similar power regression plots were developed for the other five return periods also. The resulting storage/runoff ratio equations were then multiplied by the runoff volume equations (converted to watershed inches instead of acre-feet as expressed in Equations 1 through 6) to develop new storage volume equations. The resulting storage volume equations (in watershed inches) are shown in Equations 8 through 13. The same process was repeated for WQCV drain times of 24 hours and 12 hours. The results were almost identical since the WQCV is such a small percentage of the total detention volume. Therefore, the equations developed for the 40-hour WQCV drain time are considered suitable for all WQCV drain times.

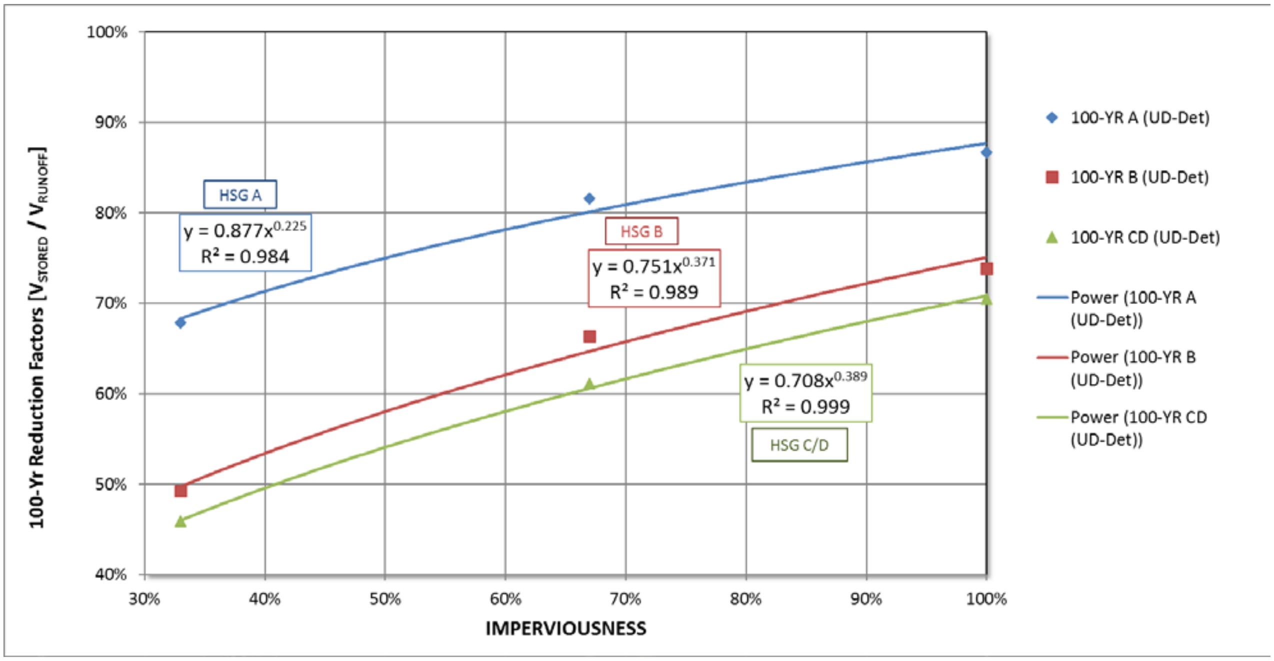

Figure 1: 100-yr Power regression equations for ratio of stored volume to runoff volume as a function of hydrologic soil group and imperviousness.

$$V_{Storage\_2yr}(in) = P_1[(0.932I^{1.324})A\% + (0.924I^{1.184})B\% + (0.920I^{1.134})CD\%] \qquad (8)$$

$$V_{Storage\_5yr}(in) = P_1[(0.960I^{1.298})A\% + (0.953I^{1.100})B\% + (0.926I^{1.001} + 0.030I^{0.001})CD\%] \qquad (9)$$

$$V_{Storage\_10yr}(in) = P_1[(0.977I^{1.251})A\% + (0.928I^{1.056} + 0.055I^{0.056})B\% + (0.831I^{1.167} + 0.138I^{0.167})CD\%] \qquad (10)$$

$$V_{Storage\_25yr}(in) = P_1[(0.998I^{1.188})A\% + (0.675I^{1.290} + 0.253I^{0.290})B\% + (0.576I^{1.382} + 0.311I^{0.382})CD\%] \qquad (11)$$

$$V_{Storage\_50yr}(in) = P_1[(0.935I^{1.182} + 0.024I^{0.182})A\% + (0.539I^{1.381} + 0.317I^{0.381})B\% + (0.450I^{1.457} + 0.360I^{0.457})CD\%] \qquad (12)$$

$$V_{Storage\_100yr}(in) = P_1[(0.806I^{1.225} + 0.109I^{0.225})A\% + (0.412I^{1.371} + 0.371I^{0.371})B\% + (0.341I^{1.389} + 0.398I^{0.389})CD\%] \qquad (13)$$

Where $Vstorage_{\#yr}$ is the calculated storage volume (watershed inches), P_1 is the one-hour rainfall depth corresponding to the return period (inches), I is the percentage imperviousness (expressed as a decimal), and *A%*, B%, and C*D%* are the percent of each hydraulic soil group (expressed as a decimal). A comparison of the 100-yr runoff and storage volumes are shown in Figure 2.

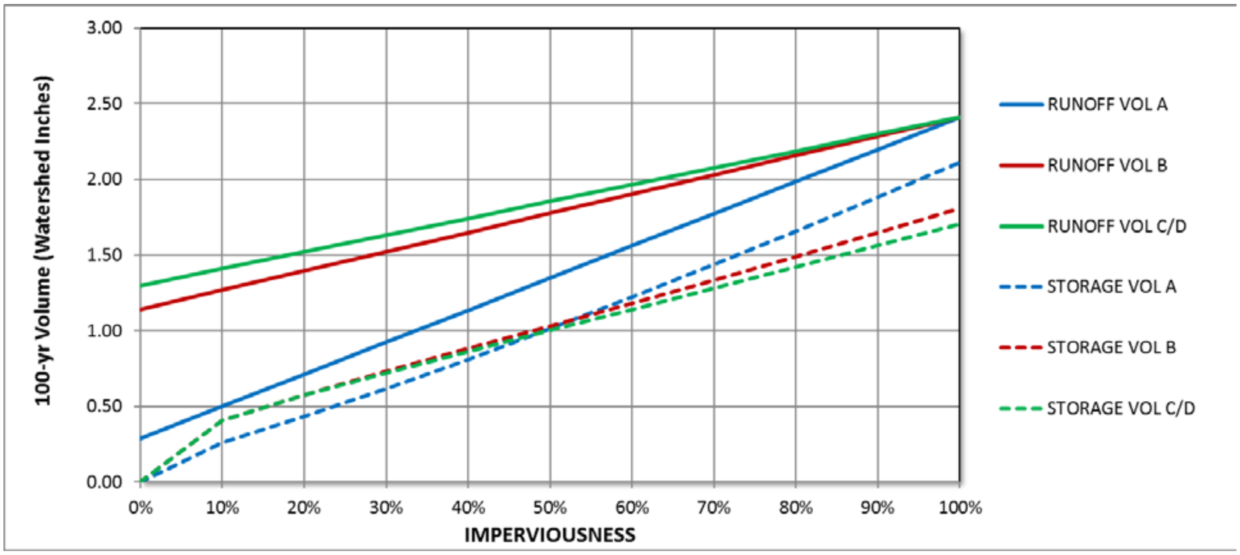

Figure 2: Plot of 100-yr runoff volumes and storage volumes.

Equations 8 through 13 can be expressed in acre-feet as shown in Equations 14 through 19.

$$V_{Storage\_2yr}(ac-ft) = P_1A[(0.078I^{1.324})A\% + (0.077I^{1.184})B\% + (0.077I^{1.134})CD\%] \quad (14)$$

$$V_{Storage\_5yr}(ac-ft) = P_1A[(0.080I^{1.298})A\% + (0.079I^{1.100})B\% + (0.077I^{1.001} + 0.003I^{0.001})CD\%] \quad (15)$$

$$V_{Storage\_10yr}(acft) = P_1A[(0.081I^{1.251})A\% + (0.077I^{1.056} + 0.005I^{0.056})B\% + (0.069I^{1.167} + 0.011I^{0.167})CD\%] \quad (16)$$

$$V_{Storage\_25yr}(ac-ft) = P_1A[(0.083I^{1.188})A\% + (0.056I^{1.290} + 0.021I^{0.290})B\% + (0.048I^{1.382} + 0.026I^{0.382})CD\%] \quad (17)$$

$$V_{Storage\_50yr}(ac-ft) = P_1A[(0.078I^{1.182} + 0.002I^{0.182})A\% + (0.045I^{1.381} + 0.026I^{0.381})B\% + (0.037I^{1.457} + 0.030I^{0.457})CD\%] \quad (18)$$

$$V_{Storage\_100yr}(ac-ft) = P_1A[(0.067I^{1.225} + 0.009I^{0.225})A\% + (0.034I^{1.371} + 0.031I^{0.371})B\% + (0.028I^{1.389} + 0.033I^{0.389})CD\%] \quad (19)$$

Where $V_{STORAGE\_\#yr}$ is the storage volume (acre-feet), P_1 is the one-hour rainfall depth corresponding to the return period (in), A is the watershed area in acres, I is the percentage imperviousness (expressed as a decimal), and *A%*, and *B&CD%* are the percent of each hydraulic soil group (expressed as a decimal).

Example Problem 1: An 18-acre Denver watershed is found to have the following characteristics: 50% imperviousness, 15% HSG A, 25% HSG B, and 60% HSG C&D. The 100-year one-hour rainfall depth is 2.31 inches.

Determine A) the estimated runoff hydrograph volume for the 100-year return period, and B) the estimated storage volume required for a full spectrum detention basin to accommodate the 100-year flood.

A) Analysis:

$$V_{Runoff_{100yr}} = P_1 A[(0.077I + 0.010)A\% + (0.046I + 0.041)B\% + (0.040I + 0.047)CD\%]$$

$$V_{Runoff\_100yr} = 2.31(18)[(0.077(0.5) + 0.010)(0.15) + (0.046(0.5) + 0.041)(0.25) + (0.040(0.5) + 0.047)(0.60)]$$

$$\boldsymbol{V_{Runoff\_100yr} = 2.64\ acre - feet.}$$

B) Analysis:

$$V_{Storage\_100yr} = P_1 A[(0.067I^{1.225} + 0.009I^{0.225})A\% + (0.034I^{1.371} + 0.031I^{0.371})B\% + (0.028I^{1.389} + 0.033I^{0.389})CD\%]$$

$$V_{Storage\_100yr} = 2.31(18)[(0.067(0.5)^{1.225} + 0.009(0.5)^{0.225})(0.15) + (0.034(0.5)^{1.371} + 0.031(0.5)^{0.371})(0.25) + (0.0285(0.5)^{1.389} + 0.033(0.5)^{0.389})(0.6)]$$

$$\boldsymbol{V_{Storage\_100yr} = 1.52\ acre - feet.}$$

Solution: The 100-year required storage volume of 1.52 acre-feet is **58%** of the runoff hydrograph of 2.64 acre-feet.

Appendix D - Modeling Detention Basin Geometry

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TECHNICAL MEMORANDUM

FROM: Ken A. MacKenzie, P.E., Master Planning Program Manager
Jason S Stawski, E.I., Construction Manager, DCM Program

SUBJECT: Modeling Detention Basins

DATE: Revised February 1, 2016 (Original January 20, 2014)

The purpose of this memorandum is to document a set of equations and method to model proposed detention basins with stage-storage relationships that produce realistic draining characteristics; for use in reservoir routing programs such as HEC-HMS and HEC-1; TR-20/TR-55; HEC-RAS unsteady flow; SWMM (including PC-SWMM and XP-SWMM); ICPR, PondPack, HydroCAD, and Hydraflow. This method is appropriate for modeling proposed flood and/or stormwater quality detention basins in watershed planning studies.

Area and Volume Calculations:

Initial Surcharge Volume:

$$ISV = 0.003WQCV; \qquad A_{ISV} = \frac{ISV}{ISD}; \qquad L_{ISV} = \sqrt{A_{ISV}}; \qquad W_{ISV} = \sqrt{A_{ISV}}$$

Where *ISV* is the initial surcharge volume (ft^3), A_{ISV} is *ISV* surface area (ft^2), *ISD* is the initial surcharge depth (ft, typically 0.33 to 0.50), and L_{ISV} and W_{ISV} are the length and width of the *ISV* (ft).

Basin Floor Volume:

$$L_{floor} = L_{ISV} + \frac{H_{floor}}{S_{TC}} + H_{floor}(S_{main}); \qquad W_{floor} = W_{ISV} + \frac{H_{floor}}{R_{L:W}(S_{TC})};$$

$$A_{floor} = L_{floor}(W_{floor}); \; V_{floor} = \frac{H_{floor}}{3}\left(A_{ISV} + A_{floor} + \sqrt{A_{ISV}(A_{floor})}\right)$$

Where L_{floor} and W_{floor} (ft) are the length and width of the basin floor section at the point where the top of the basin floor section meets the toe of the basin main section, H_{floor} is the depth of the basin floor section (ft), S_{TC} is the trickle channel slope (ft/ft), S_{main} is the side slope of the basin main section (H:V; e.g., 4 if the horizontal:vertical ratio is 4:1), $R_{L:W}$ is the basin length:width ratio (e.g., 2 if the basin length is twice the basin width), A_{floor} is top area of the basin floor section (ft^2), and V_{floor} is volume of the basin floor section (ft^3).

Main Basin Volume:

$$L_{main} = L_{floor} + 2H_{main}(S_{main}); \qquad W_{main} = W_{floor} + 2H_{main}(S_{main}); \qquad A_{main} = L_{main}(W_{main});$$

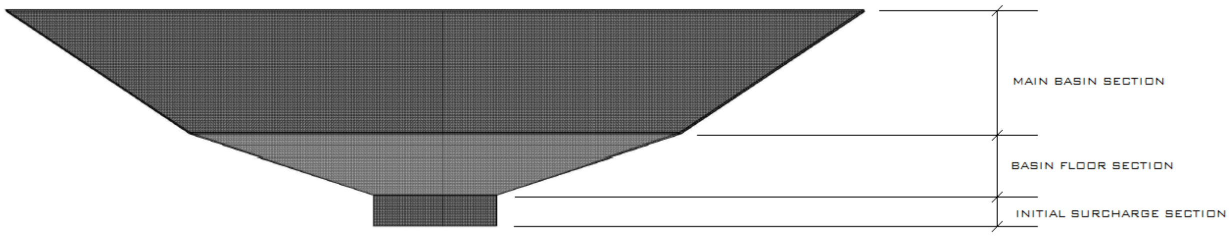
$$V_{main} = \frac{H_{main}}{3}\left(A_{main} + A_{floor} + \sqrt{A_{main}(A_{floor})}\right)$$

Where L_{main} and W_{main} (ft) are the length and width of the main basin section at the point at the top of the basin, H_{main} is the depth of the main basin section (ft), A_{main} is top area of the main basin section (ft^2), and V_{main} is volume of the main basin section (ft^3).

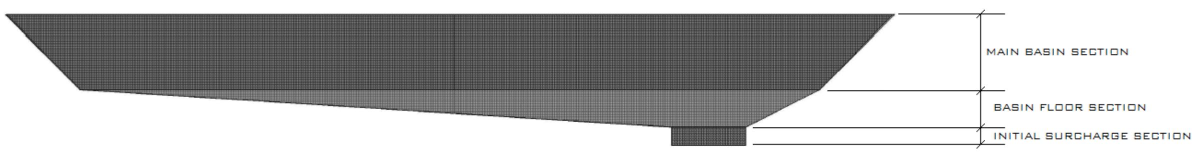
Total Basin Volume:

$$V_{total} = ISV + A_{ISV}(D_{TC}) + V_{floor} + V_{main}$$

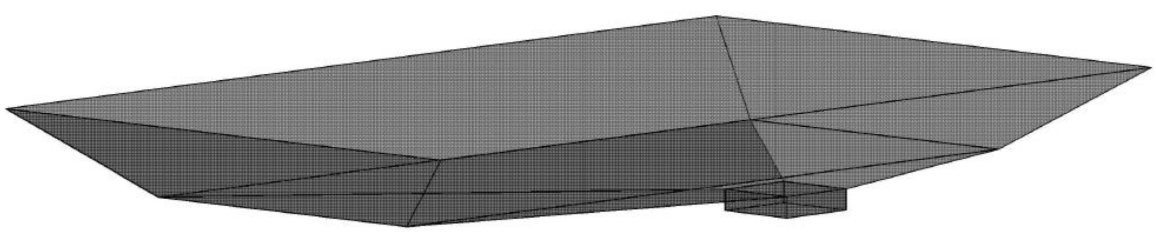
Where V_{total} is the total basin volume (ft^3) and D_{TC} is the depth of the trickle channel (ft).



Front view of detention basin model



Side view of detention basin model



Axonometric projection of detention basin model

Appendix E - Detention Basin Alternative Outlet Design Study

Applied Research and Innovation Branch

Detention Basin Alternative Outlet Design Study

Author:
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Urban Drainage and Flood Control District,
Denver, Colorado

Report No. CDOT-2016-04
Oct 2016

The contents of this report reflect the views of the author(s), who is(are) responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Colorado Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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16. Abstract

This study examines the outlets structures CDOT has historically employed to drain water quality treatment detention basins and flood control basins, presents two new methods of metering the water quality capture volume (WQCV), namely 1) the *Elliptical Slot Weir* alternative and 2) the *Maximized Orifice Area* alternative. The study also develops new sizing guidance for the overflow outlet, and new design software that should be very helpful to CDOT and its contractors. The report documents the findings of mathematical and physical hydraulic models, presents new equations and mathematical functions to help CDOT better design these facilities, presents new design methods and software tools, and provides an overview of a new Colorado statute that CDOT must comply with regarding all stormwater detention and infiltration facilities.

Implementation

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Detention Basins, Extended Detention Basins, Stormwater Quality Best Management Practices, BMPs, Stormwater Control Measures, Outlet Structure Design, MS4 Compliance	This document is available on CDOT's website http://www.coloradodot.info/programs/research/pdfs

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Detention Basin Alternative Outlet Design Study

Prepared by:

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Urban Drainage and Flood Control District,
Denver, Colorado

Submitted to:

Colorado Department of Transportation
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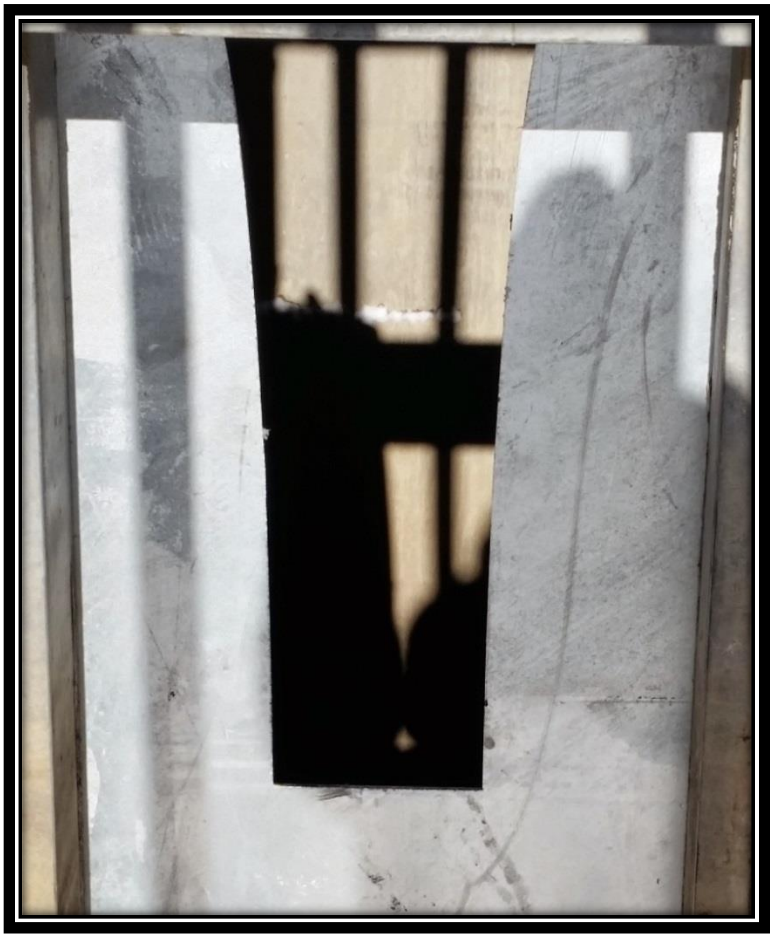

April 18, 2016 Draft

Detention Basin Alternative Outlet Design

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Detention Basin Alternative Outlet Design

RESEARCH STATEMENT

Extended detention and full-spectrum detention basins improve the quality of stormwater runoff through settling of sediment. This is achieved by detaining and slowly releasing the stormwater over a prescribed time duration of generally 40-72 hours. The metering of the impounded stormwater through the outlet structure is accomplished through one or more vertical columns of orifices in a steel plate that is affixed to the face of the structure, such that the orifices span the depth of the water quality impoundment. These orifices are protected from debris clogging with a well screen, as shown in Figure 1. While this practice has been proven to reduce TSS and related pollutants, maintenance of the orifices and the well screen is significant. An alternative outlet that is less susceptible to clogging and therefore requires less frequent maintenance would be of great benefit to the Colorado Department of Transportation (CDOT) and others.

Figure 1. The current standard for water quality outlet design includes a column of small orifices, protected from clogging by a well screen (shown removed for maintenance). The well screen was added after earlier installations demonstrated a great propensity for clogging. Unfortunately, the well screen also becomes clogged and is considered a significant maintenance issue for CDOT field personnel.

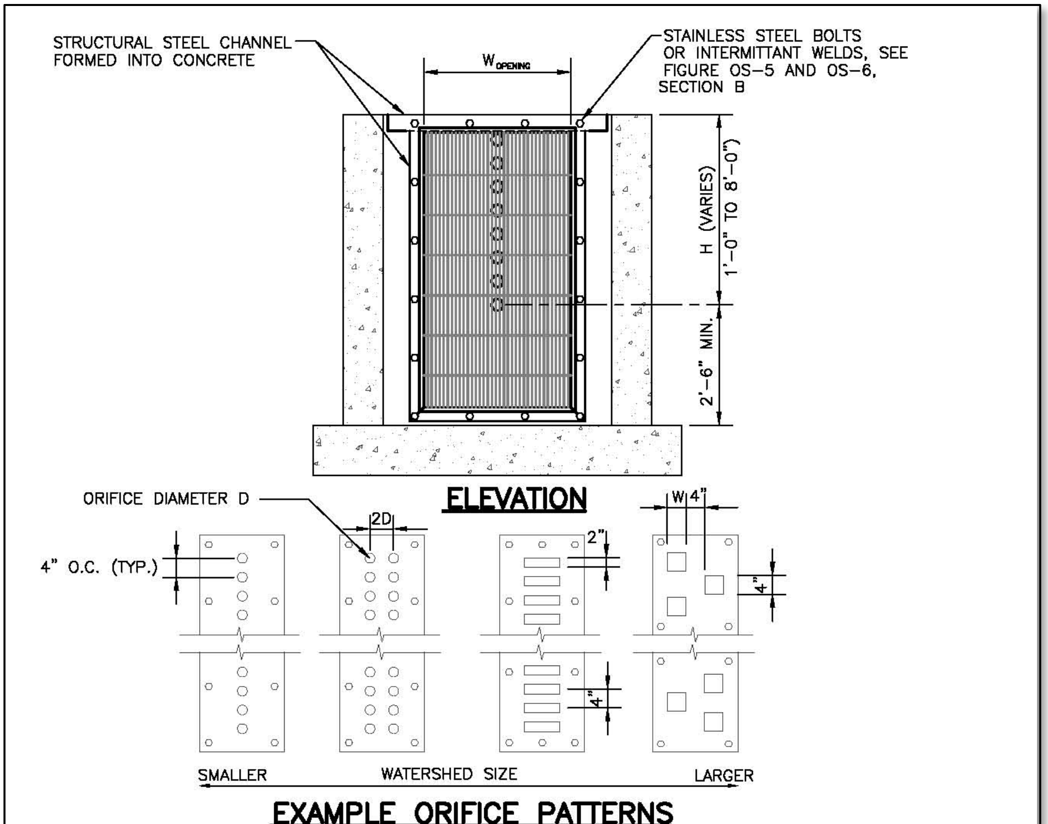
Key Words: Stormwater Detention Practices, Water Quality Capture Volume, Excess Urban Runoff Volume, Extended Detention Basin, Outlet Structure, Micropool, Stormwater Maintenance.

1. INTRODUCTION

All new construction and redevelopment sites in the CDOT MS4 (Municipal Separate Storm Sewer System) permit area are required to evaluate whether stormwater controls are required per CDOT's NDRD (New Development and Redevelopment) Program requirements to address higher runoff volumes and pollutant loads associated with an increase in impervious surfaces. These controls are here referred to as Permanent Water Quality Control Measures (also known as permanent Best Management Practices or BMPs). Water quality control measures must be periodically maintained to ensure functionality. Therefore, CDOT requires facility inspections to identify any maintenance needs such as sediment or weed removal.

The Urban Drainage and Flood Control District (UDFCD) promulgates regional stormwater quality criteria including design standards for extended detention basins to remove sediment by settling action. For many highway projects, extended detention basins represent the default water quality BMP and there are thousands of these basins in service across the Colorado. As recently as 2010, the UDFCD's *Urban Storm Drainage Criteria Manual* (USDCM) *Volume 3* recommended an outlet structure for detention basins that included a water quality plate having orifices spaced 4" vertically on center and being sized such that the water quality capture volume (WQCV) drain out in 40 hours or longer, as shown in Figure 2.

The problem with this guidance is that smaller water quality orifices clog more quickly, and the UDFCD guidance often resulted in very small orifices. CDOT has followed the UDFCD guidance in numerous detention basin outlet structure designs. In September 2012, UDFCD and CDOT partnered to jointly fund a study to examine alternatives to the columns of small orifices and accompanying well screens which represent the state of practice for water quality, and also to examine the hydraulic characteristics of detention basin overflow outlets and develop equations, methods, and tools to better design stormwater quality extended detention basins.



ORIFICE PLATE NOTES:

1. MINIMIZE THE NUMBER OF COLUMNS.
2. PROVIDE GASKET MATERIAL BETWEEN THE ORIFICE PLATE AND CONCRETE.
3. BOLT PLATE TO CONCRETE 12" MAX. ON CENTER.

EURV AND WQCV TRASH RACKS:

1. WELL-SCREEN TRASH RACKS (FOR CIRCULAR ORIFICES) SHALL BE STAINLESS STEEL AND SHALL BE ATTACHED BY INTERMITTENT WELDS ALONG THE EDGE OF THE MOUNTING FRAME.
2. BAR GRATE TRASH RACKS (FOR RECTANGULAR ORIFICES) SHALL BE ALUMINUM AND SHALL BE BOLTED USING STAINLESS STEEL HARDWARE.
3. TRASH RACK WIDTHS PROVIDED IN TABLE OS-2A AND OS-3A ARE FOR SPECIFIED TRASH RACK MATERIAL AND NEED TO BE ADJUSTED FOR MATERIALS HAVING A DIFFERENT OPEN AREA/GROSS AREA RATIO (R VALUE)
4. STRUCTURAL DESIGN OF TRASH RACKS SHALL BE BASED ON FULL HYDROSTATIC HEAD WITH ZERO HEAD DOWNSTREAM OF THE RACK.

OVERFLOW TRASH RACKS:

1. ALL TRASH RACKS SHALL BE MOUNTED USING STAINLESS STEEL HARDWARE AND PROVIDED WITH HINGED AND LOCKABLE OR BOLTABLE ACCESS PANELS.
2. TRASH RACKS SHALL BE STAINLESS STEEL, ALUMINUM, OR STEEL. STEEL TRASH RACKS SHALL BE HOT DIP GALVANIZED AND MAY BE HOT POWDER COATED AFTER GALVANIZING.
3. TRASH RACKS SHALL BE DESIGNED SUCH THAT THE DIAGONAL DIMENSION OF EACH OPENING IS SMALLER THAN THE DIAMETER OF THE OUTLET PIPE.
4. STRUCTURAL DESIGN OF TRASH RACKS SHALL BE BASED ON FULL HYDROSTATIC HEAD WITH ZERO HEAD DOWNSTREAM OF THE RACK.

Figure 2. As recently as 2010, the USDCM recommended a water quality metering plate with orifices spaced vertically 4" on center. This guidance often resulted in very small water quality orifices that were prone to clogging and created nuisance ponding of water and maintenance problems.

2. ELLIPTICAL SLOT WEIR ALTERNATIVE

In order to provide the slow metering of the WQCV necessary to remove sediment through settling, a V-notch weir was analyzed. It was apparent that the slot would have to be very narrow in order to not drain too quickly and an adjustment to the shape of the V-notch resulted in an elliptical slot. The principal benefit of the elliptical shape over the simple V shape is that it drains the top zone more quickly and the lower zone more slowly, allowing better settling of the storage volume and resulting in cleaner stormwater discharges. A schematic is shown as Figure 3.

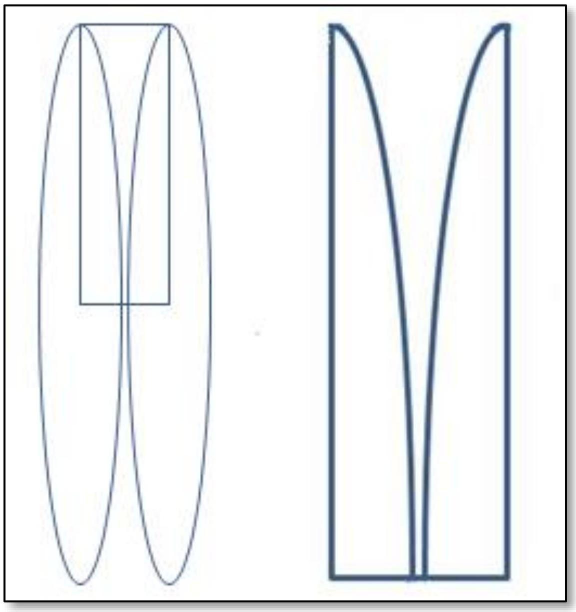
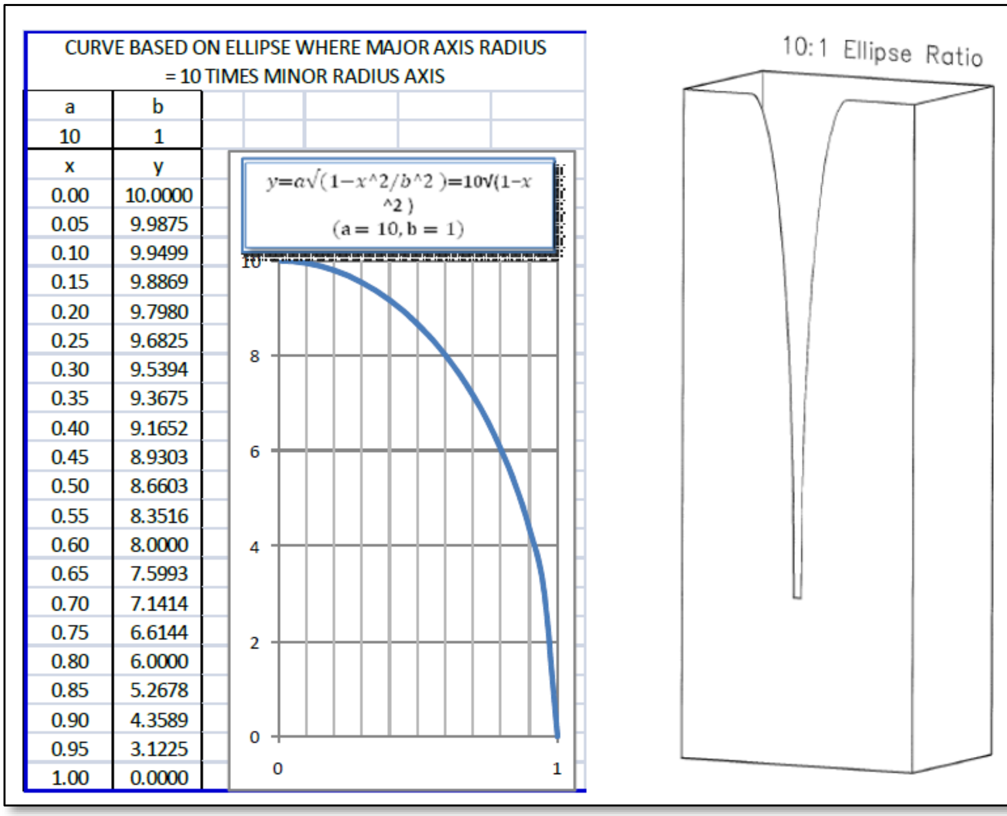


Figure 3. A visualization of the construction of the elliptical slot weir from the gap between the upper halves of two vertical ellipses having a large major-to-minor axis ratio.

2.1 Computational Fluid Dynamics (CFD) Modeling

In May 2011 UDFCD contracted with ARCADIS U.S., Inc. to perform Computational Fluid Dynamics (CFD) modeling of the weir. This modeling was based on a design where the major axis of the ellipse was ten times greater than the minor axis of the ellipse used to construct the profile of the weir. The gap width at the bottom of the notch was equal to 0.04 ft, and the total height of the weir was equal to 3.0 ft as shown in Figure 4.



a	b
10	1
x	y
0.00	10.0000
0.05	9.9875
0.10	9.9499
0.15	9.8869
0.20	9.7980
0.25	9.6825
0.30	9.5394
0.35	9.3675
0.40	9.1652
0.45	8.9303
0.50	8.6603
0.55	8.3516
0.60	8.0000
0.65	7.5993
0.70	7.1414
0.75	6.6144
0.80	6.0000
0.85	5.2678
0.90	4.3589
0.95	3.1225
1.00	0.0000

Figure 4. Elliptical slot weir design information.

Three different CFD models of the weir were constructed. The first model contained 216,000 control volumes (40 x 60 x 70) and the other models contained 25% more total control volumes and 25% less total control volumes. Comparison tests, based on results provided by these different models, were used to assess grid sensitivity. Other tests were also carried out to determine the sensitivity of model results to turbulence closure and program version. In all of the simulations carried out, an *.stl file was used to define the weir structure inside of the model grid.

In each of the calculations flow was introduced at the model boundary upstream of the weir (specified water surface elevation) and flow left the domain downstream of the weir (continuative boundaries at the at the bottom and downstream side of the grid). No-slip boundary conditions were specified at all solid walls, and two different turbulence closure schemes were invoked (the Renormalized Group (RNG) model for turbulence was used in some of the calculations and the standard k-e model for turbulence was used in others). Sample graphics showing the calculated fluid configuration for flows with head elevations equal to 1.0 ft and 2.0 ft are provided in Figure 5. In these visualizations the fluid free-surface is defined as the location of the three-dimensional contour where the volume fraction is equal to 0.5. In frames (c) and (d), the fluid body has been colored by pressure - a hydrostatic distribution exists upstream of the weir and pressures in the nappe are atmospheric.

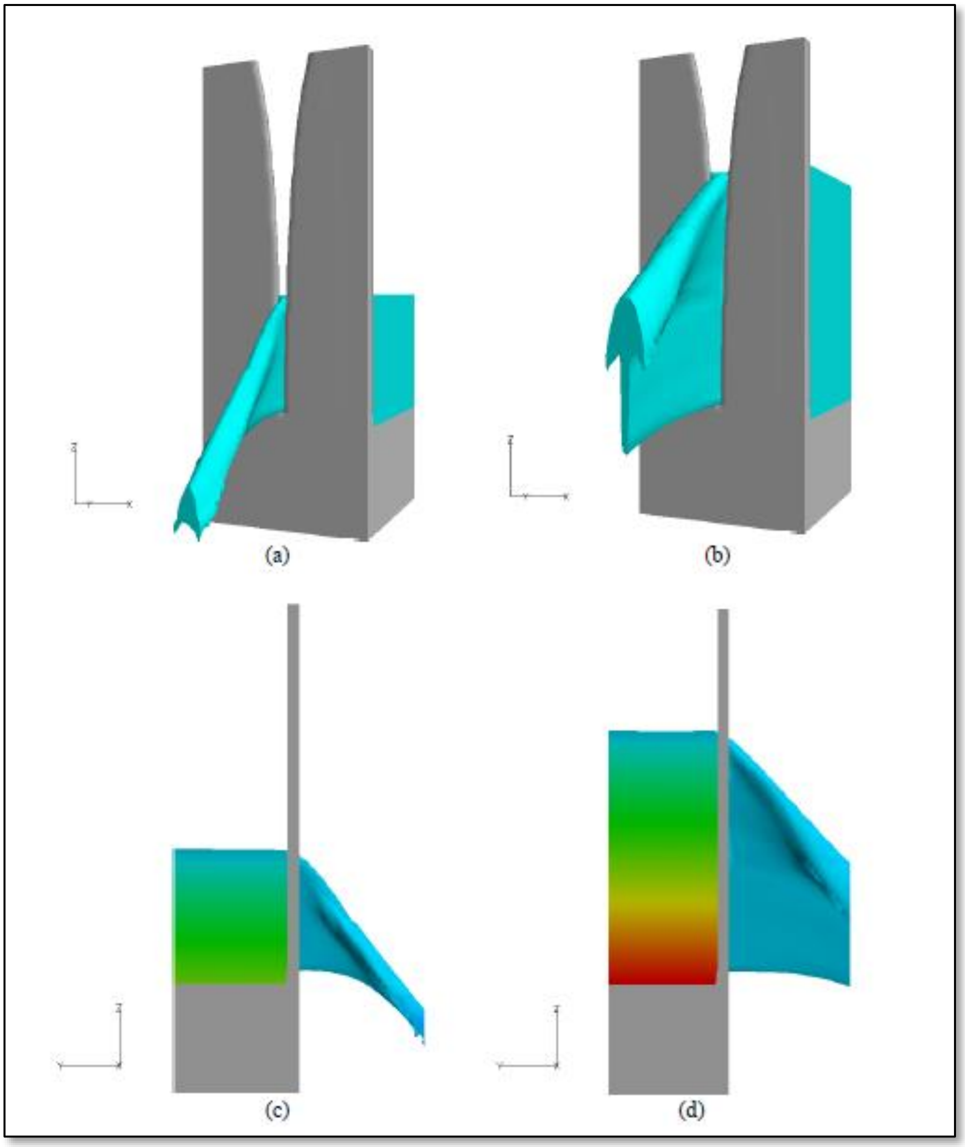


Figure 5. Fluid configuration: (a) 1.0 ft Head, (b) 2.0 ft Head, (c) 1.0 ft Head, Side View, Colored by Pressure – Common Scale, (d) 2.0 ft Head, Side View, Colored by Pressure – Common Scale.

Stage-discharge curves for an elliptical slot weirs having a 10:1 major-to-minor axis ratio and slot gaps of 0.01, 0.02, and 0.03 ft were developed from the CFD model and are shown in Figures 6 through 9.

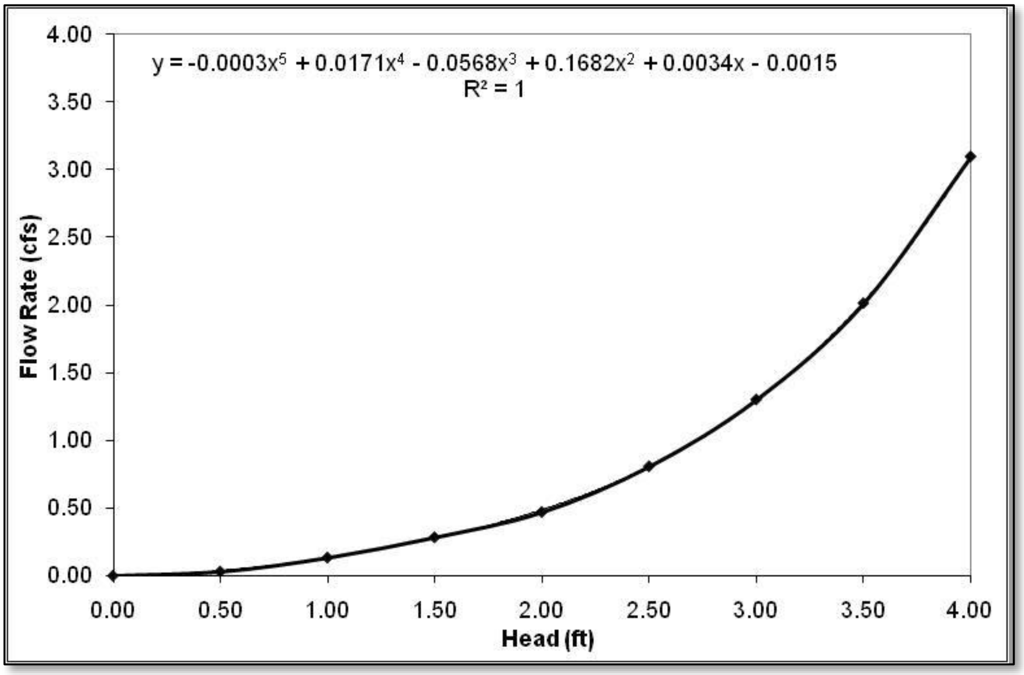


Figure 6. Stage-discharge curve for elliptical slot weir having a 10:1 major-to-minor axis and a slot width of 0.01 ft.

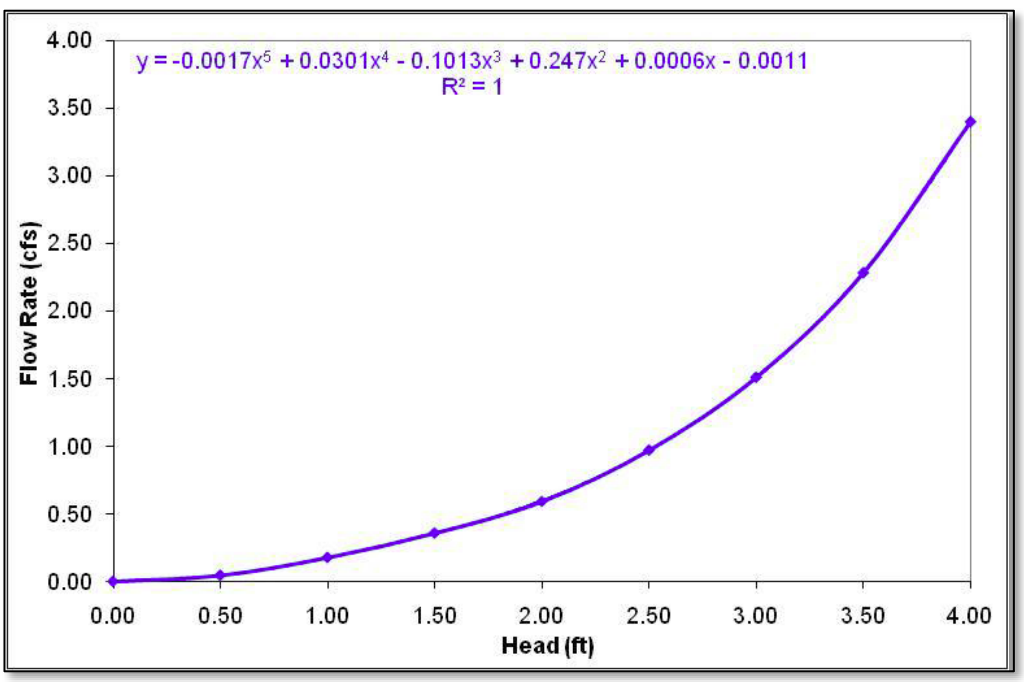


Figure 7. Stage-discharge curve for elliptical slot weir having a 10:1 major-to-minor axis and a slot width of 0.02 ft.

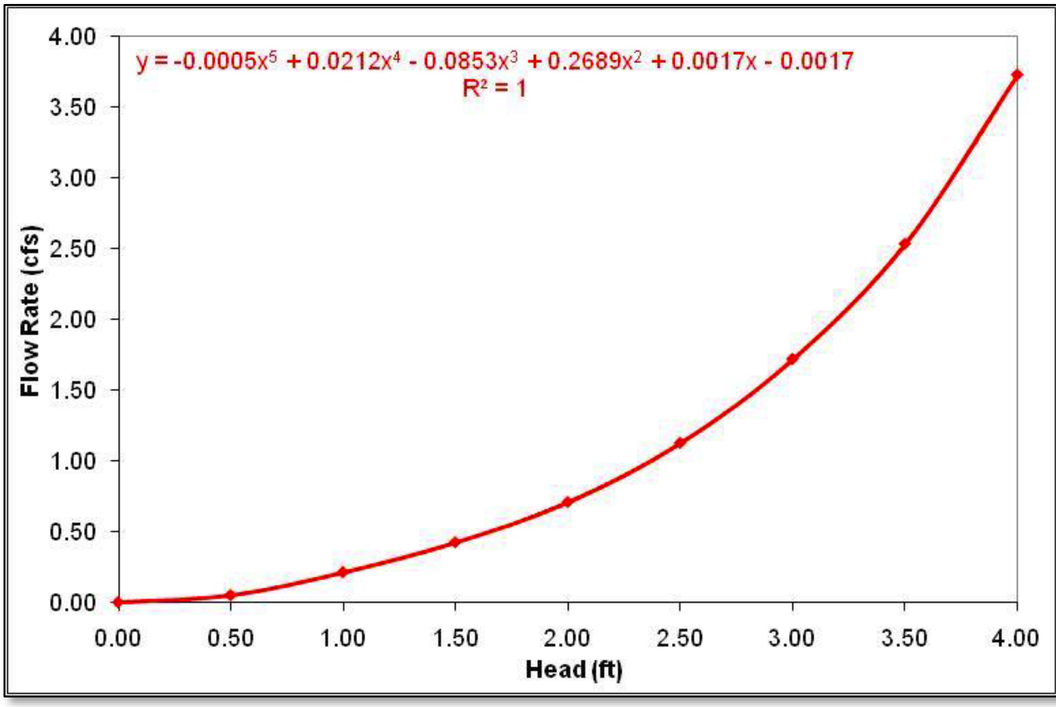


Figure 8. Stage-discharge curve for elliptical slot weir having a 10:1 major-to-minor axis and a slot width of 0.03 ft.

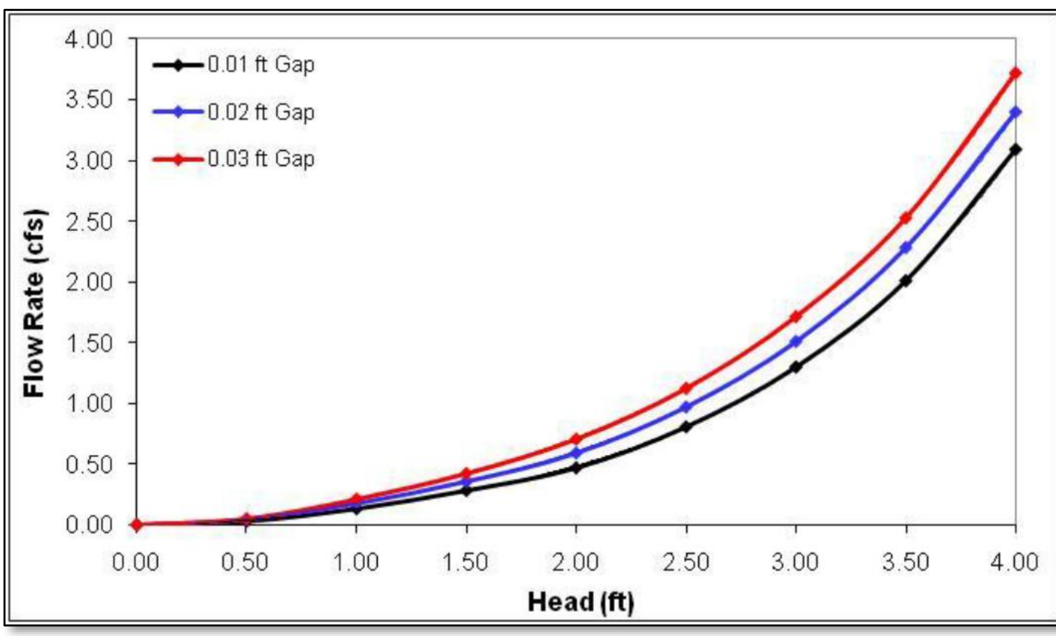


Figure 9. Comparison of stage-discharge family of curves for elliptical slot weirs having a 10:1 major-to-minor axis ratio and slot gaps of 0.01, 0.02, and 0.03.

2.2 Physical Modeling at Colorado State University

In December 2011 UDFCD contracted with Colorado State University to perform physical modeling of the elliptical slot weir. The results of that study were reported by Cox et al. in the ASCE Journal of Irrigation and Drainage Engineering in June 2014 (Volume 140, Issue 6) and are repeated here. A 2:1 Froude-scale physical model was constructed and stage-discharge data were collected to analyze the stage-discharge relationship of the new weir. A total of 45 steady-state tests were conducted encompassing nine unique weir geometric configurations. The ellipse ratio varied from 12 to 16, and the gap width varied from 1.5 to 9.1 mm (0.005 to 0.030 ft). A theoretical rating equation was derived for the elliptical weir and a discharge coefficient of 0.642 was determined from analyzing the physical-model data. Trapezoidal integral approximation was used to develop an explicit approximate solution for the theoretical rating equation. By using the trapezoidal integral approximation, measured discharges were predicted with a mean absolute percent error of 3.55% for the data set, excluding discharges lower than 2.83 L/s (0.10 cfs).

The objective of this research was to develop a rating equation for the elliptical sharp-crested weir. The elliptical sharp-crested weir was fabricated and tested to provide data for validation of a theoretical rating equation and calibration of discharge coefficients.

A common approach for developing theoretical stage-discharge relationships for weirs is integrating the flow velocity over elementary flow layers as shown by Eq. 1:

$$Q = \int_0^h U\, dA = \int_0^h \sqrt{2gy'}\, L dy \tag{1}$$

Where Q = outlet discharge [L^3T^{-1}]; U = flow velocity [LT^{-1}]; dA = elementary flow area [L^2]; g = gravitational acceleration [LT^{-2}]; y' = distance measured from the water surface (see Figure 10(a)) [L]; L = weir-opening length [L]; h = head above the horizontal sill [L]; and dy = elementary flow vertical distance [L]. Discharge coefficients are generally a function of weir geometry, approach velocity, fluid viscosity, and surface tension. The FHWA HEC-22 presented the rating equation for a proportional weir with a linear head-discharge relationship and a 0.62 coefficient of discharge (FHWA 2009).

Sharp-crested weirs were constructed with 16:1, 12:1, and 14:1 ellipse ratios at the Engineering Research Center (ERC) of CSU using computer numeric control (CNC) technology with linear positional tolerance of 12.7 μm (0.0005 inch). Each ellipse ratio was tested with three different gap widths ranging from 1.5 to 9.1 mm (0.005 to 0.030 ft) to provide laboratory data for the development of a stage-discharge prediction equation. Gap widths were verified using a caliper with an accuracy of ±50.8 μm (0.002 inch).

The physical model was constructed within an existing facility that measured 2.44-m (8-ft) wide, 10.82-m (35.5-ft) long, and 0.91-m (3-ft) deep with a constant longitudinal slope of 0.0135 m/m as shown in Figure 10. The model consisted of a supply pipe network, a flume headbox, a flume/reservoir section containing the weir outlet, a tailbox to capture returning flow, and the supporting superstructure. The ellipse weir was located inside the flume section and was the only flow outlet. The vertical distance measured from the bottom of the approach channel to the weir crest was constant at 0.1524 m (0.5 ft) for all tests. A diffuser screen was installed at the junction between the headbox and the reservoir section to provide quiescent approach-flow conditions. A 2:1 exact Froude scale was chosen for the model study based on maximizing the model size with the available laboratory space. Additionally, the weir crest incorporated a 1-mm horizontal section followed by a 45° taper on the downstream side of the weir which is consistent with sharp-crested weir specifications.

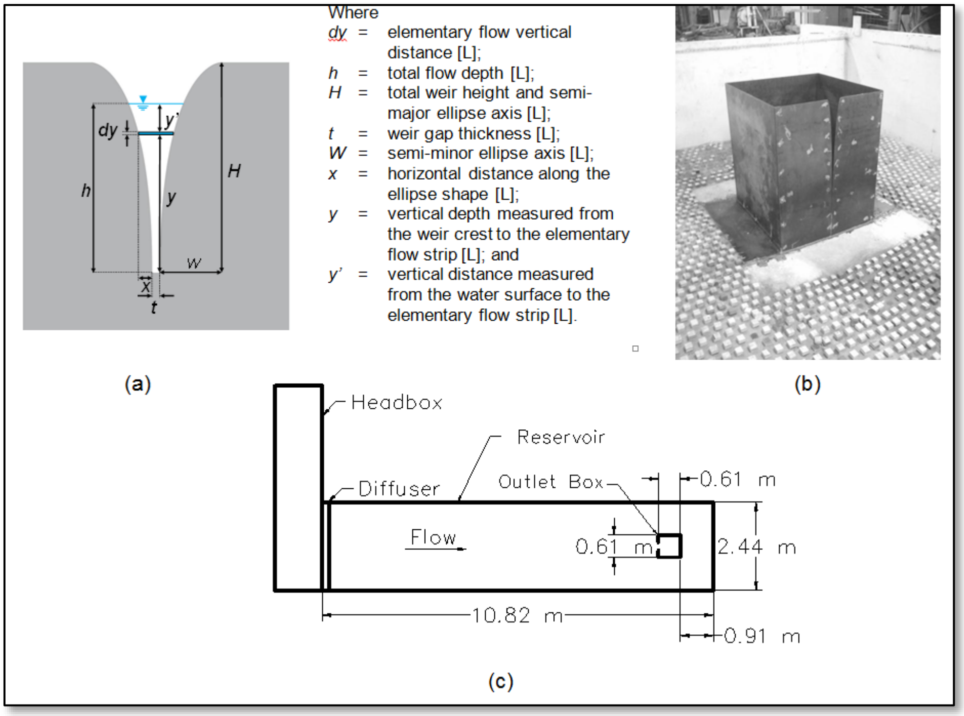


Figure 10. Elliptical sharp-crested weir: (a) cross-section sketch of elliptical-weir parameters; (b) photograph of the elliptical sharp-crested weir; (c) plan-view sketch of the test flume setup.

For all tests, water was gravity-fed into the flume directly from Horsetooth Reservoir. Discharge to the flume was controlled with two gate valves on a 76-mm (3-inch) pipeline and was measured using a Venturi meter with an accuracy of ±2.5%. Water-surface elevations were recorded approximately 0.6 m (2 ft) upstream of the outlet using a Vernier point gage (accurate to ±0.30 mm (±0.001 ft)). The point-gage measurement location was chosen to ensure the elevation was not within the draw-down section at the outlet. Additionally, water-surface elevations were measured using a pressure transducer located on the side of the outlet box away from the outlet draw-down section.

Initially, the discharge was set to achieve a flow depth corresponding to the top of the 0.61-m (2-ft) weir to determine the maximum capacity of the weir before overtopping the entire outlet box. Subsequently, steady-state discharge tests were conducted using 15, 26, 42, 65, and 100% of maximum flow capacity for each of the nine configurations resulting in a total of forty-five steady-state tests. Steady-state conditions were achieved for each test, where the discharge was set at a constant value and the water-surface elevation was monitored over time using the pressure transducer with LabVIEW software. Data collected for each test included water temperature, water-surface elevations, and discharge. Figure 11 provides the stage-discharge relationship for the 14:1 tests to illustrate the general stage-discharge trends for the elliptical weir. Stage-discharge data for the 16:1, 12:1, and 14:1 configurations are provided in Table 2.

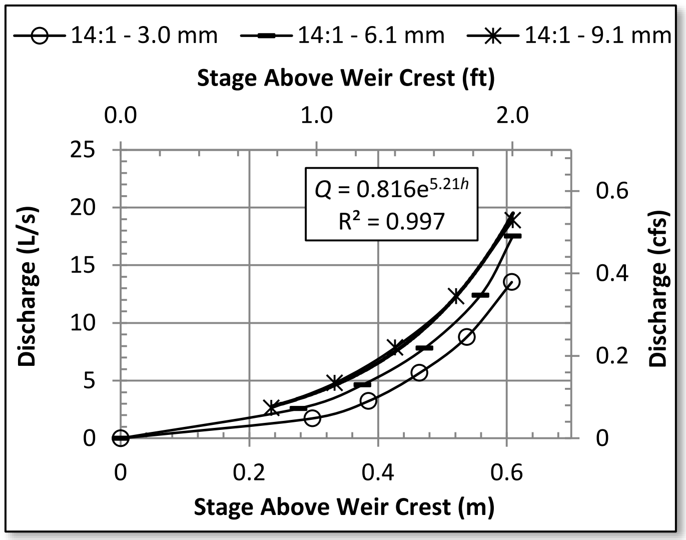


Figure 11. Stage-discharge relationship for the 14:1 elliptical weir with 3.0, 6.1, and 9.1 mm (0.010, 0.020, and 0.030 ft) gap widths, and exponential trend for 9.1 mm.

Table 1. Physical-modeling Data, Computed Theoretical Flow Rates, Predicted Flow Rates, and Percent Errors

	Test ID	t		h		Q_{meas}		Q_{int}		Q_{app}		Q_{pred}		% Error
		(cm)	(ft)	(m)	(ft)	(L/s)	(cfs)	(L/s)	(cfs)	(L/s)	(cfs)	(L/s)	(cfs)	
16:1 Ellipse Ratio	1	0.152	0.005	0.314	1.031	1.13	0.040	2.04	0.072	2.03	0.072	1.30	0.046	15%
	2	0.152	0.005	0.394	1.292	2.55	0.090	3.91	0.138	3.91	0.138	2.51	0.089	-1%
	3	0.152	0.005	0.469	1.538	4.39	0.155	6.76	0.239	6.79	0.240	4.36	0.154	-1%
	4	0.152	0.005	0.536	1.760	6.80	0.240	10.60	0.375	10.68	0.377	6.85	0.242	1%
	5	0.152	0.005	0.610	2.000	10.73	0.379	16.84	0.595	16.91	0.597	10.85	0.383	1%
	6	0.305	0.010	0.300	0.985	1.44	0.051	2.54	0.090	2.51	0.089	1.61	0.057	12%
	7	0.305	0.010	0.384	1.261	3.00	0.106	4.71	0.166	4.69	0.166	3.01	0.106	0%
	8	0.305	0.010	0.476	1.563	5.61	0.198	8.60	0.304	8.60	0.304	5.52	0.195	-2%
	9	0.305	0.010	0.546	1.791	8.44	0.298	13.08	0.462	13.11	0.463	8.41	0.297	0%
	10	0.305	0.010	0.610	2.000	13.11	0.463	18.98	0.670	19.00	0.671	12.19	0.430	-7%
	11	0.457	0.015	0.294	0.963	1.64	0.058	3.12	0.110	3.07	0.109	1.97	0.070	20%
	12	0.457	0.015	0.377	1.238	3.28	0.116	5.53	0.195	5.48	0.193	3.51	0.124	7%
	13	0.457	0.015	0.459	1.507	5.52	0.195	9.13	0.323	9.08	0.321	5.83	0.206	6%
	14	0.457	0.015	0.539	1.768	8.64	0.305	14.33	0.506	14.30	0.505	9.18	0.324	6%
	15	0.457	0.015	0.610	2.000	13.37	0.472	21.12	0.746	21.08	0.744	13.52	0.478	1%
12:1 Ellipse Ratio	16	0.152	0.005	0.317	1.039	1.76	0.062	2.51	0.089	2.51	0.089	1.61	0.057	-8%
	17	0.152	0.005	0.395	1.297	3.51	0.124	4.91	0.174	4.93	0.174	3.16	0.112	-10%
	18	0.152	0.005	0.467	1.531	5.78	0.204	8.41	0.297	8.46	0.299	5.43	0.192	-6%
	19	0.152	0.005	0.539	1.767	9.09	0.321	13.74	0.485	13.86	0.489	8.89	0.314	-2%
	20	0.152	0.005	0.610	2.000	13.93	0.492	21.73	0.767	21.86	0.772	14.03	0.495	1%
	21	0.305	0.010	0.314	1.031	1.90	0.067	3.25	0.115	3.22	0.114	2.07	0.073	9%
	22	0.305	0.010	0.391	1.284	3.54	0.125	5.86	0.207	5.85	0.207	3.75	0.132	6%
	23	0.305	0.010	0.468	1.537	5.97	0.211	9.96	0.352	9.97	0.352	6.40	0.226	7%
	24	0.305	0.010	0.539	1.770	9.17	0.324	15.61	0.551	15.67	0.553	10.06	0.355	10%
	25	0.305	0.010	0.610	2.000	14.41	0.509	23.87	0.843	23.94	0.845	15.36	0.542	7%
	26	0.457	0.015	0.277	0.908	2.12	0.075	3.02	0.107	2.98	0.105	1.91	0.067	-10%
	27	0.457	0.015	0.370	1.215	4.13	0.146	6.04	0.213	5.99	0.212	3.84	0.136	-7%
	28	0.457	0.015	0.446	1.462	6.51	0.230	9.89	0.349	9.86	0.348	6.33	0.223	-3%
	29	0.457	0.015	0.531	1.742	10.56	0.373	16.55	0.584	16.56	0.585	10.62	0.375	1%
	30	0.457	0.015	0.610	2.000	16.28	0.575	26.02	0.919	26.02	0.919	16.70	0.590	3%
14:1 Ellipse Ratio	31	0.914	0.03	0.234	0.769	2.63	0.093	3.57	0.126	3.48	0.123	2.24	0.079	-15%
	32	0.914	0.03	0.333	1.092	4.79	0.169	6.93	0.245	6.81	0.240	4.37	0.154	-9%
	33	0.914	0.03	0.427	1.400	7.87	0.278	11.82	0.418	11.67	0.412	7.49	0.264	-5%
	34	0.914	0.03	0.522	1.711	12.32	0.435	19.23	0.679	19.07	0.673	12.24	0.432	-1%
	35	0.914	0.03	0.610	2.000	18.89	0.667	29.64	1.047	29.44	1.040	18.89	0.667	0%
	36	0.610	0.02	0.276	0.907	2.58	0.091	3.52	0.124	3.45	0.122	2.22	0.078	-14%
	37	0.610	0.02	0.376	1.234	4.64	0.164	6.87	0.243	6.78	0.240	4.35	0.154	-6%
	38	0.610	0.02	0.472	1.550	7.82	0.276	12.10	0.427	12.02	0.424	7.71	0.272	-1%
	39	0.610	0.02	0.559	1.835	12.40	0.438	19.40	0.685	19.34	0.683	12.41	0.438	0%
	40	0.610	0.02	0.610	2.000	17.53	0.619	25.36	0.896	25.28	0.893	16.22	0.573	-7%
	41	0.305	0.01	0.298	0.978	1.73	0.061	2.64	0.093	2.62	0.092	1.68	0.059	-3%
	42	0.305	0.01	0.385	1.264	3.23	0.114	5.11	0.181	5.09	0.180	3.27	0.115	1%
	43	0.305	0.01	0.465	1.524	5.66	0.200	8.72	0.308	8.72	0.308	5.60	0.198	-1%
	44	0.305	0.01	0.538	1.765	8.78	0.310	13.76	0.486	13.80	0.487	8.85	0.313	1%
	45	0.305	0.01	0.608	1.994	13.54	0.478	20.85	0.736	20.89	0.738	13.40	0.473	-1%

Where Q_{meas} = discharge measured in the physical model [L^3T^{-1}]; Q_{int} = discharge computed by implicitly solving the integral in Eq. (5) [L^3T^{-1}]; Q_{app} = discharge computed using the trapezoidal integral approximation [L^3T^{-1}]; Q_{pred} = predicted discharge [L^3T^{-1}]; and all other variables have been previously defined.

2.21 Development of Theoretical Rating Equation

The elliptical weir stage-discharge data exhibited an exponential trend as illustrated by the exponential trend line for the 9.1-mm configuration shown in Figure 7. Although an exponential trend fits the measured data well, the discharge does not approach a value of zero when the stage value approaches zero. To accurately predict discharge throughout the entire range of stage values, a theoretical rating equation was developed for the elliptical weir following the method described by Horton (1906) and presented in Eq. 1. Figure 10(a) provides a sketch identifying the variables used in the derivation. To determine an explicit solution for the elliptical weir using the integral in Eq. 1, expressions were derived for the flow velocity (*U*) and the weir opening length (*L*) as a function of the vertical depth measured from the weir crest to the elementary flow strip (*y*). Eq. 2 provides the expression for flow velocity as a function of the total flow depth (*h*) and the vertical depth from the weir crest to the elementary flow layer (*y*)

$$U = \sqrt{2gy'} = \sqrt{2g(h-y)} \tag{2}$$

where all variables have been previously defined. To determine *L* as a function of *y*, initially the expression for the horizontal distance along the ellipse shape (*x*) was determined as Eq. 3:

$$x = \frac{H}{R} - \frac{H}{R}\sqrt{1-\frac{y^2}{H^2}} \tag{3}$$

Accordingly, the weir-opening length (*L*) is equal to the sum of twice the horizontal distance along the ellipse shape (*x*) and the weir-gap thickness (*t*) as shown by Eq. 4:

$$L = 2x + t = 2\left(\frac{H}{R} - \frac{H}{R}\sqrt{1-\frac{y^2}{H^2}}\right) + t \tag{4}$$

Typically, end contractions are considered when computing the weir-opening length (*L*) and an effective length is determined by subtracting the product of 0.1 times the number of contractions and the head above the horizontal sill (*h*) (Horton 1906); however, for the ellipse weir, using that expression to compute effective weir-opening length resulted in negative weir-opening lengths for even the largest value of *h* (0.610 m (2.0 ft)). Therefore, the effect of end contractions was not included in the computation of the weir-opening length for the ellipse weir. Substitution of Eq. 2 and Eq. 4 into Eq. 1 provides the final form of the integral for discharge

$$Q = \int_0^h \sqrt{2g(h-y)}\left[2\frac{H}{R}\left(1-\sqrt{1-\frac{y^2}{H^2}}\right)+t\right]dy \tag{5}$$

where all variables have been previously defined.

An explicit solution is not obtainable for Eq. 5 due to the complexity of the equation; therefore, trapezoidal numerical integration of Eq. 5 was used to determine an approximate solution of the definite integral (Jeffrey 1995). Eq. 6 provides the general expression for trapezoidal approximation of Eq. 5 using non-uniform intervals

$$\int_0^h f(y)dy \approx \frac{1}{2}\sum_{k=1}^{N}(y_{k+1}-y_k)(f(y_{k+1})+f(y_k)) \tag{6}$$

Where k = integer for individual intervals; and N = total number of intervals. Eq. 7 provides the function equation $f(y)$ for approximation of Eq. 5 using Eq. 6:

$$f(y) = \sqrt{2g(h-y)}\left[2\frac{H}{R}\left(1-1\sqrt{1-\frac{y^2}{H^2}}\right)+t\right] \tag{7}$$

Through an optimization analysis comparing the implicit integral solution to the explicit trapezoidal approximation, the optimal intervals for the trapezoidal approximation were determined to be 0 to 0.603, 0.603 to 0.886, and 0.886 to 1.000 times the flow depth (h). Eq. 8 provides the simplified expression for trapezoidal numerical approximation of Eq. 5 using Eq. 7 with the optimal intervals:

$$\begin{aligned} Q_{app} &= 0.3015h[f(0)+f(0.603h)] \\ &+ 0.1415h[f(0.603h)+f(0.886h)] \\ &+ 0.0570h[f(0.886h)] \end{aligned} \tag{8}$$

The trapezoidal approximation (Eq. 6) predicted the integral solution with a mean absolute percent error of 0.62%.

A key objective for the weir design is to convey a majority of the flow from the upper portion of the water column. Figure 4 shows the theoretical percent of total flow conveyed about a given vertical depth *y* versus the distance along the vertical depth (*y*) for the example scenario. Approximately 50% of the flow is conveyed in the top one-third of the water column.

2.22 Weir Discharge Coefficient Analysis

A discharge coefficient (C_d) was necessary to correct the theoretical flow equation for energy losses, velocity distribution, and streamline curvature. Three parameters were evaluated to determine the coefficient of discharge for the elliptical weir: (1) the discharge measured in the physical model (Q_{meas}); (2) the discharge solved implicitly from the integral (Q_{int}); and (3) the discharge calculated from the integral approximation (Q_{app}). Initially, values for Q_{int} and Q_{app} were computed for each of physical model tests, where Q_{int} was computed from Eq. 5 using Maple™, a mathematics software program (Maplesoft™ 2013), and Q_{app} was computed using Eq. (8). Discharges predicted using the integral solution (Q_{int}) were compared to measured discharges (Q_{meas}) to evaluate the discharge coefficient. Measured discharges were predicted using discharges computed from implicitly solving Eq. 5 (Q_{int}) with the discharge coefficient (C_d) of 0.642. The predicted discharges had a mean absolute percent error of 5.11% for the entire data set and a mean absolute percent error of 3.49% for the data set excluding discharges lower than 2.83 L/s (0.10 cfs). The residual errors are evenly distributed at both high and low flows. This indicates that the effect of end contractions was encompassed within the constant discharge coefficient without the introduction of any bias associated with flow depth.

The integral approximation, Eq. 8, provides a solution that can be directly computed (explicit), which is preferable over the implicit integral technique for its ease of use. Discharge can be predicted as a function of the discharge computed using the integral approximation (Q_{app}) and the discharge coefficient (C_d)

$$Q_{pred} = C_d Q_{app} \tag{9}$$

Where C_d = 0.642.

Discharges were predicted for the elliptical-weir data set using Eq. 9. The percent errors are elevated for the low discharges because the error is large relative to the magnitude of the discharges; however, the magnitudes of the errors for the lower discharges are not generally greater than the errors for the remaining discharges. The mean absolute percent error for the entire data set is 5.20% and the mean absolute percent error for the data set excluding discharges lower than 2.83 L/s (0.10 cfs) is 3.55%.

2.23 Qualitative Observations of Debris Handling Characteristics

Qualitative debris handling tests were performed by introducing a number of neutrally buoyant items into the detained water volume as shown in Figure 12. While many of the smaller pieces of introduced debris slipped through the elliptical slot unimpeded, larger masses were caught in the slot. While these larger pieces did have to be cleared by hand, they did not completely block the elliptical slot, in every case leaving the top 1/3 – 1/2 open for flow. Initial sizing estimates indicated that the elliptical slot weir is preferable to an orifice plate for larger detention basins, but not for very small detention basins. This is due to the fact that for very small detention basins, the slot in the weir becomes unmanageably narrow. From the laboratory observations, any slot narrower than 3/8” would pose clogging issues and become a maintenance problem.

Figure 12. Qualitative observations on debris handling indicate that the elliptical slot weir handles debris better than does an orifice plate with a well screen covering the water quality orifices.

2.3 Field Installation

Two sites were selected for field installation of the elliptical slot weirs, those being the Northfield Detention Basin and the USPS Detention Basin (Figure 13). Using the slot sizing guidance produced by CSU, both of these detention basins were retrofitted with slot weirs.

Figure 13. Extended Detention basins at Northfield Stapleton where traditional water quality orifice plates were removed and replaced with elliptical slot weirs.

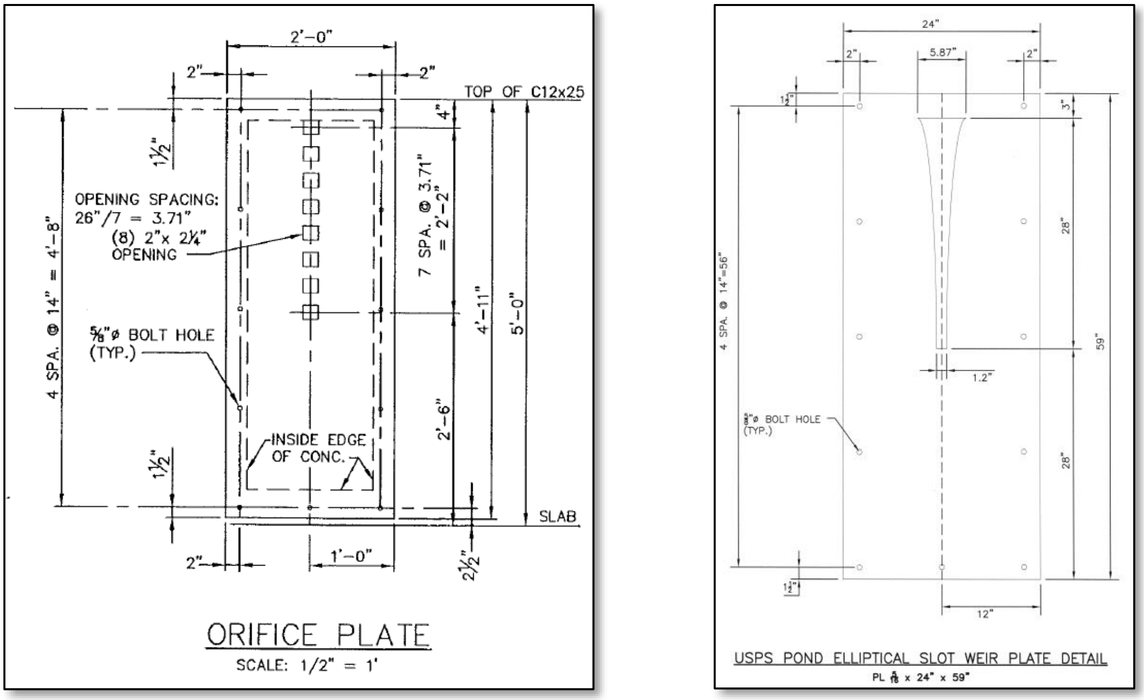


Figure 14. Fabrication details of (left) water quality orifice plate, and (right) elliptical slot weir for USPS Detention Basin.

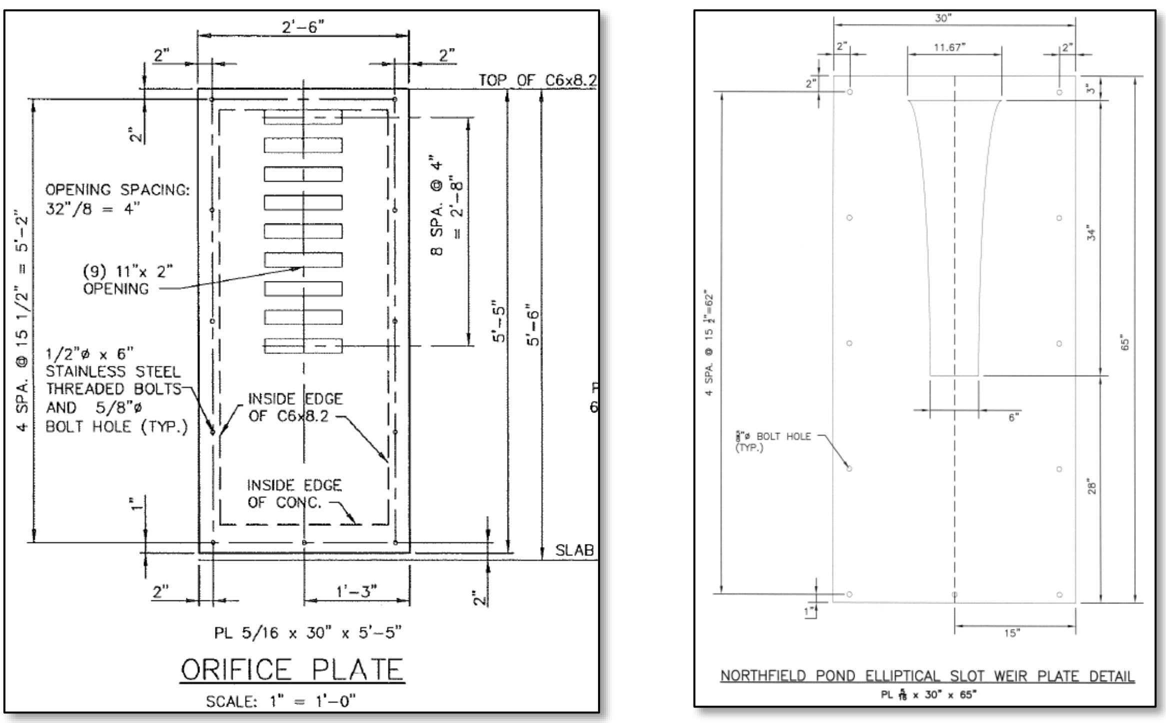


Figure 15. Fabrication details of (left) water quality orifice plate, and (right) elliptical slot weir for Northfield Detention Basin.

Figure 16. Actual installation of (left) water quality orifice plate, and (right) elliptical slot weir for the Northfield Detention Basin.

Figure 17. Levelogger™ pressure transducer installation over 2014 and 2015 rainfall seasons allowed testing of the elliptical slot weir in the field.

Each of the two sites were fitted with stilling wells for Levelogger™ pressure transducers. Data were collected for the two precipitation years of 2014 and 2015, in an effort to determine whether the weir sizing algorithm that had been adopted in the laboratory produced good drain time results in a real world setting. The data from these Leveloggers™ were analyzed in early 2016.The results are produced in Figures 18 and 19.

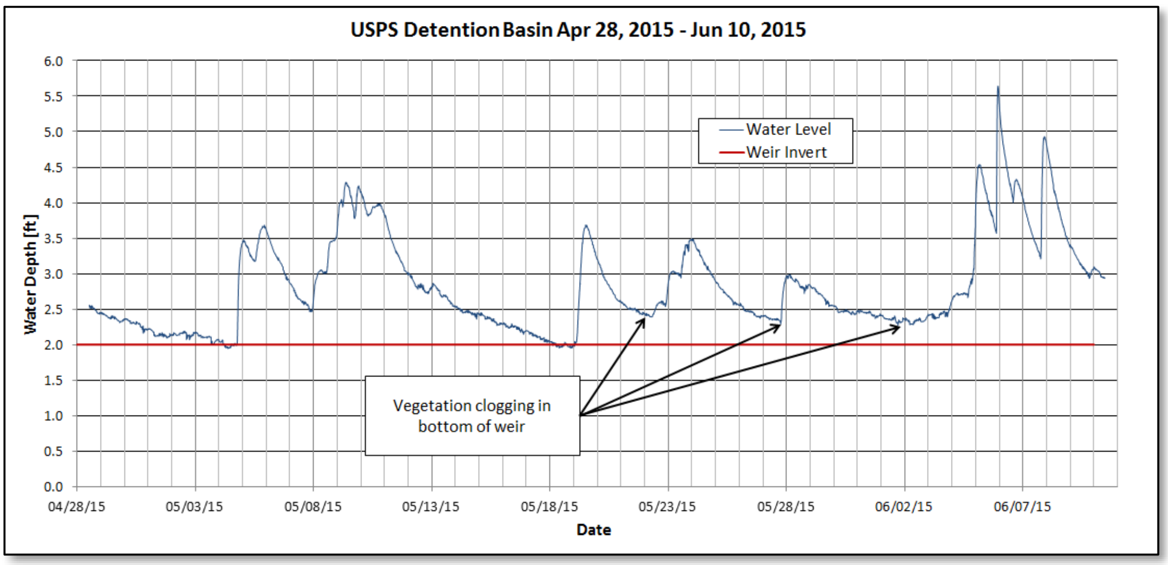


Figure 18. USPS detention basin storage levels during period of April 28, 2015 through June 10, 2015. The bottom 6 inches of the slot began to clog with cattails during the storm on May 19th.

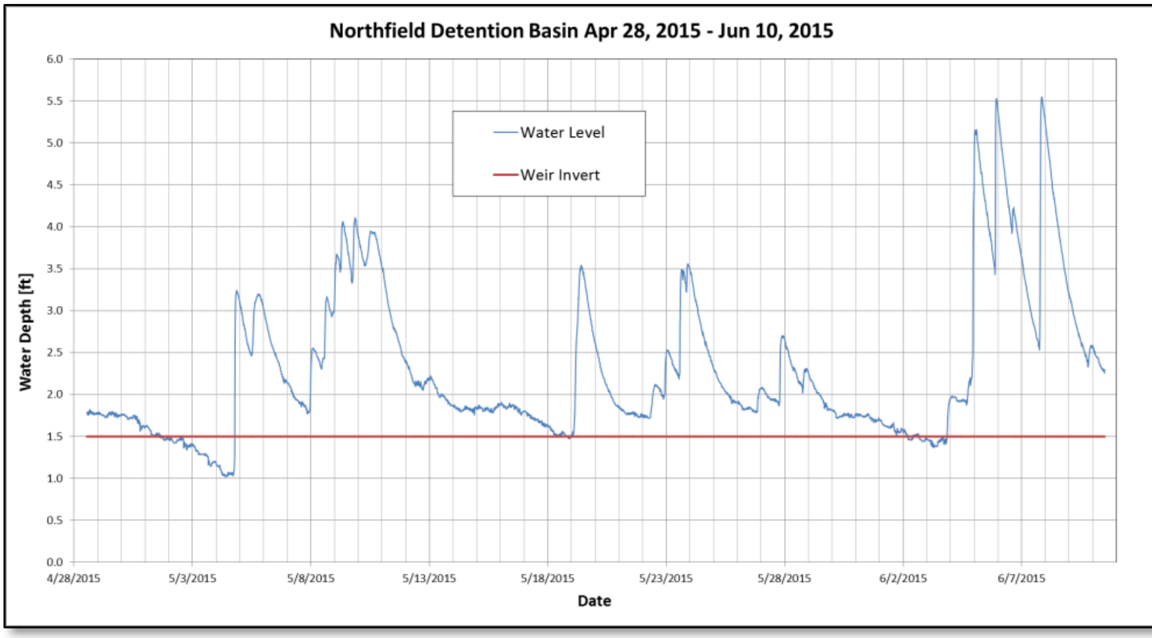


Figure 19. Northfield detention basin storage levels during period of April 28, 2015 through June 10, 2015.

Difficulties encountered during the monitoring period included the removal of the Northfield elliptical weir plate by the metropolitan district (and replacement with an orifice plate) during maintenance, subsequent leakage of the replacement plate, and clogging of the narrower weir slot at the USPS detention basin. Both of these detention basins have large permanent pools with heavy wetland vegetation cover. This creates maintenance issues regardless of what type of outlet plate is chosen. The graphs in Figures 18 and 19 indicate that, in the absence of clogging, the basins emptied the WQCV in approximately 40 hours, which was the goal of the design.

3. MAXIMIZED ORIFICE AREA ALTERNATIVE

The qualitative debris handling investigation in the CSU hydraulics laboratory and the two-year field testing made it clear that while the elliptical slot weir handles debris very well when the slot is wide (say greater than 1 inch), debris clogging becomes an issue as the slot grows more narrow. Based on these investigations, UDFCD does not recommend an elliptical slot weir having a slot width of less than 3/8-inch. This equates roughly to a WQCV of one acre-ft or larger, assuming a 40-hour drain time; or an excess urban runoff volume (EURV, refer to the USDCM Volume 3 for details on the EURV concept) of 1.6 acre-ft or larger, assuming a 60-hour drain time.

Understanding that most of CDOT's stormwater extended detention basins will not be large enough to qualify for the application of the elliptical slot weir, the orifice plate concept was re-evaluated to determine if there was a way to minimize clogging with that type of outlet configuration. As shown in Figure 2, the standard of practice as promulgated by UDFCD in the USDCM Volume 3 since 1999 was a column (or multiple columns) of water quality orifices spaced 4 inches vertically on center. In the USDCM, a well screen as shown in Figure 20 was specified for circular openings up to 2 inches in diameter and a bar grate as shown in Figure 21 was specified for larger orifices. The problem with this strategy was that 1) the well screen is prone to clogging, and 2) orifices larger than 2 inches in diameter were a rarity due to the close vertical spacing so the bar grate was seldom applicable.

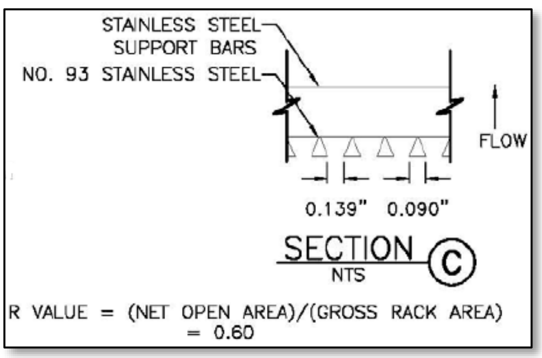


Figure 20. Well Screen from 1999 USDCV Vol 3.

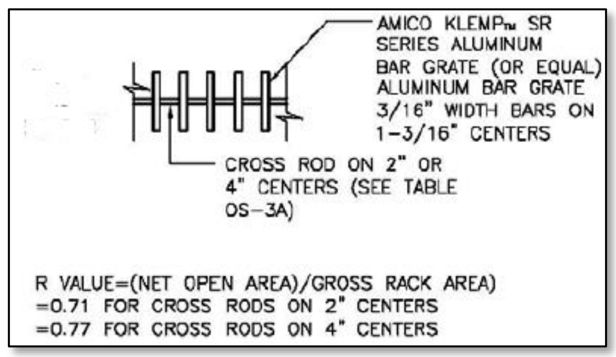


Figure 21. Bar Grate from 1999 USDCV Vol 3.

Figure 22 shows an all-too-common clogged well screen. The solution to the problem for these basins that are too small to warrant the application of the elliptical slot weir is to maximize the open area of each individual orifice such that the lower maintenance bar can be applied in lieue of grate the higher maintenance well screen.

Figure 22. Typical frequent clogging issues associated with well screens (Grant Ranch Research Extended Detention Basin, Denver, CO 2009).

From a hydrualic standpoint, the ideal scenario would be one water quality orifice at the bottom of the WQCV that would drain the entire volume in 40 hours. But from a water quality perspective, this results in the resuspension and release of more sediment as compared to a column of smaller orifices. Resuspension and an increased amount of sediment release is due to concentration of sediment and associated pollutants being larger toward the bottom of the WQCV. This causes the extended detention basin less effective. It was determined that three is the minimum number of orifices to properly drain the WQCV without releasing excessive sediment. The most recent update of the USDCM Volume 3 reflects this change in practice. UDFCD now recommends only three orifices to maximize the individual orifice area and avoid clogging of the orifice plate. A detail showing the recommended orifice configuration is provided in Figure 23.

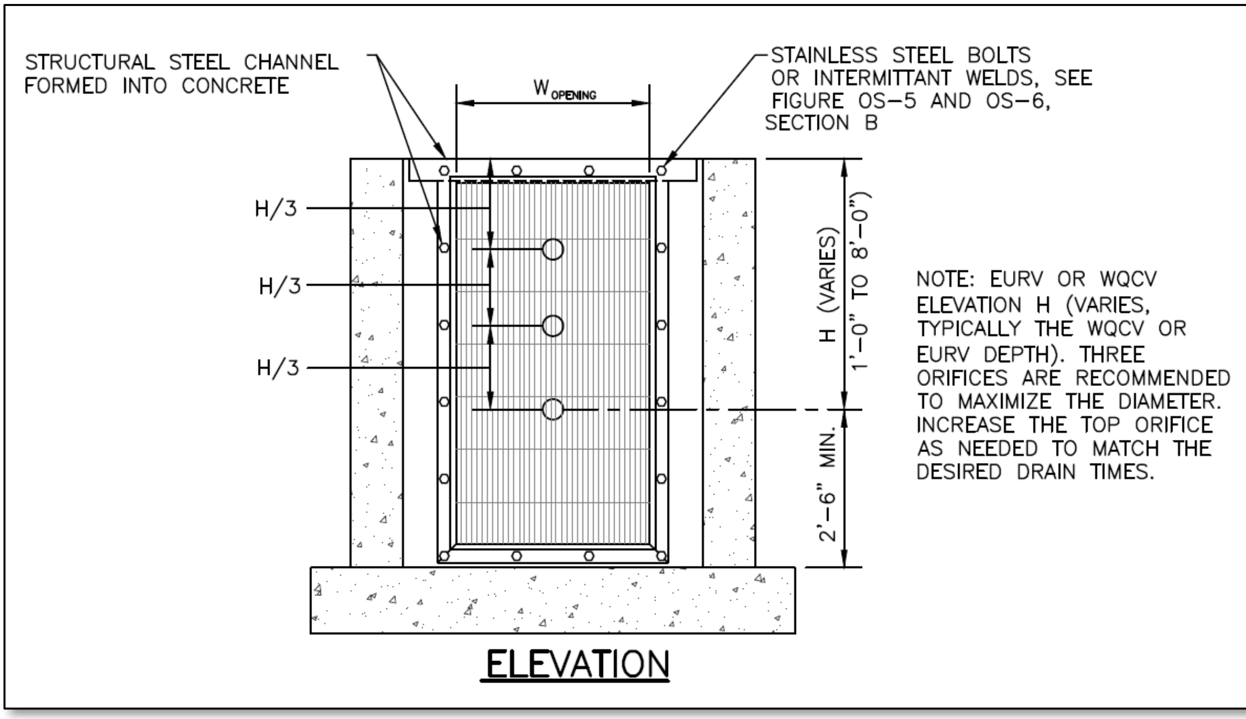


Figure 23. New practice of minimizing the number of water quality orifices while maximizing the area of each individual orifice is presented in the 2016 USDCM Volume 3 as Figure OS-4 in Fact Sheet T-12.

In the case of a detention basin incorporating the EURV and WQCV, the top orifice oftentimes will need to be enlarged such that the lower two orifices drain the WQCV in 40 hours and the top orifice works with the others to drain the EURV in less than 72 hours. The 72-hour rule will be discussed in Section 6 of this report.

4. NEW SIZING GUIDANCE FOR OVERFLOW OUTLET

Detention basins that provide flood control in addition to stormwater quality management have outlet structures fitted with metering plates (either elliptical slot weirs or orifice plates). This is used to control the release of the EURV and/or WQCV, and have an overflow outlet to direct flows in excess of the EURV and/or WQCV into the outlet vault. This is where typically the 100-year volume is metered into the receiving system via a restrictor on the final discharge pipe, as shown in Figure 24.

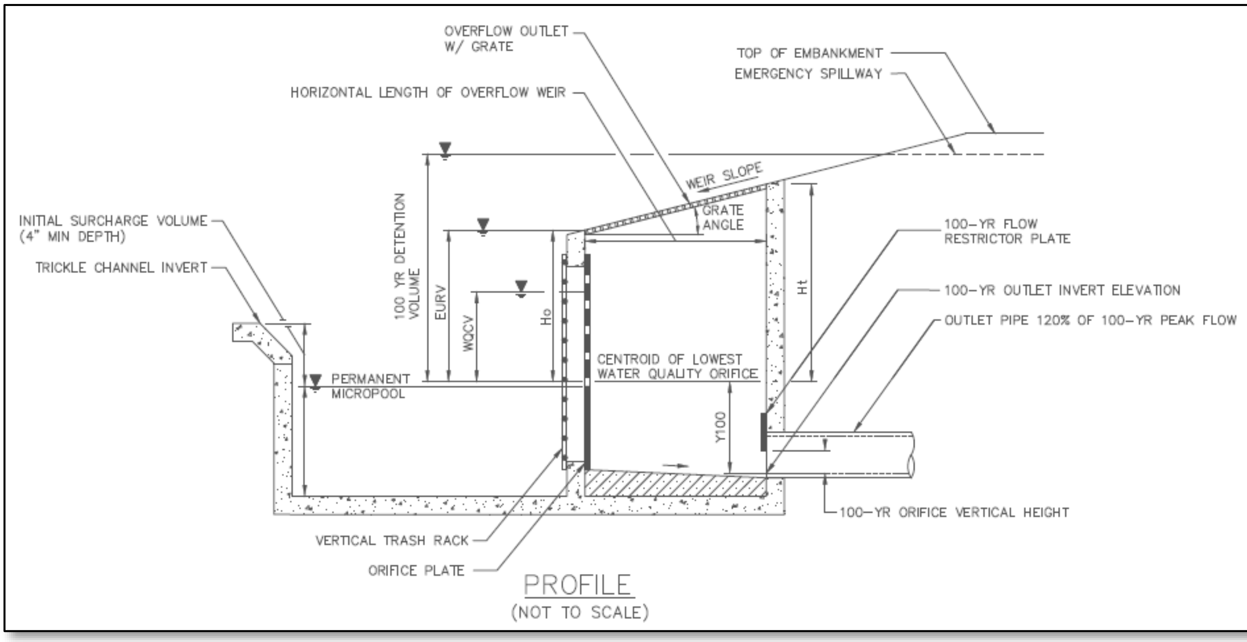


Figure 24. Section of detention basin outlet structure showing water quality plate and overflow outlet with grate.

The overflow outlet acts to regulate the flow of storm events larger than the EURV and/or WQCV but smaller than the 100-year event. When properly designed, these overflow outlets operate under weir flow and not orifice. The final metering of the design (e.g., 100-year) discharge from the detention basin should always be provided by an orifice plate covering the final discharge pipe inside the outlet box, and never from the overflow grate. There are two reasons for this strategy:

1. Orifice flow through the overflow indicates an excessive depth of ponding and a patently dangerous pinning/drowning hazard should a person slip or fall into the water, and
2. The overflow grate must be oversized to accommodate some level of clogging (UDFCD recommends a 50% clogging factor). Since the actual clogging condition cannot be assured, accurate metering cannot be achieved.

The hydraulic design of a detention basin requires knowledge of the discharge characteristics of the overflow outlet. If the outlet structure has a flat-topped (horizontal) overflow grate, then the classic weir and orifice equations can be used with area and perimeter reductions to account for the effects of the grate and assumed clogging thereof. If, however, the overflow grate is inclined in order to fit flush with the dam embankment, the discharge characteristics become much more complex and a different set of equations needs to be applied. Prior to this study, no standard guidance was available to calculate the stage-discharge curves necessary for the hydrualic design of extended detention basins.

4.1 Computational Fluid Dynamics (CFD) Modeling

In March 2012, UDFCD contracted with ARCADIS U.S., Inc. to apply computational fluid dynamics to estimate the stage-discharge relationship of various overflow outlets. The computational flow model was based on outlet boxes designed with a 3:1 H:V and 4:1 H:V sloped top, as shown in Figure 25.

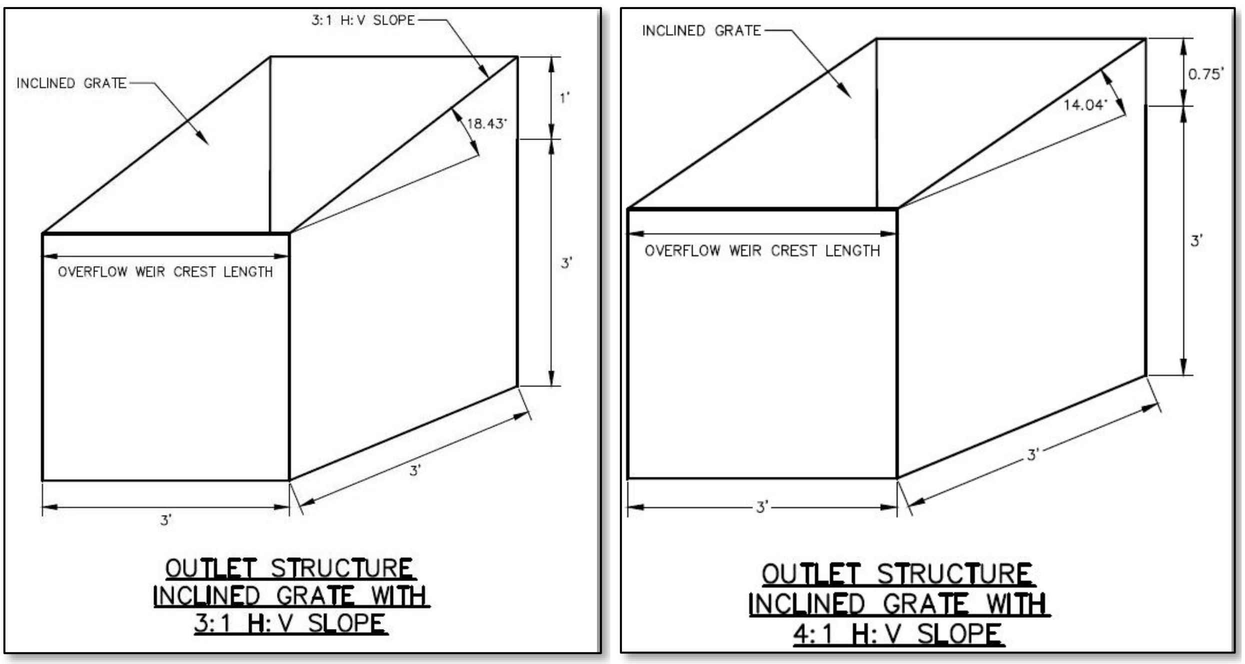


Figure 25. Basic model setup for 3:1 and 4:1 sloped overflow weirs in the CFD model.

The outlet box was modeled as being constructed into the dam embankment. The CFD model of the outlet box with 3:1 slide slopes was constructed as shown in Figure 26. The outlet box was a 3’ x 3’square with the top tapered to provide a good match with the slope of the embankment. The outlet box top was cut at an angle of 18.43 degrees for the 3:1 slope and 14.04 degrees for the 4:1 slope.

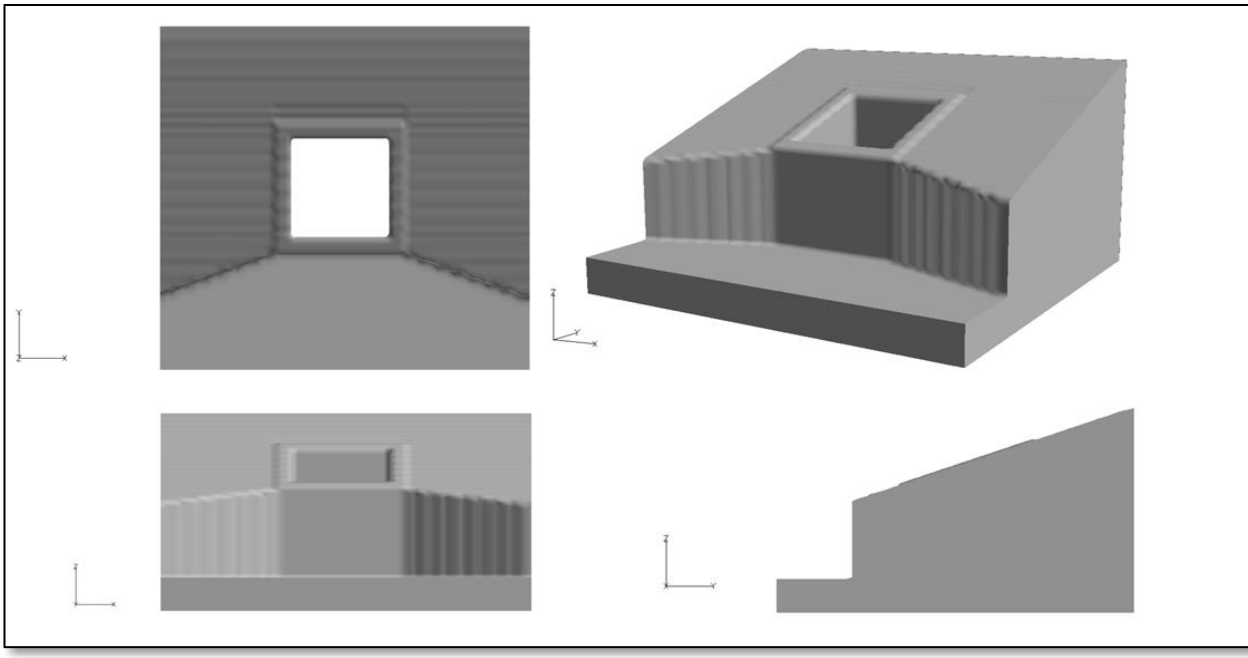


Figure 26. Outlet box model.

Based on the results provided by different grid sensitivity comparison tests, a mesh size of 1,200,000 control volumes (100 x 100 x 120) was selected to resolve the structure and to provide accurate results. In each of the calculations, water surface elevations were specified at each of the open boundaries, and flow left the domain through the bottom of the outlet box (continuative boundaries at the bottom). No-slip boundary conditions were specified at all solid walls, and the Renormalized Group (RNG) model was used for turbulence closure. A visualization of the model is shown in Figure 27.

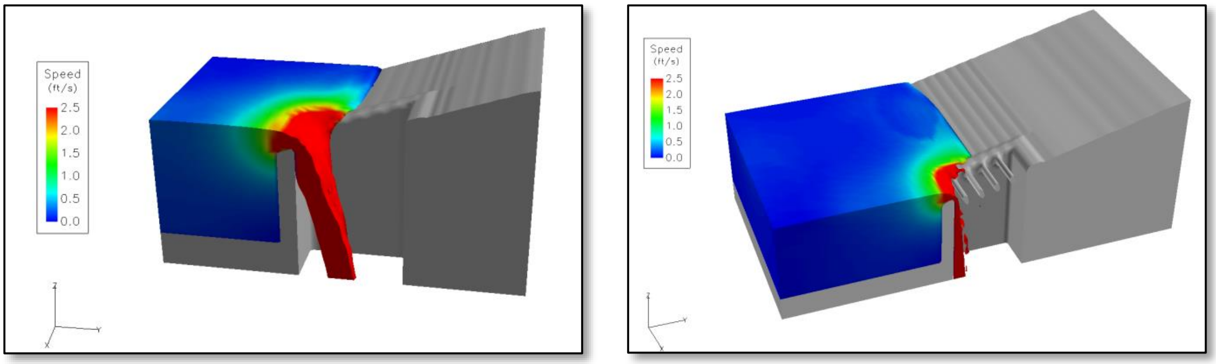
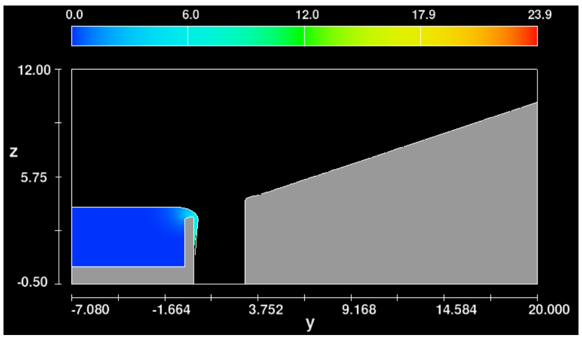
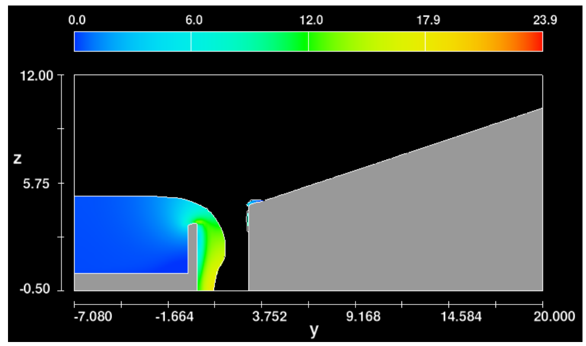


Figure 27. Water Surface Cutaway (colored by velocity, without grate and with grate)

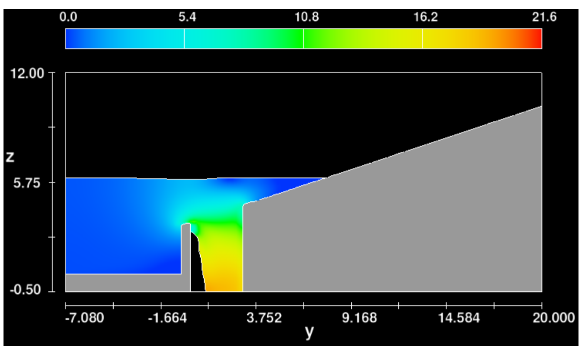
Nine calculations were carried out for each configuration using the FLOW-3D® computer program. The results were used to determine rating curves for the outlet box over the range of water levels from 1 to 5 ft above the lower front edge of the weir. The model results were calculated using the 1,200,000 control volume mesh, the most recent release version 10 of FLOW-3D, and the RNG turbulence model, with the simulation results shown in Figure 28.



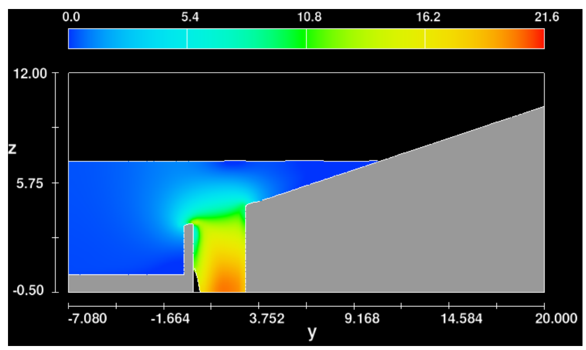
(a) 1' Head



(b) 2' Head



(c) 3' Head



(d) 4' Head

Figure 28. FLOW-3D® simulations with gradually-increasing water depths above the low front edge of the overflow weir.

The resulting rating curve for the outlet box with 3:1 top slope is shown in Figure 29 and the resulting rating curve for the outlet box with 4:1 top slope is shown in Figure 30. A side-by-side comparison is shown in Figure 31.

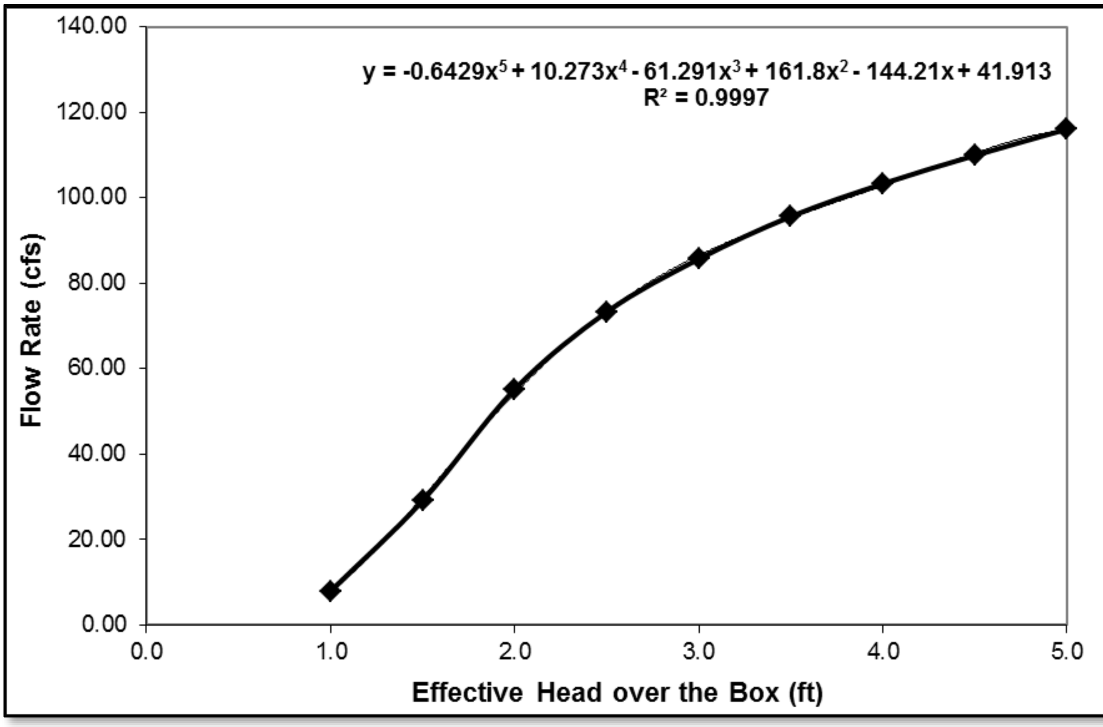


*Figure 29. FLOW-3D® resulting rating curve for the outlet box with **3:1** H:V top slope (5th degree polynomial regression curve fit).*

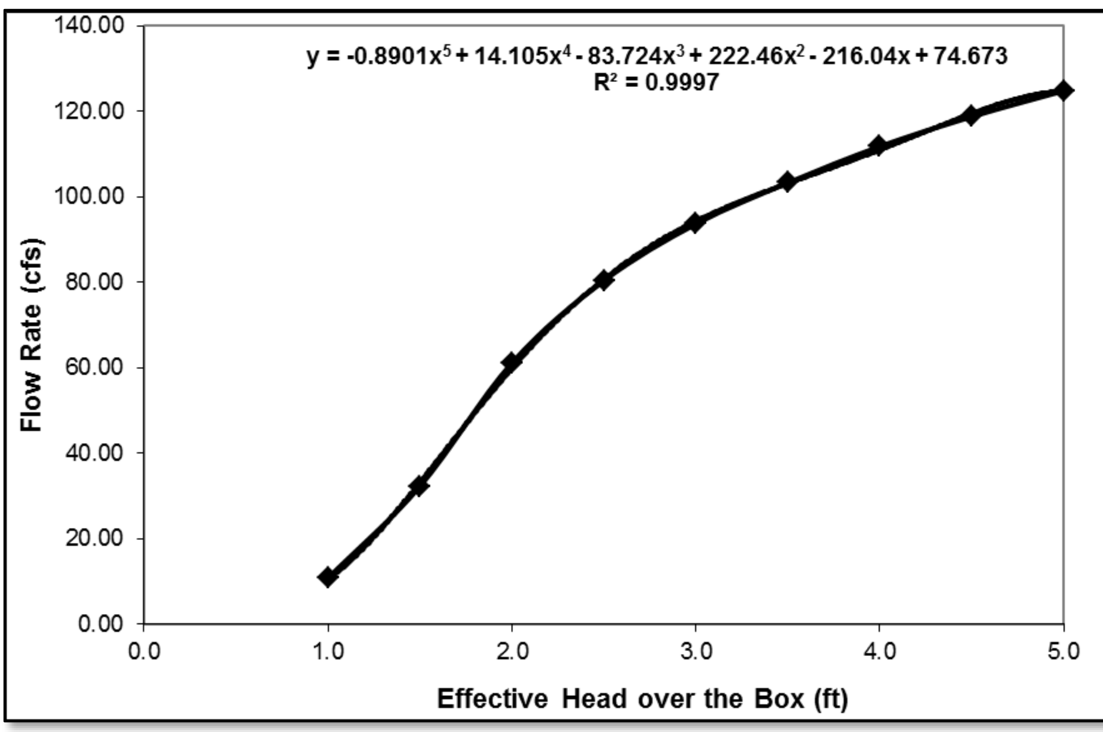


*Figure 30. FLOW-3D® resulting rating curve for the outlet box with **4:1** H:V top slope (5th degree polynomial regression curve fit).*

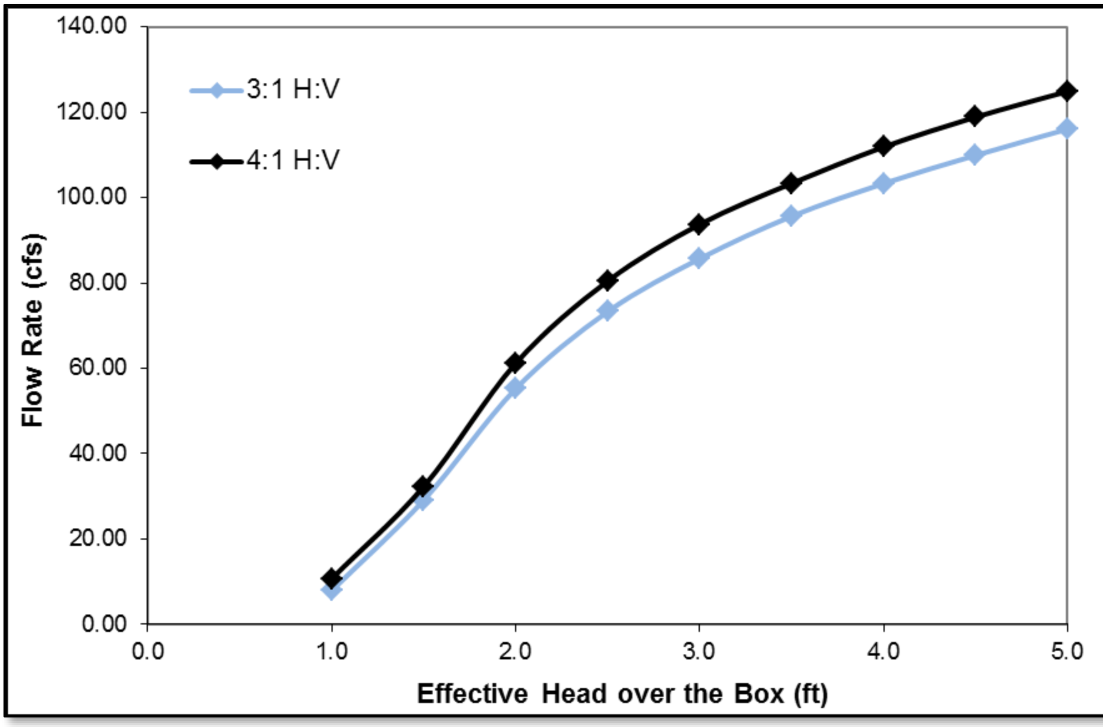


Figure 31. Side-by-side comparison of FLOW-3D® rating curves for the outlet box with 3:1 and 4:1 H:V top slopes.

Further analysis of the ARCADIS work indicated that the 5^{th} degree polynomials shown in Figures 29 and 30 are inadequate to use for design since 1) the Y-intercept must go through the origin (flow at zero depth must equal zero), and 2) instability issues with high degree polynomial regression equations such as these result in negative flow rates at a very shallow depth.

4.2 Guo's Analysis by Comparing to CDOT Type C and D Grated Inlet Study

The hydraulics of the inlet grates commonly used for overflow outlets were were studied in 2012 as part of a previous collaborative project between CDOT and UDFCD. CDOT Type C and D inlets were modeled at the CSU hydraulics laboratory, where a study was conducted to investigate the hydraulic performance of a 1/3-scaled model Type C grate with an inclined angle varied from zero to 30 degrees. The results of that study were reported by Guo et al. in the ASCE Journal of Irrigation and Drainage Engineering in April 2016 (Volume 140, Issue 6) and are summarized here. The hydraulic performance of a grate to a large degree depends on the ponding depth on the grate. When the water depth is too shallow to submerge the entire grate surface, the grate operates as a weir. When the grate area is completely submerged, the grate operates like an orifice. The transition from weir to orifice flow is called mixed flow (Guo et al. 2008). As shown in Figure 32, a grate is formed with I-beam bars. The net opening ratio for a grate is defined as the clear opening area for water to flow through the grate surface as:

$$n = (1 - C\log)\frac{LB - L_b B}{LB} = (1 - C\log)\frac{L - L_b}{L} \tag{10}$$

Where n = net area opening ratio, $Clog$ = clogging factor $0 \leq Clog \leq 1.0$ due to debris, L = grate length, B = grate width, and L_b = cumulative width of bars on grate. Eq. 10 indicates that the grate's area opening ratio for an orifice flow is equal to the length opening ratio for a weir flow. The selection of clogging factor depends on the highway condition, and a decayed clogging factor is recommended for multiple grates.

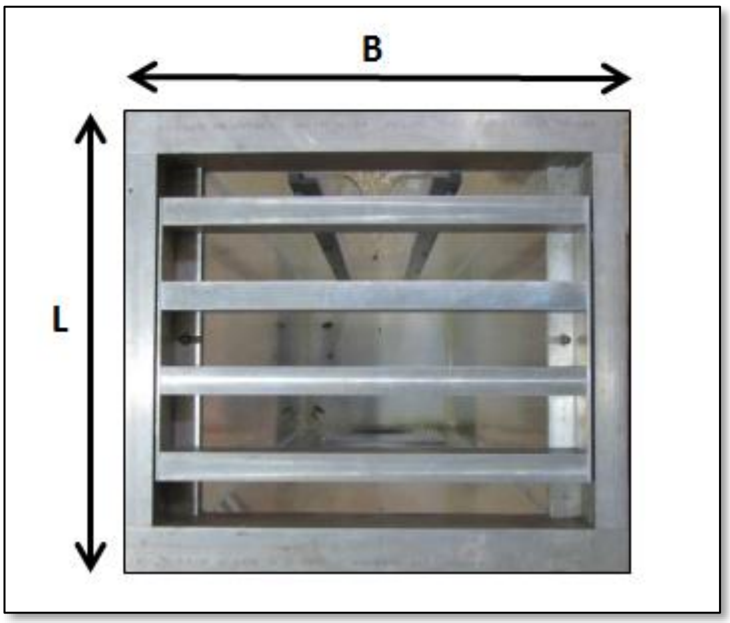


Figure 32. Grate dimensions.

The hydraulic capacity of a Type C grate is quantified according to its flow interception. The integral of flow interception is described as:

$$Q = nC_d \int \sqrt{2gh}\, dA \tag{11}$$

Where Q = flow rate, C_d = discharge coefficient, v = flow velocity, g = gravitational acceleration, dA= flow area, and h = headwater depth on dA. For a given water depth, the grate may operate like a weir or an orifice, whichever is less in flow interception. In this study, two sets of equations were derived to predict both the weir and the orifice flows. The discharge coefficients are respectively derived and then calibrated with the observed measurements.

4.21 Weir Flow Capacity

As illustrated in Figure 33, the inclined angle is formed by the gate length, L, and its height, H_b. The coordination system (h,x) is set to describe the flow condition in which h = water depth variable measured downward from the water surface, and x = distance variable measured upward from the base width. Under a shallow water depth, the grate's wetted perimeter may operate like a weir. Water overtops the three submerged sides into the inlet box, including two inclined sides and the lower base width.

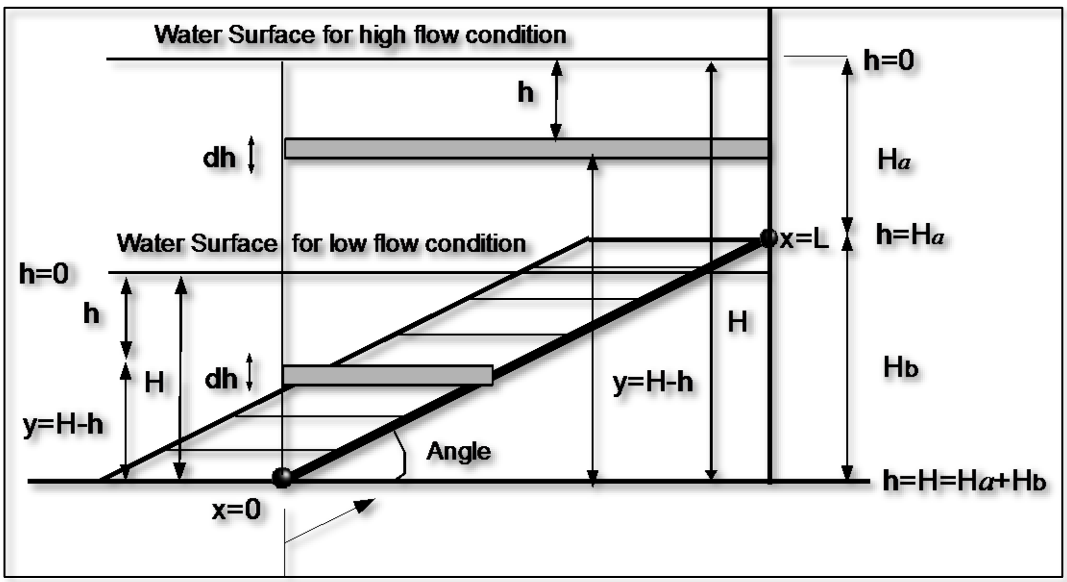


Figure 33. Weir Flow Overtopping Submerged Side along Grate.

Under a low flow condition as shown in Figure 33, only the lower portion of the grate is submerged. The infinitesimal flow area for a weir flow is derived as:

$$dA = (H - h) \cot \theta \, dh \qquad \text{for } H<H_b \tag{12}$$

$$y = H - h \tag{13}$$

Where θ = inclined angle, H= water depth, y = location of dA above the ground, and dh = infinitesimal thickness for flow area. The weir flow overtopping the wetted length along the grate's side is integrated from h=0 to h=H. Aided by Eq. 12, Eq. 14 yields:

$$Q_{WS} = \frac{4}{15} n \, C_d \sqrt{2g} \cot \theta \, H^{\frac{5}{2}} \qquad \text{for } H<H_b \tag{14}$$

Where Q_{ws}= side weir flow. Under a high water depth as illustrated in Figure 33, the integration limit is divided into two zones for mathematical convenience as:

$$H = H_b + H_a \tag{15}$$

Where H_a= surcharge depth above the top base of the grate. The infinitesimal areas for the weir flow in these two flow zones are respectively formulated as:

$$dA_1 = (H - h) \cot \theta \, dh \qquad 0 < h < H_a \text{ for Zone 1} \tag{16}$$

$$dA_2 = L \cos \theta \, dh \qquad H_a < h < H \text{ for Zone 2} \tag{17}$$

The weir flow overtopping the wetted length is integrated as:

$$Q_{WS} = nC_d \int_{h=0}^{h=H_a} \sqrt{2gh} L \cos \theta dh + nC_d \int_{h=H_a}^{h=H} \sqrt{2gh} (H - h) \cot \theta dh \tag{18}$$

Integrating Eq. 18 yields:

$$Q_{WS} = \frac{4}{15} nC_d \sqrt{2g} \cot \theta (H^{5/2} - H_a{}^{5/2}) \tag{19}$$

Re-arranging Eq. 19 yields:

$$Q_{WS} = \frac{4}{15} nC_d \sqrt{2g} L \, Cos\theta \, H^{\frac{3}{2}} [\frac{H^{\frac{5}{2}}}{H^{\frac{3}{2}} H_b} - \frac{(H - H_b)^{\frac{5}{2}}}{H^{\frac{3}{2}} H_b}] \qquad \text{for } H > H_b \tag{20}$$

At $H = H_b$, Eq. 20 agrees with Eq. 14. The total flow collected into the inlet box is the sum of the weir flows overtopping the two wetted sides along the grate and the lower base width of the grate. The weir flow, Q_{WB}, over the lower base is computed as:

$$Q_{WB} = \frac{2}{3} nC_d \sqrt{2g} \, B \, H^{\frac{3}{2}}$$

(21)

In which Q_{WB}= flow overtopping the low base width. The total weir flow is the sum as:

$$Q_W = 2Q_{WS} + Q_{WB} \tag{22}$$

In which Q_w = total interception for weir flow

4.22 Orifice Flow Capacity

When the grate surface area operates under orifice flow as illustrated in Figure 34, the integration of the orifice flow into the inlet box is separately conducted for the low and high water depth conditions.

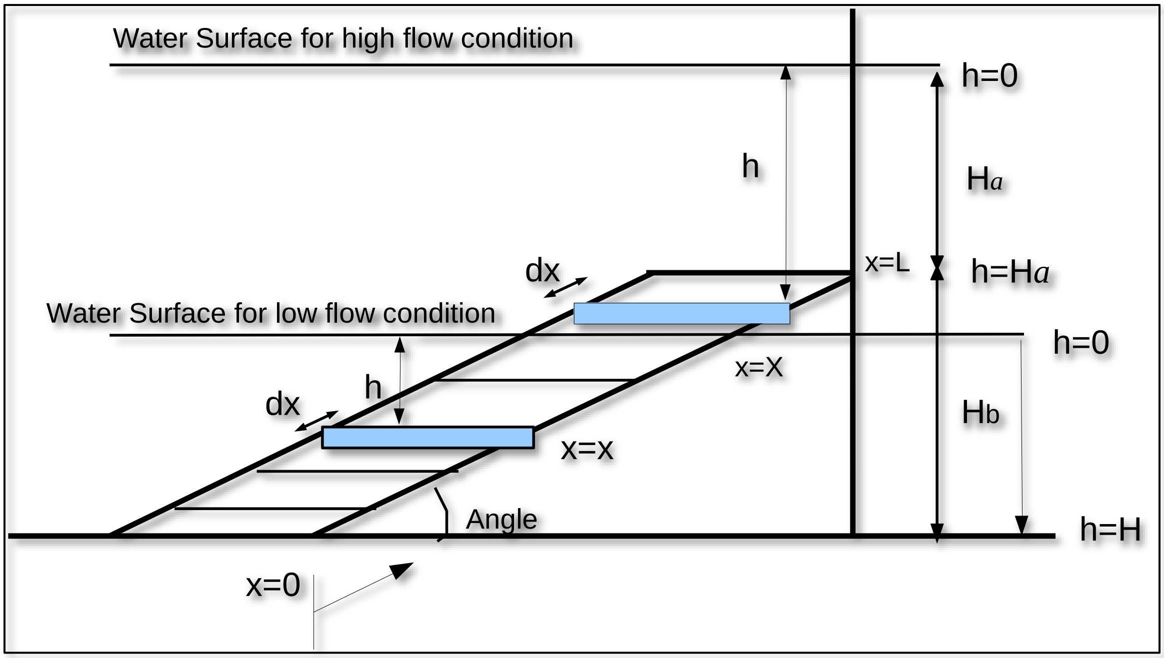


Figure 34. Orifice Flow through Submerged Area on Grate.

For $H<H_b$, the infinitesimal flow area for orifice flow in Figure 34 is defined as:

$$dA = n\ B\ cos\theta\ dx \tag{23}$$

The head water depth, h, can be related to the wetted length, x, along grate's side as:

$$h = (1 - \frac{x}{X})H \tag{24}$$

Where X= wetted length that varies between $0 \le X \le L$, x= integration variable that varies between $0 \le x \le X$. Under a low flow condition, $H \le H_b$, the orifice flow through the submerged surface area on the grate is integrated from x=0 to x=X as:

$$Q_o = \frac{2}{3} n C_d BH \cot\theta \sqrt{2gH} \qquad \text{for } H \le H_b \tag{25}$$

When $\theta=0$, Eq. 25 is reduced to a horizontal orifice as:

$$Q_o = \frac{2}{3} n C_d BL \sqrt{2gH} \qquad \text{for } H_b = 0 \text{ and } \theta = 0 \tag{26}$$

Under a high flow condition, the entire grate surface area is submerged. The headwater is related to the wetted length along the grate as:

$$h = H - \frac{x}{L}(H - H_a) = H - \frac{x}{L} H_b \tag{27}$$

For mathematical convenience, the flow depth is divided into two zones for numerical integration as: (1) above the top of the grate and (2) below the top of the grate. The orifice flow under a high water depth is integrated from x=0 to x=L as:

$$Q_o = \frac{2}{3} nC_d BL\cos\theta\sqrt{2gH}\left[\frac{H^{\frac{3}{2}}}{H_b\sqrt{H}} - \frac{(H-H_b)^{\frac{3}{2}}}{H_b\sqrt{H}}\right] \quad \text{for } H>H_b \tag{28}$$

At $H= H_b$, Eq. 28 agrees with Eq. 25. Comparing with the conventional approach, the orifice and weir coefficients can be related to the discharge coefficient as:

$$C_o = \frac{2}{3} C_d \tag{29}$$

$$C_w = \frac{4}{15} C_d \sqrt{2g} \tag{30}$$

In which C_o = orifice coefficient and C_w= weir coefficient. Using the orifice and weir coefficients, the governing equations for various flow conditions are summarized as follows. For $H \le H_b$, the orifice and weir flows are respectively estimated as:

$$Q_o = nC_o BHCot\theta\sqrt{2gH} \quad \text{for low orifice flow} \tag{31}$$

$$Q_w = 2n\, C_w Cot\theta\, H^{\frac{5}{2}} + nC_w BH^{\frac{3}{2}} \quad \text{for low weir flow} \tag{32}$$

For $H \ge H_b$, the orifice and weir flows are respectively estimated as:

$$Q_o = nC_o BLCos\theta\sqrt{2gH}\left[\frac{H^{\frac{3}{2}}}{H_b\sqrt{H}} - \frac{(H-H_b)^{\frac{3}{2}}}{H_b\sqrt{H}}\right] \quad \text{for high orifice flow} \tag{33}$$

$$Q_w = 2nC_w LCos\theta\, H^{\frac{3}{2}}\left[\frac{H^{\frac{5}{2}}}{H^{\frac{3}{2}}H_b} - \frac{(H-H_b)^{\frac{5}{2}}}{H^{\frac{3}{2}}H_b}\right] + nC_w BH^{\frac{3}{2}} \quad \text{for high weir flow} \tag{34}$$

For a given water depth, the interception capacity through an inclined grate is dictated by weir or orifice flows, whichever is less as:

$$Q_c = \min(Q_w, Q_o) \text{ for a given water depth} \tag{35}$$

In which Q_c = flow interception through grate. On the contrary, for a given design flow, the required headwater depth, H, acting on an inclined grate is determined as:

$$H = \max(H_w, H_o) \text{ for a given design flow} \tag{36}$$

Where H_w = headwater for weir flow, H_o= headwater for orifice flow, and H= design headwater. The equations developed by Guo et al. are summarized in Table 2.

Table 2. Summary of equations by Guo et al. for calculating discharge through CDOT Type C and D median inlets.

Flow Type	Flow Overtopping Two Sides of Inclined Grate	Flow overtopping the Lower Base Width	Condition
Orifice	$Q_o = \frac{2}{3} nC_d BHCot\theta\sqrt{2gH} = \frac{2}{3} nC_d BXCos\,\theta\sqrt{2gH}$ Subject to: $X = \frac{H}{Sin\theta} < L$		$H<H_b$ Un-submerged
Weir	$Q_{WS} = \frac{4}{15} nC_d \sqrt{2g} Cot\theta\, H^{\frac{5}{2}} = \frac{4}{15} nC_d XCos\theta\sqrt{2g} H^{\frac{3}{2}}$ subject to: $X = \frac{H}{Sin\theta} < L$ $Q_W = 2Q_{WS} + Q_{WB}$	$Q_{WB} = \frac{2}{3} nC_d \sqrt{2g} BH^{3/2}$	$H<H_b$ Un-submerged
Orifice	$Q_o = \frac{2}{3} nC_d BLCos\theta\sqrt{2gH}[\frac{H^{\frac{3}{2}}}{H_b\sqrt{H}} - \frac{(H-H_b)^{\frac{3}{2}}}{H_b\sqrt{H}}]$ In case of θ=0 and Hb=0, then $Q_o = \frac{2}{3} nC_d BL\sqrt{2gH}\ if\ \theta = 0$		$H \geq H_b$ Submerged
Weir	$Q_{WS} = \frac{4}{15} nC_d \sqrt{2g} LCos\theta\, H^{\frac{3}{2}}[\frac{H^{\frac{5}{2}}}{H^{\frac{3}{2}} H_b} - \frac{(H-H_b)^{\frac{5}{2}}}{H^{\frac{3}{2}} H_b}]$ In case of θ=0 and Hb=0, then $Q_{WS} = \frac{2}{3} nC_d L\sqrt{2g} H^{\frac{3}{2}}$ $Q_W = 2Q_{WS} + Q_{WB}$	$Q_{WB} = \frac{2}{3} nC_d \sqrt{2g} BH^{3/2}$	$H \geq H_b$ Submerged

4.3 Physical Modeling at the USBR Hydraulics Lab

In December 2012, UDFCD contracted with the U.S. Bureau of Reclamation (USBR) Hydraulics Laboratory in Lakewood, Colorado to perform physical modeling of the overflow weir and grate in different configurations. The results of that study were published by Heiner in 2014 and are summarized here. The purpose of this effort was to verify that the equations developed by Guo (see Section 4.2).

4.31 Model Setup

A model box approximately 25-ft wide, 45-ft long and 4-ft deep was configured to simulate an extended detention basin. One end of the box contained a 12-inch diameter inlet pipe and a 6-inch thick rock baffle to evenly distribute the flow entering the model. The opposite end of the box contained several configurations of the overflow outlet structure with and without grating, as shown in Figure 35.

The outlet structure was modeled at a geometric scale of 1:3, which means model dimensions are one-third of the prototype dimensions. Since hydraulic performance for open channel flow depends primarily on gravitational and inertial forces, Froude law scaling was used to establish a relationship between the model and prototype. Froude law scaling causes the ratio of gravitational to inertial forces to be equal in the model and prototype; stated in another way, the Froude numbers of the model and prototype are kept equal to one another.

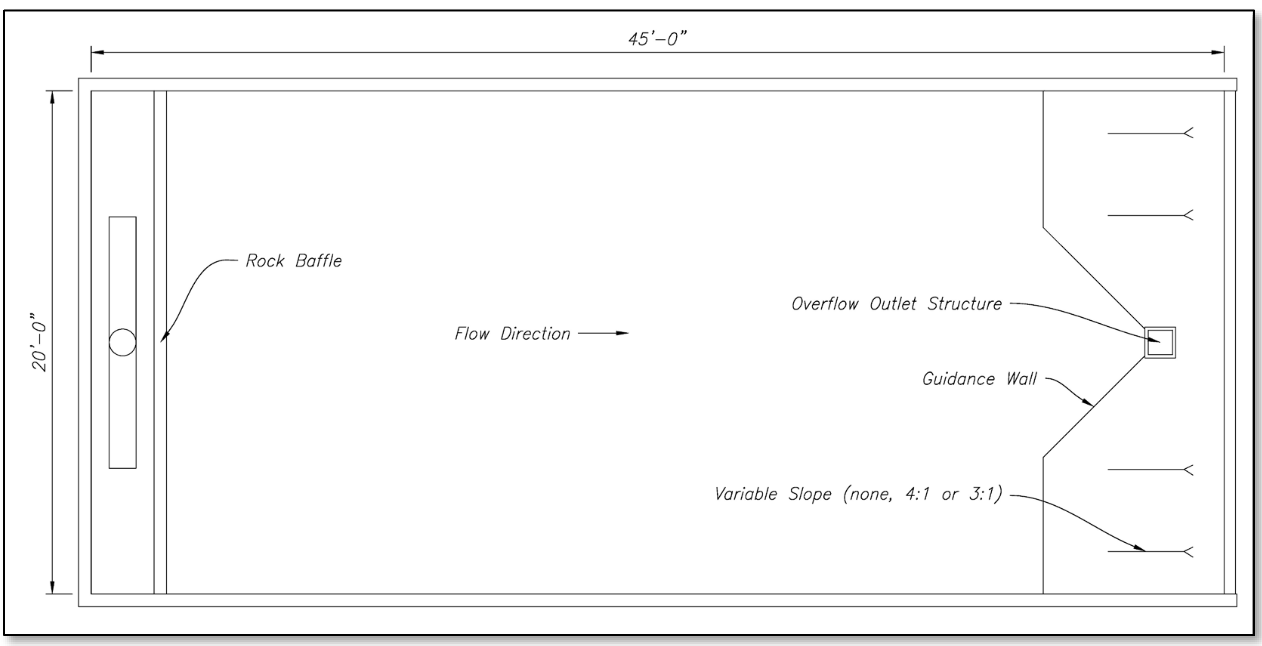


Figure 35. Physical model layout of an extended detention basin (EDB, model scale)

Froude law similitude produces the following relationships between model (m) and prototype (p), as:

Length Ratio: $L_r = L_m/L_p = 1:3$ (37)

$$\text{Velocity Ratio: } V_r = V_m/V_p = L_r^{1/2} = 1:1.732 \quad (38)$$

$$\text{Discharge Ratio: } Q_r = Q_m/Q_p = L_r^{5/2} = 1:15.59 \quad (39)$$

Three different grates were tested, including a Standard CDOT Type C grate (Figure 36a), a CDOT close-mesh grate (Figure 36b), and a “No Grate” scenario (Figure 36c) where only the grate frame, which was a rectangular opening approximately 41 inches by 35 inches, was tested. Each grate configuration was tested at slopes of 3:1 (H:V), 4:1, and 1:0 horizontal (no slope). The two sloped configurations were modeled as though the outlet structure was constructed into the dam embankment as this is the typical reason for the sloped top. The flat-topped outlet was modeled as a free standing structure as this configuration is common in the field.

(a) CDOT Type C grate *(b) Type C close-mesh grate* *(c) No grate*

Figure 36. Types of grates tested in USBR hydraulics lab 1/3-scale model.

Table 3 contains a summary of the test configurations modeled and indicates where surrounding topography was set at the same slope as the overflow outlet structure and grate (Figure 37), as opposed to a no slope with no topography configuration (Figures 38 and 39). Most test configurations modeled the flow passing through the overflow outlet portion of the outlet works. One final configuration was modeled that tested no slope with no topography and included a complete outlet structure with water quality orifice plate and 100-yr orifice (Figure 40) restricting flow downstream of the overflow outlet. The water quality orifice plate was modeled as both the standard configuration with a series of orifice holes and as an alternative elliptical weir (Figure 41).

Table 3 - Summary of test configurations that were modeled

Slope	Grate	Surrounding Topography
3:1 (H:V)	Standard CDOT Type C	YES
3:1 (H:V)	CDOT Close Mesh	YES
3:1 (H:V)	None	YES
4:1 (H:V)	Standard CDOT Type C	YES
4:1 (H:V)	CDOT Close Mesh	YES
4:1 (H:V)	None	YES
Horizontal	Standard CDOT Type C	NO
Horizontal	CDOT Close Mesh	NO
Horizontal	None	NO

Each model configuration was tested by completing the following steps:

1. Establish a specific flow rate measured by a calibrated Venturi meter accurate to ±0.25 percent (USBR 1989) into the model box.
2. Allow the flow to stabilize for the necessary amount of time so that no change in water surface in the EDB is noticed for at least 5 minutes.
3. Obtain the water surface elevation (stage) above the lower edge of the inlet using both a calibrated laboratory ultrasonic sensor and a point gauge (redundant measurements for consistency).
4. Record both the stage and flow.
5. Repeat steps 1-4 to create a complete rating curve that identifies any transitions between weir and orifice flow.

Inflow and stage were recorded and plotted to generate stage-discharge relationships for each configuration. Collected data were then compared to the provided rating equations by Guo in Section 4.2.

Figure 37. Model setup for 3:1 (H:V) and 4:1 (H:V) grate slope testing.

Figure 38. Model setup for horizontal grate testing.

Figure 39. Close-up of horizontal grate testing.

Figure 40. 100-year restrictor plate covering the final discharge pipe inside the outlet structure.

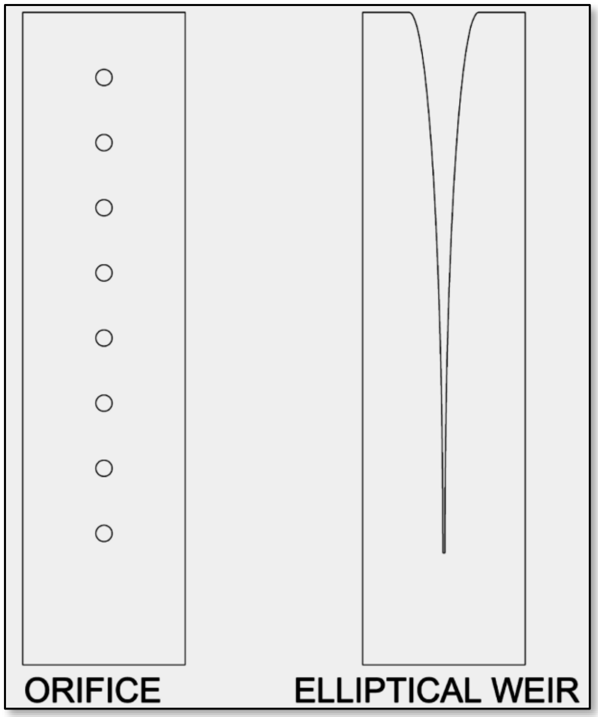


Figure 41. Water quality orifice plate configurations tested in the complete EDB model.

4.32 Model Results

Figure 42 shows data collected at the 1:0 (H:V) (no slope, aka horizontal) configuration for each of the three tested grates. Figure 43 shows data collected at the 4:1 (H:V) slope configuration for each of the three tested grates. Figure 44 shows data collected at the 3:1 (H:V) slope configuration for each of the three tested grates. Each figure plots stage above the lowest edge of the overflow outlet structure in ft on the x-axis and discharge through the overflow outlet in cfs on the y-axis.

Figure 45 provides data collected on the complete EDB with micropool, water quality orifice, horizontal overflow outlet, and 100-year controlling orifice. This plot also shows stage (ft) above the lowest edge of the overflow outlet structure on the x-axis and discharge through the overflow outlet in cfs on the y-axis. All three grates were tested with a series of orifice holes in the water quality plate. One test was conducted with the orifice holes being replaced with an elliptical weir which releases a significantly larger discharge for a given head.

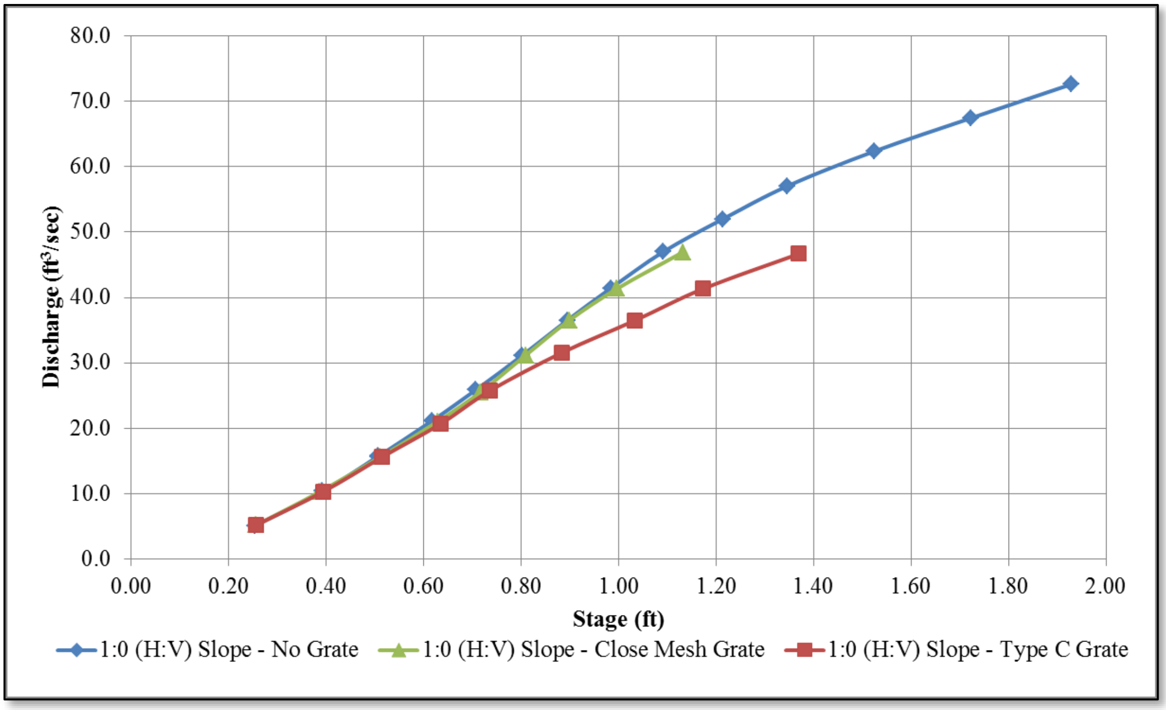


Figure 42. Data collected in the 1:0 (H:V) slope configuration for each grate (prototype dimensions).

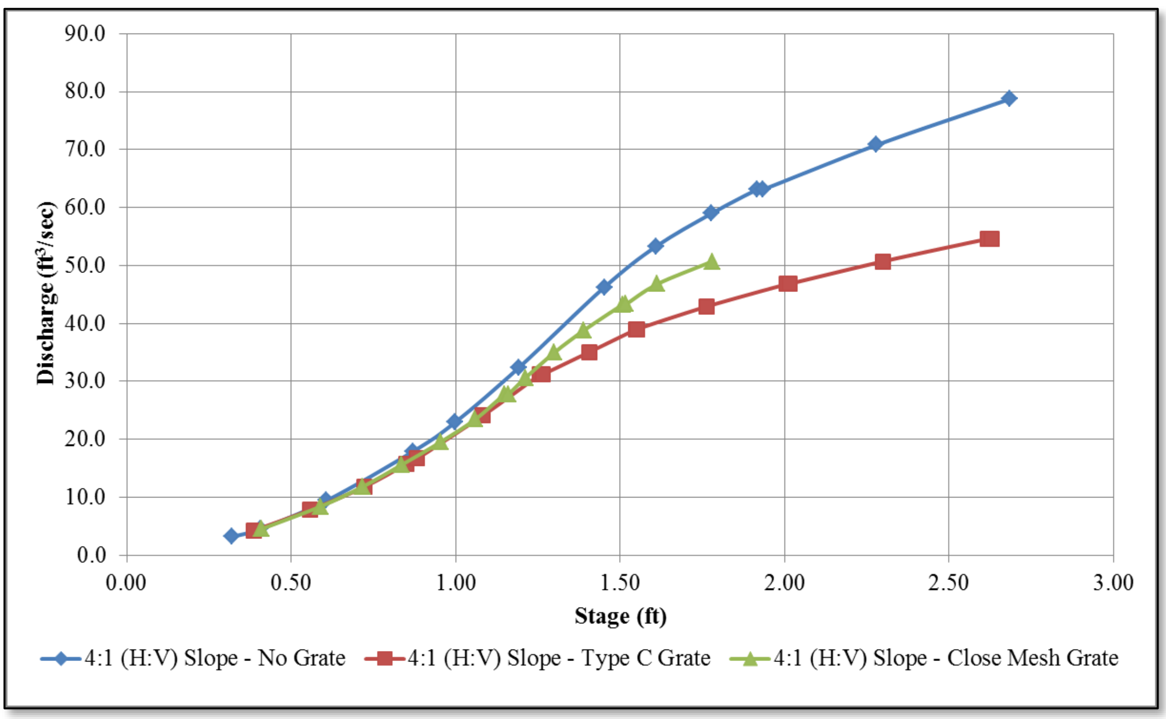


Figure 43. Data collected in the 4:1 (H:V) slope configuration for each grate (prototype dimensions).

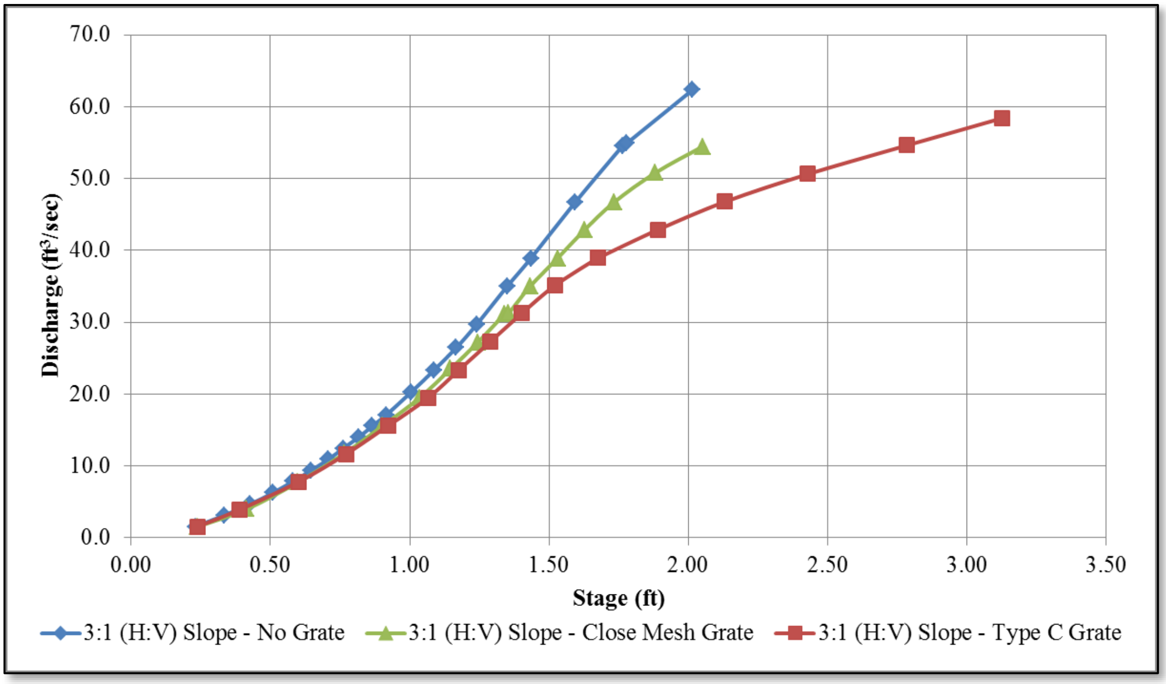


Figure 44. Data collected in the 3:1 (H:V) slope configuration for each grate (prototype dimensions).

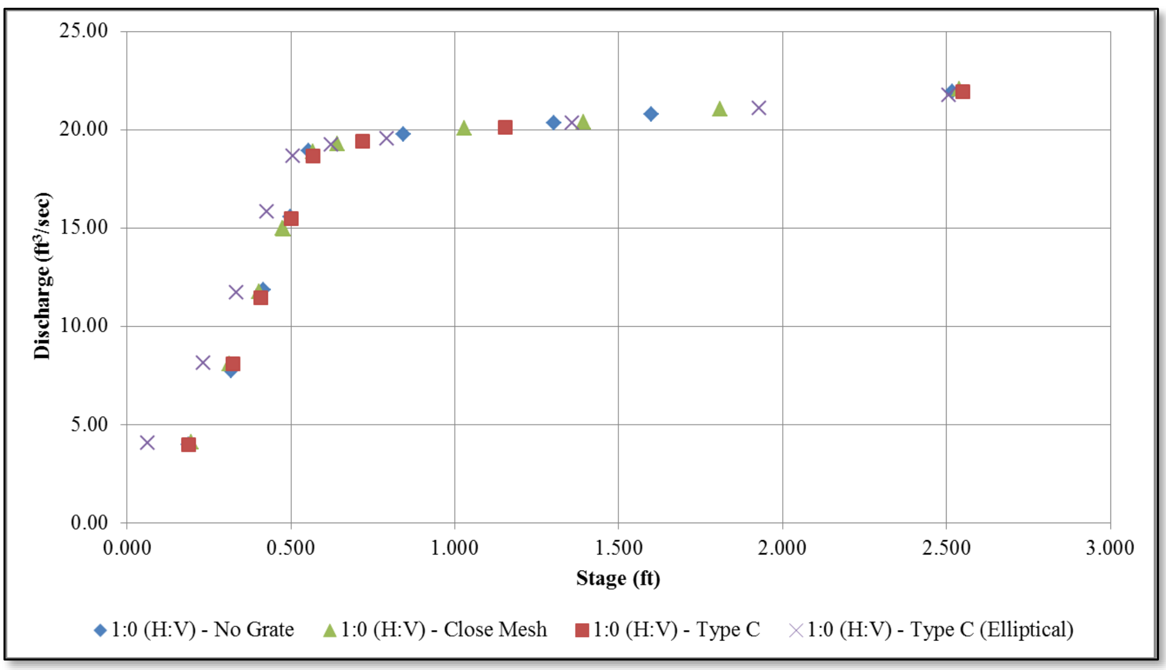


Figure 45. Data collected on the complete EDB with micropool and 1:0 (H:V) slope overflow outlet structure. Water quality plates and the 100-year controlling orifice were installed for each configuration tested.

Each scenario was compared to equations by Guo provided in Table 2 to determine if the equations generated rating curves consistent with the physical model. The shape of the stage-discharge curve observed in the model makes it apparent that flow control varies from weir flow at low heads to transitional (mixed flow) at intermediate heads, and finally orifice flow at high heads. Approximate bounds of these zones are illustrated in Figure 46. Zones will change slightly depending on the geometry and configuration of the outlet structure and overflow weir.

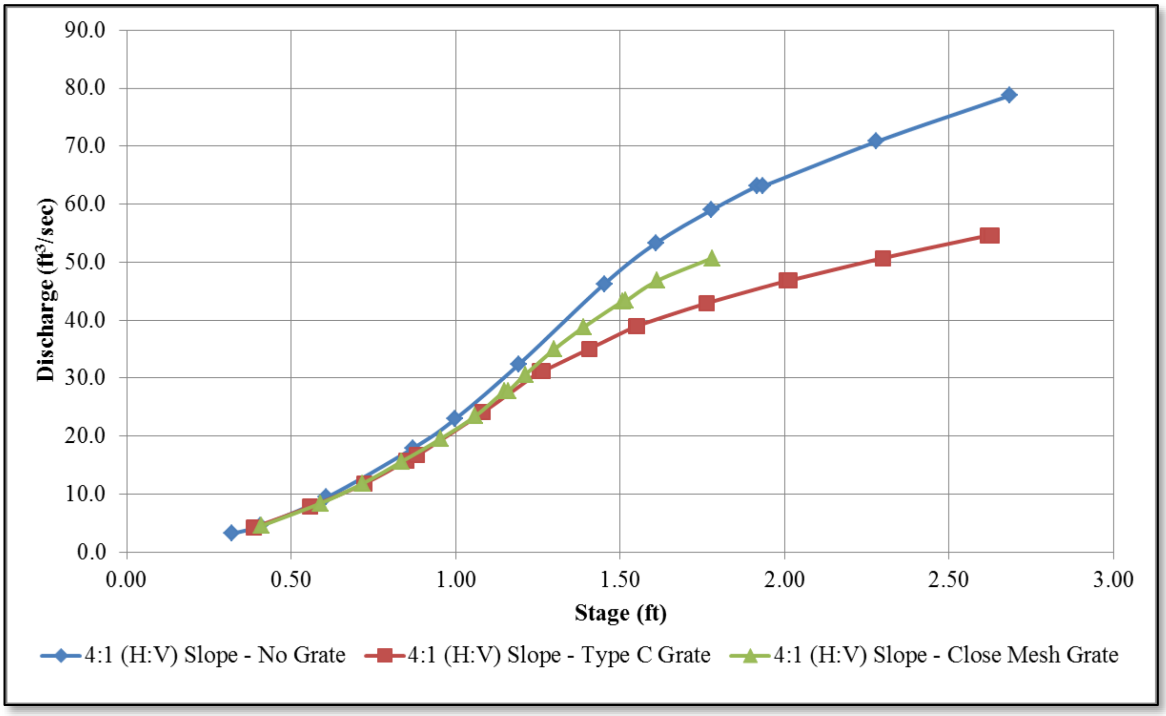


Figure 46. Approximate boundary zones for weir flow, mixed flow and orifice flow.

When flows were in the mixed flow zone they became unstable and the stage in the EDB would fluctuate significantly with a constant inflow. Figure 47 shows this phenomenon, which was present at all configurations. Data was collected for each configuration until the stage oscillations were noticed. As can be seen in Figures 42 through 44, oscillations occurred at different head and discharge for each configuration.

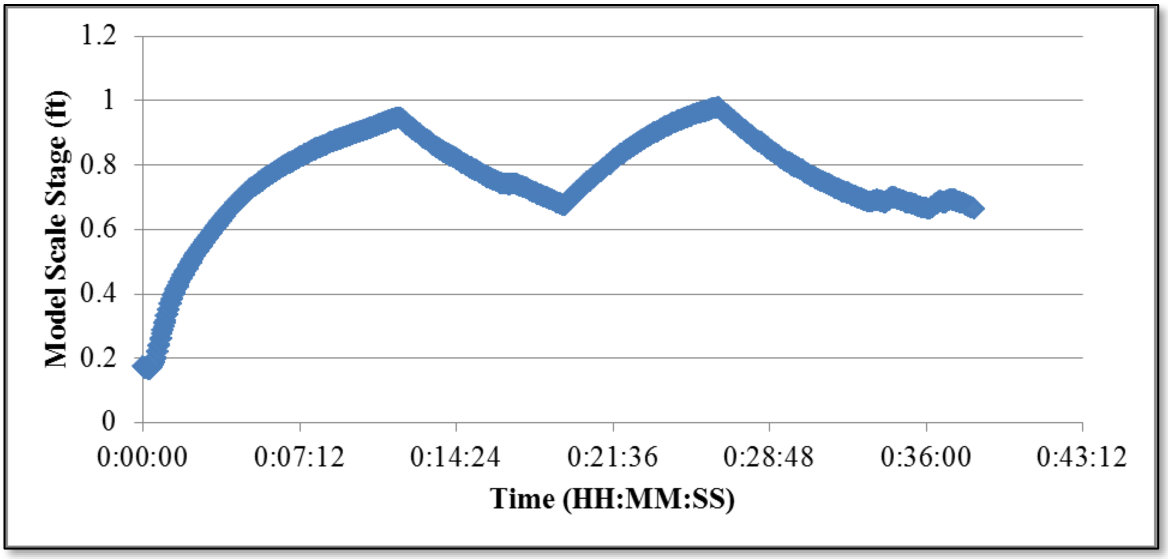


Figure 47. Sample flow oscillations that occurred when flows entered mixed zone for the 4:1 slope with standard CDOT Type C grate.

The USBR analyzed the data to determine if a single new equation or set of equations of consistent form could be generated that would accurately describe the flow through the overflow outlet works for all structure configurations. The data was plotted in TableCurve 2D and TableCurve 3D, utilizing different dependent and independent variables. No single relationship was found that accurately described the overflow outlet discharge for all configurations tested. It was determined that it would be difficult if not impossible to develop a new equation that would accurately describe the flow through the overflow outlet in all zones (weir, mixed, and orifice) for all slopes, especially with the limited data that were collected during this modeling effort.

Calculating the discharge through the overflow outlet in all three zones (weir, mixed, and orifice) was determined to be unnecessary from a practical perspective. When installed, the outlet structure typically employs a 100-yr orifice that restricts the flow downstream of the overflow outlet and prevents the overflow outlet from ever functioning as the flow control in the transitional or orifice mode. It was therefore determined that modeling a complete EDB would adequately verify how the overflow outlet and the 100-yr orifice combine to control the flow. As shown in Figure 45, the complete model of the EDB confirmed that flow would be restricted by the 100-yr orifice prior to the overflow outlet entering the mixed flow or orifice flow zones; the overflow outlet is in the weir flow zone for the entire range in which it controls the flow.

The 100-yr orifice installed downstream of the overflow outlet performs several valuable functions for the EDB:

1. First and foremost, this improves the safety of the outlet structure by minimizing the possibility of a drowning by becoming pinned to the grate as the result of the suction force accompanying greater ponding depths and orifice flow. This pinning phenomenon was reported by Guo and Jones in the ASCE Journal of Irrigation and Drainage Engineering in February 2010.
2. The flow rate from the EDB must be limited to the 100-yr flow so that open channels or piping systems downstream of the EDB outlet are not overwhelmed.
3. The 100-yr orifice makes calculating the flow from the overflow outlet less complicated because the flow would remain primarily in the weir flow zone. Discharge calculations from the EDB would transfer to using the 100-yr orifice before utilizing the overflow outlet as an orifice.
4. The 100-yr orifice would prevent the overflow outlet from reaching an unstable oscillating water surface with associated unstable outflows that could not be accurately calculated from the EDB stage.

The limiting action of the 100-year orifice on the overflow outlet is shown as the blue line in in Figure 48.

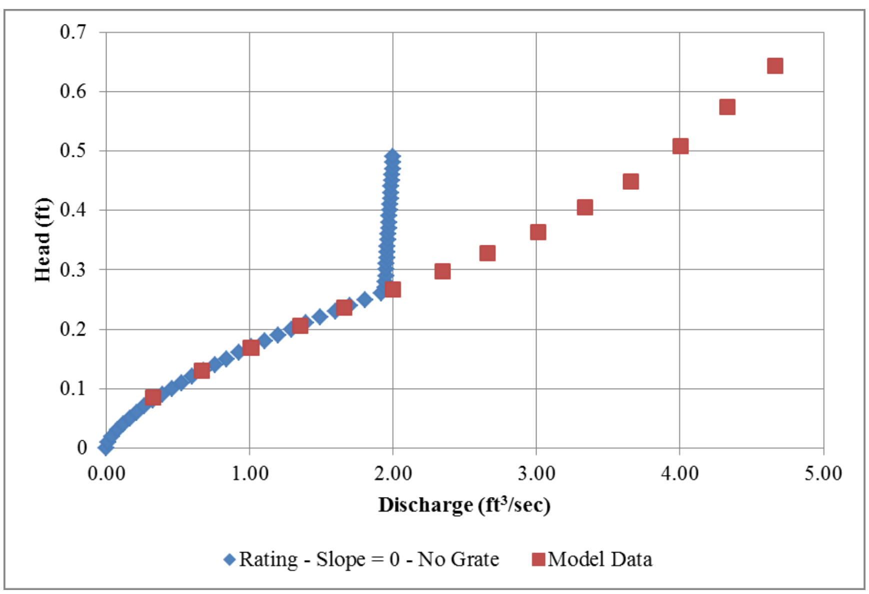


Figure 48. Final calculated stage discharge plot showing the flow from the 100-year orifice acting with the overflow in blue and the overflow outlet acting alone in red, using data for a 1:0 (H:V) slope with no grate.

Flows entering the outlet structure become very turbulent between the overflow outlet and the 100-yr orifice. Under these circumstances, it was necessary to determine whether using a standard orifice discharge coefficient of 0.61 would yield accurate discharge calculations from the 100-yr orifice. Data from the physical model were used to determine that the coefficient in the model was 0.60. When calculating flow from the 100-yr orifice, head relative to the center of the orifice was used.

When calculating flow through an overflow outlet, a clogging factor is recommended by UDFCD which is a reduction factor to represent typical clogging. To this clogging factor, an additional factor is added to represent the reduction in area caused by the grates. For the USBR study, it was determined that it would be more appropriate to use a discharge coefficient to account for the reduction in flow caused by the grate and have a separate clogging factor to account for debris clogging. By creating custom discharge coefficients from the physical model data for each grate and slope, the physical model data were able to be matched to the weir equations provided by Guo in Table 2. Discharge coefficients for each slope and grate are shown in Table 4. These discharge coefficients are used in the equations presented in Table 5 (adapted from Guo) to calculate the flow from the overflow outlet structure; variable locations are shown in Figure 49.

Table 4. Discharge coefficients for each slope and grate.

100-yr Orifice Coefficient	
0.60	100-yr orifice
Overflow Outlet Coefficient, C_d	
0.64	1:0 (H:V) Slope - No Grate
0.62	1:0 (H:V) Slope - Close Mesh
0.60	1:0 (H:V) Slope - Type C
0.68	4:1 (H:V) Slope - No Grate
0.63	4:1 (H:V) Slope - Close Mesh
0.62	4:1 (H:V) Slope - Type C
0.68	3:1 (H:V) Slope - No Grate
0.60	3:1 (H:V) Slope - Close Mesh
0.58	3:1 (H:V) Slope - Type C

Table 5. Equations to determine discharge from the EDB overflow outlet, adapted from Guo et al.

Flow Type	Equation
100-yr orifice	$Q_O = C_o A_o \sqrt{2gH}$
Flat Weir	$Q_{Flat} = \frac{2}{3} n C_d (2B + 2L)\sqrt{2g} H^{\frac{3}{2}}$
Sloped Un-Submerged Weir ($H < H_b$)	$Q_{WS} = \frac{4}{15} n C_d \sqrt{2g} \cot(\theta)\, H^{\frac{5}{2}}$ $Q_{WB} = \frac{2}{3} n C_d \sqrt{2g} B H^{\frac{3}{2}}$ $\boldsymbol{Q_W = 2Q_{WS} + Q_{WB}}$
Sloped Submerged Weir ($H \geq H_b$)	$Q_{WS} = \frac{4}{15} n C_d \sqrt{2g} L \cos(\theta) \left[\frac{H^{\frac{5}{2}} - (H - H_b)^{\frac{5}{2}}}{H_b} \right]$ $Q_{WB} = \frac{2}{3} n C_d \sqrt{2g} B H^{\frac{3}{2}}$ $\boldsymbol{Q_W = 2Q_{WS} + Q_{WB}}$

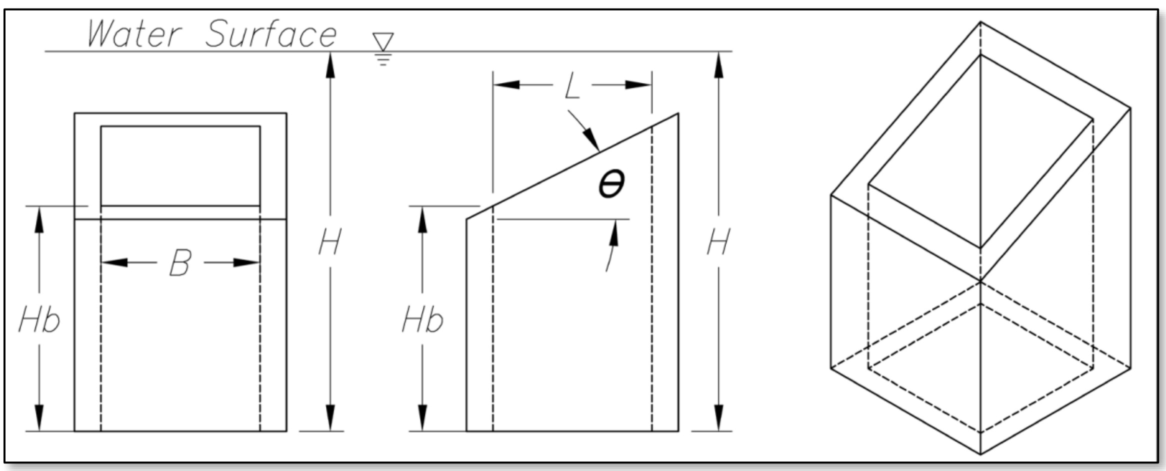


Figure 49. Locations of variables used in Table 5 equations.

The USBR used Guo's weir-flow equations to calculate flow over only three sides of the overflow outlet, based on the assumption that flow over the top edge is considered negligible because the head acting on this section is limited by the overland flow across the ground surface.

For the 1:0 (H:V) horizontal case, this assumption is not realistic because flow can enter equally from all four sides. This is the result of these outlets typically not being installed in the bank of the EDB and do not have surrounding topography. When modeling the complete EDB, two different water quality orifice options were tested: a series of orifice holes and an elliptical weir configuration. The elliptical weir configuration is desirable from a debris standpoint because the orifice holes have a tendency to clog when floating debris enters the EDB, however, the elliptical slot will be prone to clogging if the width of the slot is insufficient to pass small debris (a minimum slot of 3/8-inch (0.375-inch) is recommended) . Figure 45 shows at higher depths of ponding, the elliptical weir will release more flow from the EDB than the orifice configuration, but that at lower depths of ponding, the opposite condition is true. In theory, this should result in better water quality from the elliptical slot weir as compared to the orifice plate since the discharge curve more closely follows the gradation-based settling velocity curve as defined by Stokes Law. Significant water quality testing would be necessary to demonstrate this theory however, and that was not included in the scope of this project.

5. DEVELOPMENT OF NEW DESIGN SOFTWARE

Directly as the result of the work completed by ARCADIS, CSU, and the USBR, major improvements were made to the UD-Detention and UD-FSD design workbooks. In their new state, these freeware design workbooks are powerful software tools for CDOT and its consultants to apply to the hydrologic and hydraulic design of extended detention basins, bioretention BMPs, sand filtration BMPs, constructed treatment wetlands, and retention ponds. These workbooks apply regression equations to user-inputted watershed data to size a suite of inflow hydrographs representing common probabilistic recurrence intervals. These inflow hydrographs are then routed through a modeled facility using the Modified Puls reservoir routing method, allowing the user to experiment with different control volumes and outlet configurations in order to achieve the desired drain times and target maximum discharge rates.

5.1 Mathematical Model of a Detention Basin

In order to apply the Modified Puls reservoir routing method to a detention facility, two things are essential:

1. A stage-storage or stage-area table or equation, and

2. A stage-discharge table or equation.

For final design, the stage-area data set is readily available from the grading plans, but in the planning or conceptual design stage, the engineer must make some basic assumptions regarding the volume and shape of the basin. To this end, a set of equations and methods to model proposed detention basins, with stage-storage relationships that produce realistic draining characteristics, were developed. In addition to the UD-Detention and UD-FSD design workbooks, these methods can be used in other reservoir routing programs such as HEC-HMS and HEC-1; TR-20/TR-55; HEC-RAS unsteady flow; SWMM (including PC-SWMM and XP-SWMM); ICPR, PondPack, HydroCAD, and Hydraflow. These methods are appropriate for modeling proposed flood and/or stormwater quality detention basins in watershed planning studies. The mathematical model of a detention basin includes the initial surcharge volume, the basin floor volume, and the main basin volume. The sum of all these is the total basin volume. The initial surcharge volume is represented as:

$$ISV = 0.003WQCV A_{ISV} = \frac{ISV}{ISD} \tag{40}$$

$$L_{ISV} = \sqrt{A_{ISV}} \tag{41}$$

$$W_{ISV} = \sqrt{A_{ISV}} \tag{42}$$

Where ISV is the initial surcharge volume (ft^3), A_{ISV} is ISV surface area (ft^2), ISD is the initial surcharge depth (ft, typically 0.33 to 0.50), and L_{ISV} and W_{ISV} are the length and width of the ISV (ft). The basin floor volume is expressed as:

$$L_{floor} = L_{ISV} + \frac{H_{floor}}{S_{TC}} + H_{floor}(S_{main}) \tag{43}$$

$$W_{floor} = W_{ISV} + \frac{H_{floor}}{R_{L:W}(S_{TC})} \tag{44}$$

$$A_{floor} = L_{floor}(W_{floor}) \tag{45}$$

$$V_{floor} = \frac{H_{floor}}{3}\left(A_{ISV} + A_{floor} + \sqrt{A_{ISV}(A_{floor})}\right) \tag{46}$$

Where L_{floor} and W_{floor} (ft) are the length and width of the basin floor section at the point where the top of the basin floor section meets the toe of the basin main section, H_{floor} is the depth of the basin floor section (ft), S_{TC} is the trickle channel slope (ft/ft), S_{main} is the side slope of the basin main section (H:V; e.g., 4 if the H:V ratio is 4:1), $R_{L:W}$ is the basin length-to-width ratio (e.g., 2 if the basin length is twice the basin width), A_{floor} is top area of the basin floor section (ft^2), and V_{floor} is volume of the basin floor section (ft^3). The main basin volume is represented as:

$$L_{main} = L_{floor} + 2H_{main}(S_{main}) \quad (47)$$

$$W_{main} = W_{floor} + 2H_{main}(S_{main}) \quad (48)$$

$$A_{main} = L_{main}(W_{main}) \quad (49)$$

$$V_{main} = \frac{H_{main}}{3}\left(A_{main} + A_{floor} + \sqrt{A_{main}(A_{floor})}\right) \quad (50)$$

Where L_{main} and W_{main} (ft) are the length and width of the main basin section at the point at the top of the basin, H_{main} is the depth of the main basin section (ft), A_{main} is top area of the main basin section (ft^2), and V_{main} is volume of the main basin section (ft^3). The total basin volume is the sum of the individual volumes:

$$V_{total} = ISV + A_{ISV}(D_{TC}) + V_{floor} + V_{main} \quad (51)$$

Where V_{total} is the total basin volume (ft^3) and D_{TC} is the depth of the trickle channel (ft).

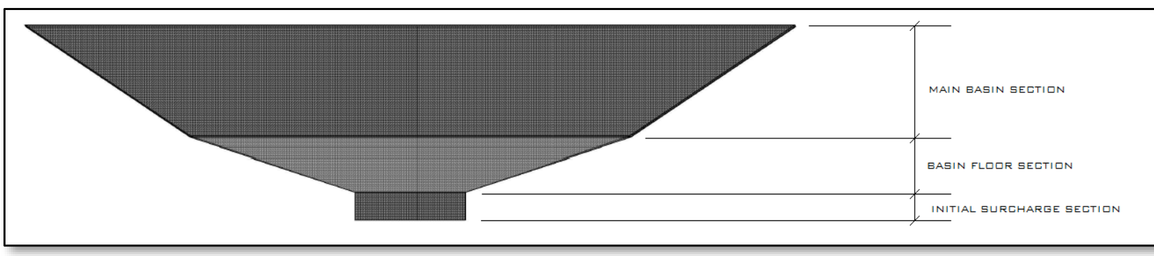


Figure 50. Front view of detention basin model.

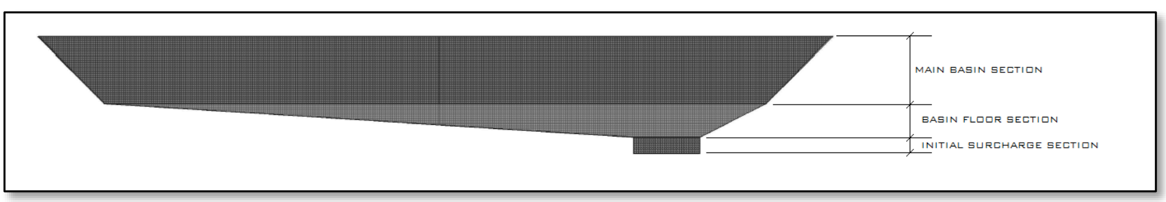


Figure 51. Side view of detention basin model.

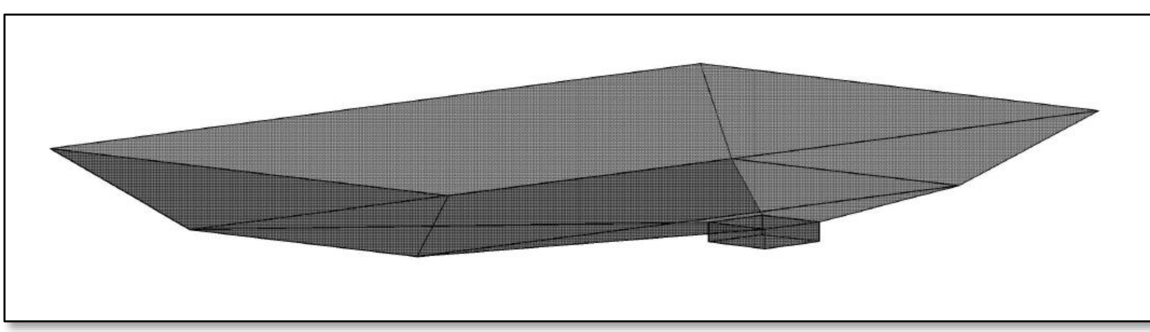


Figure 52. Axonometric projection of detention basin model.

5.2 Sizing of Runoff Volumes and Required Storage Volumes

The runoff volume equations developed in this memorandum were based on Colorado Urban Hydrograph Procedure (CUHP 2005, v1.4.4) modeling and one-hour rainfall depths in the Rainfall chapter of the USDCM. CUHP is a Snyder-based unit hydrograph program that temporally distributes the one-hour rainfall depth into a design storm to create runoff hydrographs for the 2-, 5-, 10-, 25-, 50-, 100, and 500-year recurrence intervals as well as the WQCV- and EURV-sized storms. CUHP was used to evaluate over 2,000 subcatchments from recent UDFCD master planning studies. Watershed characteristics (e.g., size, shape, slope, location of centroid, and imperviousness) were taken directly from the master planning studies. Various combinations of Soil Type (A, B, and C/D) were evaluated for each subcatchment.

By performing a multiple regression analysis on those CUHP subcatchments, equations were developed for the 2-, 5-, 10-, 25-, 50-, 100- and 500-yr return periods for each hydrologic soil group and combined to provide the following watershed runoff equations:

$$V_{Runoff\_2yr} = P_1 A[(0.084 I^{1.440})A\% + (0.084 I^{1.173})B\% + (0.084 I^{1.094})CD\%] \tag{52}$$

$$V_{Runoff\_5yr} = P_1 A[(0.084 I^{1.350})A\% + (0.077I + 0.007)B\% + (0.070I + 0.014)CD\%] \tag{53}$$

$$V_{Runoff\_10yr} = P_1 A[(0.085 I^{1.220})A\% + (0.069I + 0.016)B\% + (0.061I + 0.024)CD\%] \tag{54}$$

$$V_{\_Runoff\_25yr} = P_1 A[(0.082I + 0.004)A\% + (0.055I + 0.031)B\% + (0.048I + 0.038)CD\%] \tag{55}$$

$$V_{Runoff\_50yr} = P_1 A[(0.078I + 0.009)A\% + (0.049I + 0.038)B\% + (0.044I + 0.043)CD\%] \tag{56}$$

$$V_{Runoff\_100yr} = P_1 A[(0.073I + 0.015)A\% + (0.043I + 0.045)B\% + (0.038I + 0.050)CD\%] \tag{57}$$

$$V_{Runoff\_500yr} = P_1 A[(0.064I + 0.025)A\% + (0.036I + 0.053)B\% + (0.031I + 0.058)CD\%] \tag{58}$$

Where $V_{Runoff\_\#yr}$ is the runoff volume for the given return period (acre-feet), P_1 is the one-hour rainfall depth (inches), A is the contributing watershed area (acres), I is the percentage imperviousness (expressed as a decimal), and $A\%$, $B\%$, and $CD\%$ are the percent of each hydraulic soil group (also expressed as a decimal). It should be noted that these equations are a mix of linear and power functions, and as shown in these equations, a watershed's runoff volume for a given return period is a function of the watershed's area, imperviousness, and soil type.

In order to develop estimated storage volume equations, the UD-FSD workbook was used to model full spectrum detention basins. UD-FSD v.1.09 was run for watershed areas of 5-, 10-, 20-, 40-, 60-, and 100-acres at 33%, 67%, and 100% imperviousness. Design storms included the 2-, 5-, 10-, 25-, 50-, and 100-year return period. Hydrologic soil groups A, B, and C/D were evaluated separately. WQCV drain times of 40 hours, 24 hours, and 12 hours were also evaluated (resulting in a total of 972 model runs). The resulting maximum required storage volumes were divided by the corresponding runoff hydrograph volume and those ratios were recorded.

For each return period, the average storage/runoff ratio was plotted vs. imperviousness for each of the three hydrologic soil groups and a power regression was applied as shown in Figure 53 for the 100-year return period. Similar power regression plots were developed for the other five return periods also.

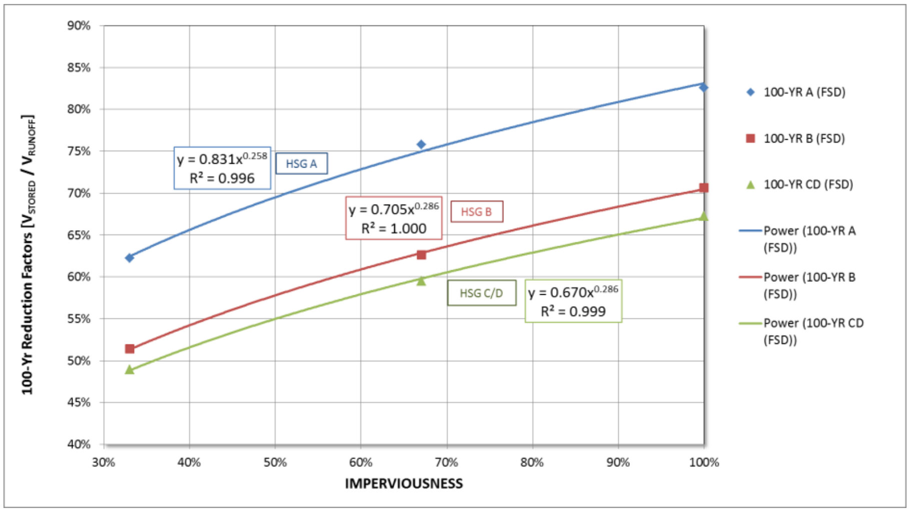


Figure 53. 100-yr Power regression equations for ratio of stored volume to runoff volume as a function of hydrologic soil group and imperviousness.

The resulting storage/runoff ratio equations were then multiplied by the runoff volume equations (converted to watershed inches instead of acre-feet as expressed in Equations 52-58) to develop new storage volume equations. The resulting storage volume equations (in acre-feet) are shown in Equations 59 through 64. The same process was repeated for WQCV drain times of 24 hours and 12 hours. The results were almost identical since the WQCV is such a small percentage of the total detention volume. Therefore, the equations developed for the 40-hour WQCV drain time are considered suitable for all WQCV drain times.

$$V_{Storage\_2yr}(ac-ft) = P_1A[(0.081I^{1.458})A\% + (0.080I^{1.183})B\% + (0.080I^{1.104})CD\%] \quad (59)$$

$$V_{Storage\_5yr}(ac-ft) = P_1A[(0.081I^{1.368})A\% + (0.075I^{1.098} + 0.007I^{0.098})B\% + (0.066I^{1.226} + 0.013I^{0.226})CD\%] \quad (60)$$

$$V_{Storage\_10yr}(acft) = P_1A[(0.082I^{1.237})A\% + (0.063I^{1.254} + 0.015I^{0.254})B\% + (0.052I^{1.371} + 0.021I^{0.371})CD\%] \quad (61)$$

$$V_{Storage\_25yr}(ac-ft) = P_1A[(0.075I^{1.246} + 0.004I^{0.246})A\% + (0.045I^{1.409} + 0.025I^{0.409})B\% + (0.036I^{1.438} + 0.029I^{0.438})CD\%] \quad (62)$$

$$V_{Storage\_50yr}(ac-ft) = P_1A[(0.067I^{1.291} + 0.008I^{0.291})A\% + (0.036I^{1.368} + 0.028I^{0.368})B\% + (0.031I^{1.346} + 0.030I^{0.346})CD\%] \quad (63)$$

$$V_{Storage\_100yr}(ac-ft) = P_1A[(0.061I^{1.258} + 0.012I^{0.258})A\% + (0.030I^{1.286} + 0.032I^{0.286})B\% + (0.025I^{1.286} + 0.034I^{0.286})CD\%] \quad (64)$$

Where $V_{STORAGE\_\#yr}$ is the storage volume (acre-feet), P_1 is the one-hour rainfall depth corresponding to the return period (in), *A* is the watershed area in acres, *I* is the percentage imperviousness (expressed as a decimal), and *A%*, and *B&CD%* are the percent of each hydraulic soil group (expressed as a decimal). A comparison of the 100-yr runoff and storage volumes are shown in Figure 54.

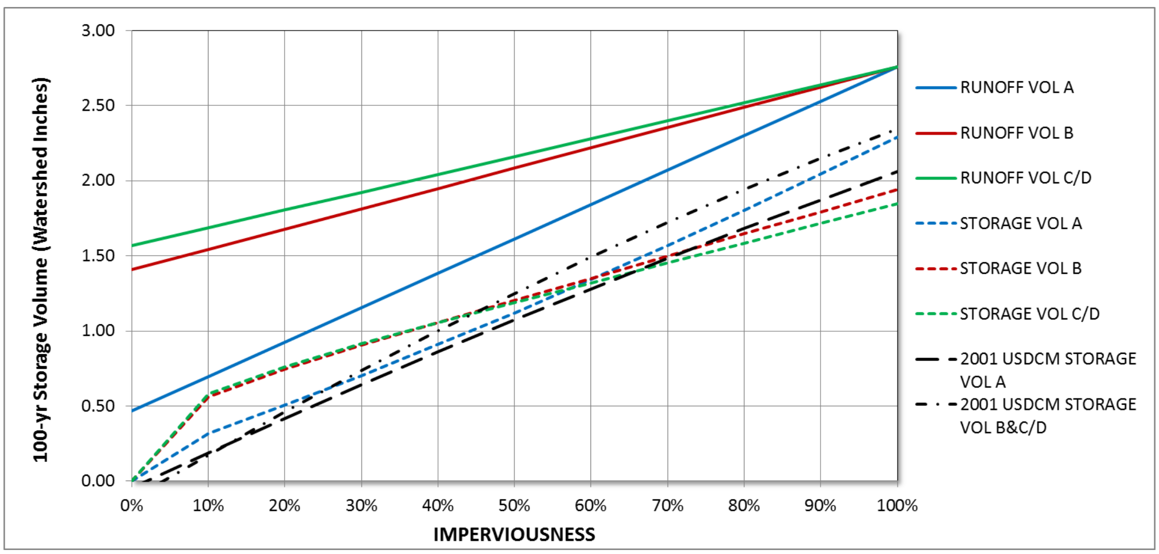


Figure 54. Plot of 100-yr runoff volumes and storage volumes.

5.3 Shaping of Inflow Hydrographs

As described in Section 5.2, the volume of the runoff inflow hydrograph is a function of the watershed size, imperviousness, and soil type. The shape this volume takes is primarily a function of the CUHP design storm distribution, which, in turn is manipulated in CUHP according to the watershed shape factor and slope. In each of the UD-Detention and UD-FSD workbooks, there is a hidden library of over 16,000 inflow hydrographs. The program selects one of these hydrographs for each recurrence interval based on the user's runoff volume input parameters. Because every inflow hydrograph in the hidden library was created in CUHP using default parameters of watershed shape factor (length^2 / area) = 2 and watershed slope = 2%, it is necessary to reshape these hydrographs based on the modeled watershed's specific shape factor and slope. The routine developed to achieve this was developed by running CUHP for watersheds of equal area, imperviousness, and soil type but varying the shape factor from 1 to 4 and varying the slope from 0.5% to 4%. The peak flow rate from each of these tests was then compared to the peak flow rate from CUHP with the default shape factor and slope parameters as a ratio of specific peak flow rate / default peak flow rate.

Plotting these ratios vs. shape factor of 1, 2, 3, and 4 for each slope or 0.5%, 1.0%, 1.5%, 2.0%, 2.5%, 3.0%, and 4.0% provided a family of seven curves for which further regression analysis could be performed in order to create the power regression equation of the form:

$$\text{Hydrograph Constant} = \alpha\left(Shape^{\beta}\right) \tag{65}$$

Where α is the leading coefficient and β is the exponent of the power regression equation. The values for α and β are shown in Table 6, the shape of the curves is shown in Figure 55, and the plots of for α and β are shown I Figure 56.

Table 6. Leading coefficient α and exponent β.

Slope	***α***	***β***
0.5	1.0013	-0.304
1.0	1.1093	-0.298
1.5	1.1706	-0.291
2.0	1.2138	-0.284
2.5	1.2391	-0.275
3.0	1.2695	-0.273
4.0	1.3118	-0.263

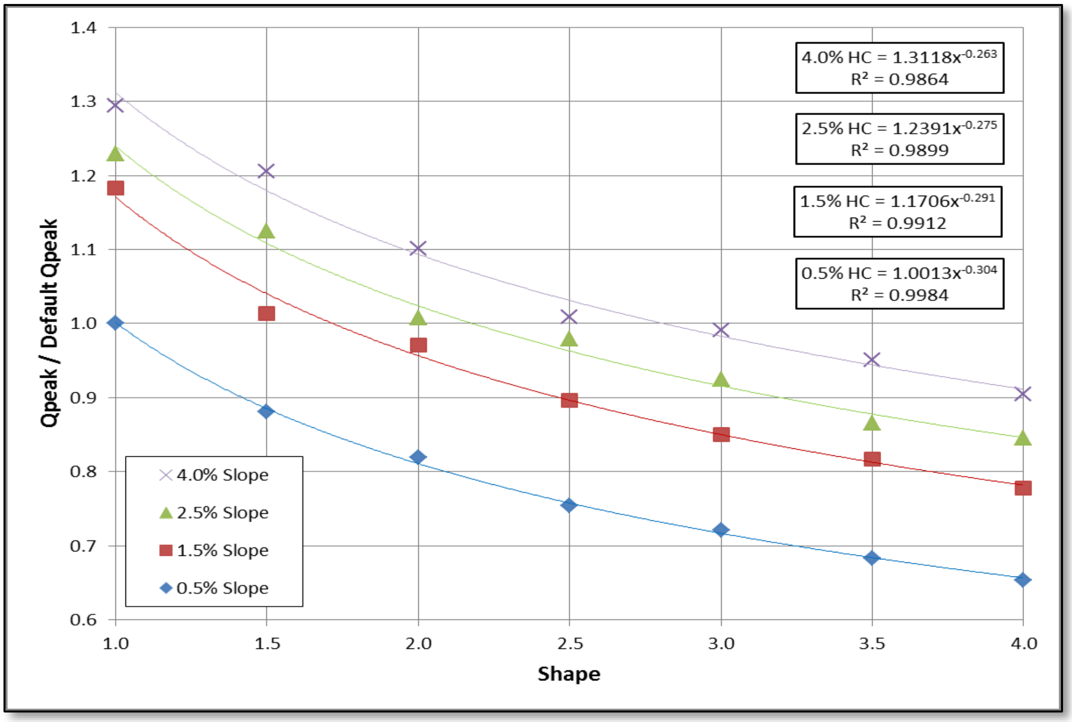


Figure 55. Plot of hydrograph constants vs. shape factors for various slopes.

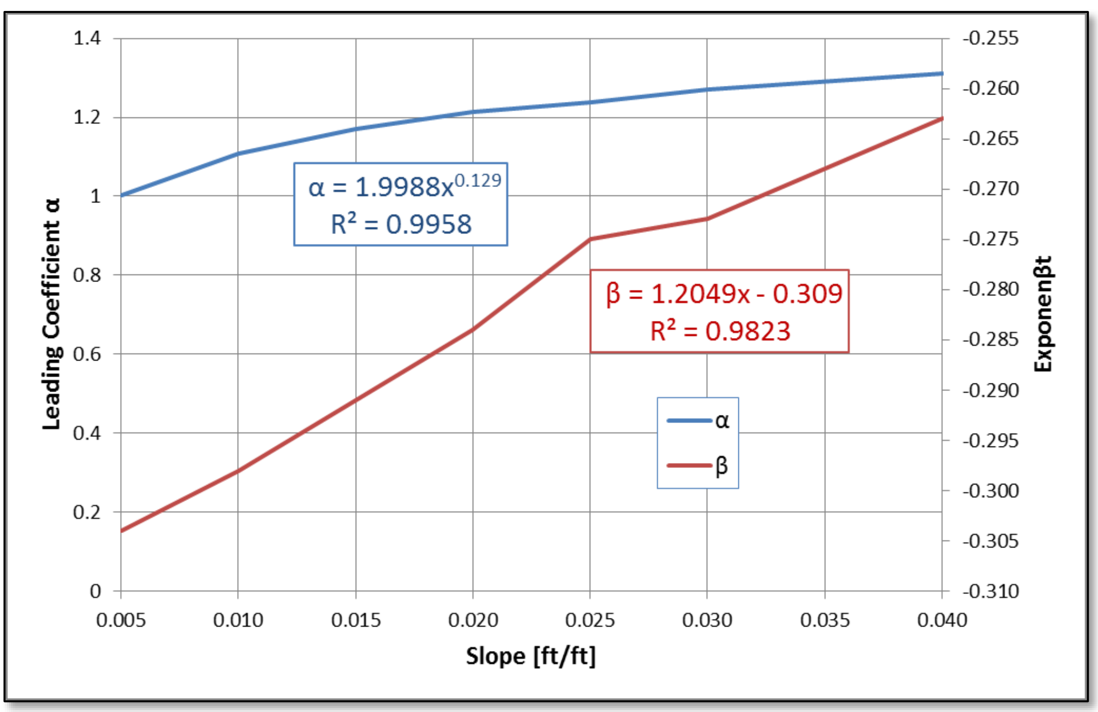


Figure 56. Plot of leading coefficient α and exponent β vs. watershed slope.

Performing regression analysis on the curves of α and β in Figure 56 provides a power equation to represent α and β and a linear equation to represent β, as:

$$\alpha = 2.03(Slope^{0.13}) \tag{66}$$

$$\beta = 1.2(Slope) - 0.31 \tag{67}$$

Combining Eqs. 66 and 67 with Eq. 65 provides the final form of the hydrograph constant:

$$\text{Hydrograph Constant} = 2.03(Slope^{0.13})\left(Shape^{(1.2(Slope)-0.31)}\right) \tag{68}$$

To make the shape adjustment to each of the recurrence interval inflow hydrographs while conserving the volume of those hydrographs, the UD-Detention and UD-FSD programs multiply each incremental flow rate by the hydrograph constant while dividing the standard 5-minute time step by the same hydrograph constant. Watersheds shorter and/or steeper than those with the default shape factor of 2 and slope of 2% will produce higher flow rates at each time step with a shorter standard time step, while the opposite condition will occur with longer and/or flatter watersheds, as demonstrated in Figure 57.

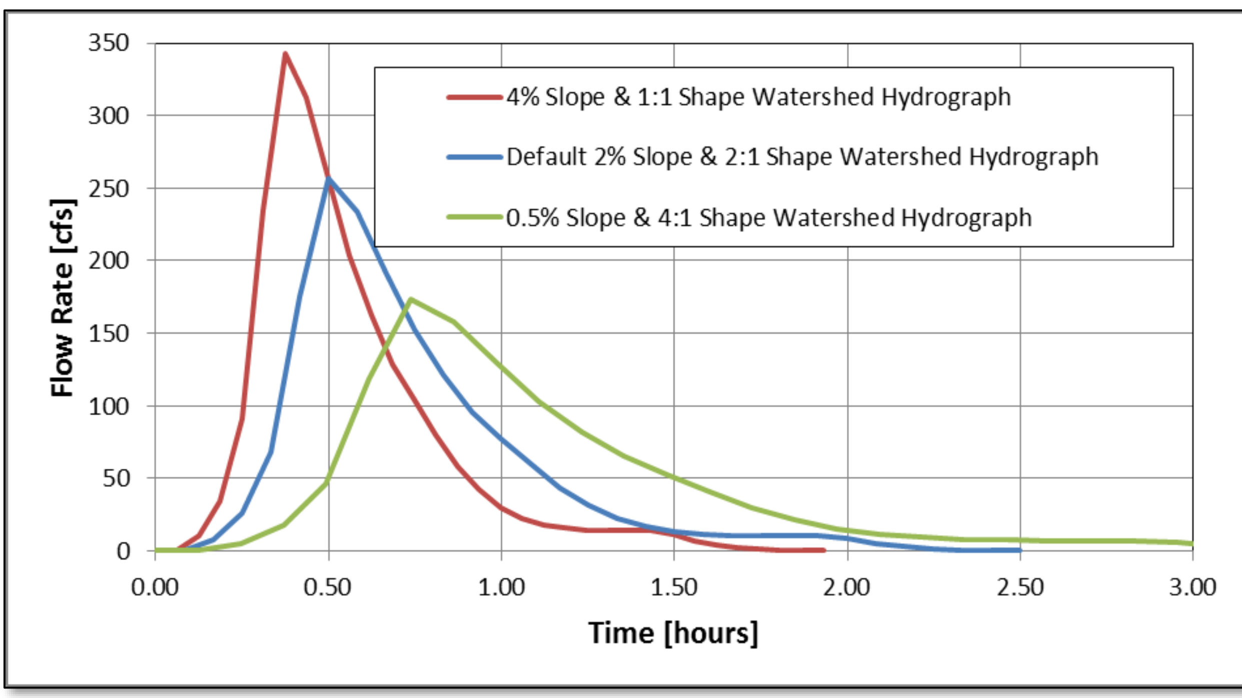


Figure 57. These three hydrographs have different flow rates at each time step based on watershed shape factor and slope, but all have the same volume (i.e., the area under the curve) based on the watershed area, imperviousness, and soil.

5.4 Using the UD-Detention Workbook Model

UDFCD has created three design workbooks to assist CDOT and others in a simplified design method for extended detention basins. The SDI (Statutory Detention and Infiltration) Design Data workbook was specifically created to allow CDOT and others to demonstrate statutory compliance with the new Colorado state law described in Section 6. UD-FSD provides tools to design a full spectrum detention (FSD) basin only. UD-Detention can be used for FSD basins but can also be used for EDBs, bioretention BMPs, sand filter BMPs, constructed treatment wetlands, and retention ponds. It is the most versatile of the three workbooks and also the most complicated, and for those reasons this section will cover the UD-Detention model. Once familiar with the UD-Detention model, the other two workbooks will be easily understood. The UD-Detention workbook is an extremely powerful design tool, featuring nearly 7,000 lines of Visual Basic programming code to aid the designer in creating a stormwater management facility.

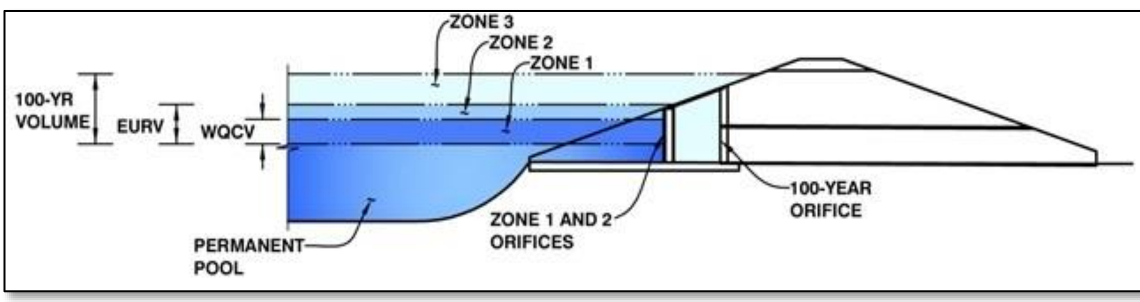


Figure 58. UD-Detention figure showing the three design zones.

5.41 Basin Worksheet

The UD-Detention workbook has two main worksheets, the Basin sheet and the Outlet Structure sheet. The Basin worksheet allows the user to size the storage volume of the basin based on mathematical model described in Section 5.1, Equations 40 through 51 and the runoff and required storage volumes presented in Section 5.2, Equations 52 through 64. In order for this process to initiate, the user must enter basic stormwater treatment type parameters and watershed parameters as shown in Figure 61, and stormwater treatment facility parameters as shown in Figure 62.

There are two dropdown menus on the Basin worksheet; the “Select BMP Type” dropdown menu shown in Figure 59, and the “Location for 1-hr Rainfall Depths” dropdown menu shown in Figure 60. The choices for the latter dropdown are all within the UDFCD boundary area, but there is an option to select “User Input” from this dropdown and then manually enter the appropriate one-hour rainfall depts. From NOAA Atlas 14 in the user input rainfall depth cells.

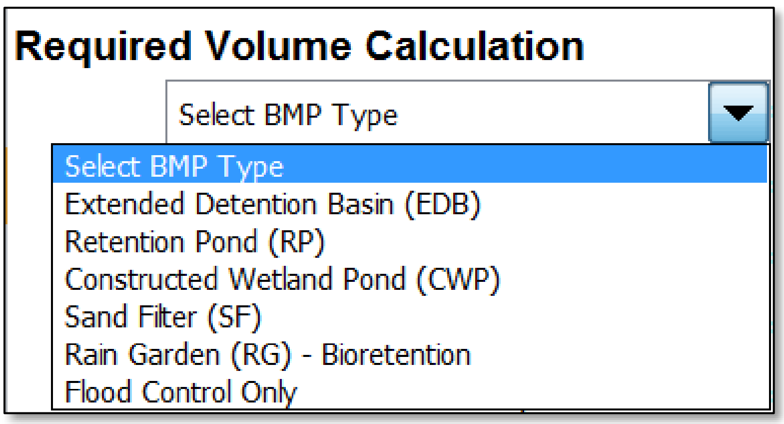


Figure 59. “Select BMP Type” dropdown menu.

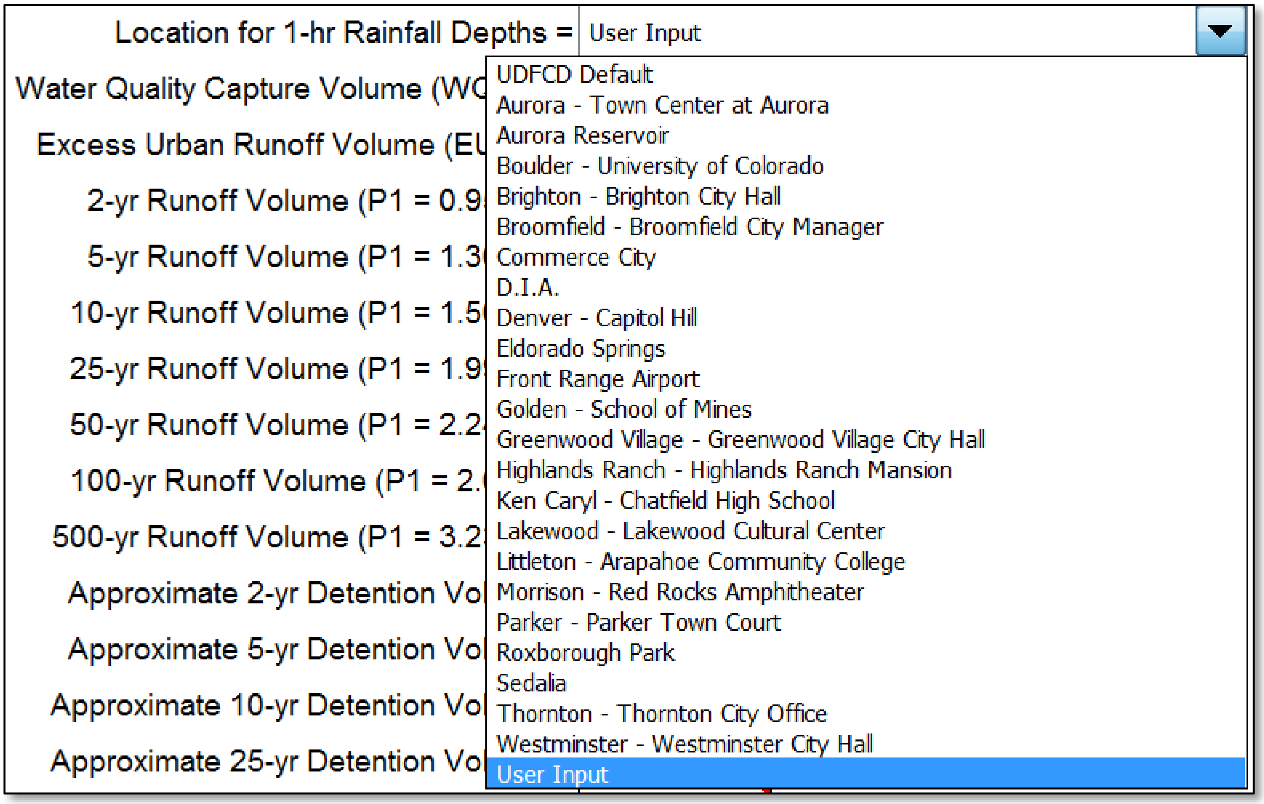


Figure 60. "Location for 1-hr Rainfall Depths" dropdown menu.

Required Volume Calculation

Extended Detention Basin (EDB)	EDB			
Watershed Area =	50.00	acres		
Watershed Length =	2,087	ft		
Watershed Slope =	0.020	ft/ft		
Watershed Imperviousness =	50.00%	percent		
Percentage Hydrologic Soil Group A =	0.0%	percent		
Percentage Hydrologic Soil Group B =	0.0%	percent		
Percentage Hydrologic Soil Groups C/D =	100.0%	percent		
Desired WQCV Drain Time =	40.0	hours		
Location for 1-hr Rainfall Depths =	Commerce City			
Water Quality Capture Volume (WQCV) =	0.859	acre-feet	Optional User Input 1-hr Precipitation	
Excess Urban Runoff Volume (EURV) =	2.365	acre-feet		
2-yr Runoff Volume (P1 = 0.95 in.) =	1.869	acre-feet		inches
5-yr Runoff Volume (P1 = 1.36 in.) =	3.332	acre-feet		inches
10-yr Runoff Volume (P1 = 1.56 in.) =	4.251	acre-feet		inches
25-yr Runoff Volume (P1 = 1.99 in.) =	6.169	acre-feet		inches
50-yr Runoff Volume (P1 = 2.24 in.) =	7.280	acre-feet		inches
100-yr Runoff Volume (P1 = 2.6 in.) =	8.970	acre-feet		inches
500-yr Runoff Volume (P1 = 3.23 in.) =	11.870	acre-feet		inches
Approximate 2-yr Detention Volume =	1.768	acre-feet		
Approximate 5-yr Detention Volume =	2.674	acre-feet		
Approximate 10-yr Detention Volume =	2.835	acre-feet		
Approximate 25-yr Detention Volume =	3.452	acre-feet		
Approximate 50-yr Detention Volume =	4.009	acre-feet		
Approximate 100-yr Detention Volume =	4.958	acre-feet		

Figure 61. User-entered treatment type and watershed design parameters (blue cells) and calculated results (white cells) in the UD-Detention Basin sheet.

Stage-Storage Calculation		
Zone 1 Volume (WQCV)	0.859	acre-feet
Zone 2 Volume (EURV - Zone 1)	1.506	acre-feet
Zone 3 Volume (100-year - Zones 1 & 2)	2.593	acre-feet
Total Detention Basin Volume =	4.958	acre-feet
Initial Surcharge Volume (ISV) =	112	ft^3
Initial Surcharge Depth (ISD) =	0.33	ft
Total Available Detention Depth (H_{total}) =	8.00	ft
Depth of Trickle Channel (H_{TC}) =	0.50	ft
Slope of Trickle Channel (S_{TC}) =	0.005	ft/ft
Slopes of Main Basin Sides (S_{main}) =	4	H:V
Basin Length-to-Width Ratio ($R_{L/W}$) =	2	
Initial Surcharge Area (A_{ISV}) =	337	ft^2
Surcharge Volume Length (L_{ISV}) =	18.4	ft
Surcharge Volume Width (W_{ISV}) =	18.4	ft
Depth of Basin Floor (H_{FLOOR}) =	0.96	ft
Length of Basin Floor (L_{FLOOR}) =	213.7	ft
Width of Basin Floor (W_{FLOOR}) =	114.1	ft
Area of Basin Floor (A_{FLOOR}) =	24,386	ft^2
Volume of Basin Floor (V_{FLOOR}) =	8,806	ft^3
Depth of Main Basin (H_{MAIN}) =	6.21	ft
Length of Main Basin (L_{MAIN}) =	263.4	ft
Width of Main Basin (W_{MAIN}) =	163.8	ft
Area of Main Basin (A_{MAIN}) =	43,137	ft^2
Volume of Main Basin (V_{MAIN}) =	206,893	ft^3
Calculated Total Basin Volume (V_{total}) =	**4.958**	acre-feet

Figure 62. User-entered stormwater treatment facility design parameters (blue cells) and calculated results (white cells) in the UD-Detention Basin sheet.

In the UD-Detention workbook, the blue cells are for user input parameters and the white cells are calculated values. After the necessary design parameters are entered as shown in Figures 61 and 62, the program creates a stage-area-volume table of the proposed facility, as shown in Figures 63 through 64.

Depth Increment =	0.1	ft							
Stage - Storage Description	Stage (ft)	Optional Override Stage (ft)	Length (ft)	Width (ft)	Area (ft^2)	Optional Override Area (ft^2)	Area (acre)	Volume (ft^3)	Volume (ac-ft)
Micropool	0.00		18.4	18.4	337		0.008		
ISV	0.33		18.4	18.4	337		0.008	111	0.003
	0.40		18.4	18.4	337		0.008	132	0.003
	0.50		18.4	18.4	337		0.008	165	0.004
	0.60		18.4	18.4	337		0.008	199	0.005
	0.70		18.4	18.4	337		0.008	233	0.005
	0.80		18.4	18.4	337		0.008	266	0.006
	0.90		30.0	24.1	722		0.017	311	0.007
	1.00		50.4	34.1	1,717		0.039	429	0.010
	1.10		70.8	44.1	3,119		0.072	667	0.015
	1.20		91.2	54.1	4,930		0.113	1,067	0.024
	1.30		111.6	64.1	7,149		0.164	1,667	0.038
	1.40		132.0	74.1	9,776		0.224	2,510	0.058
	1.50		152.4	84.1	12,811		0.294	3,636	0.083
	1.60		172.8	94.1	16,254		0.373	5,086	0.117
	1.70		193.2	104.1	20,104		0.462	6,900	0.158
Floor	1.79		213.6	114.1	24,363		0.559	9,120	0.209
	1.80		213.6	114.1	24,363		0.559	9,120	0.209
	1.90		214.5	114.9	24,647		0.566	11,572	0.266
	2.00		215.3	115.7	24,911		0.572	14,050	0.323
	2.10		216.2	116.6	25,203		0.579	16,806	0.386
	2.20		217.0	117.4	25,470		0.585	19,340	0.444
	2.30		217.8	118.2	25,738		0.591	21,900	0.503
	2.40		218.6	119.0	26,008		0.597	24,487	0.562
	2.50		219.4	119.8	26,278		0.603	27,102	0.622
	2.60		220.2	120.6	26,550		0.610	29,743	0.683
	2.70		221.0	121.4	26,824		0.616	32,412	0.744
	2.80		221.8	122.2	27,098		0.622	35,108	0.806
Zone 1 (WQCV)	2.89		222.5	122.9	27,346		0.628	37,558	0.862
	2.90		222.6	123.0	27,374		0.628	37,831	0.868
	3.00		223.4	123.8	27,651		0.635	40,583	0.932
	3.10		224.2	124.6	27,929		0.641	43,362	0.995
	3.20		225.0	125.4	28,209		0.648	46,168	1.060
	3.30		225.8	126.2	28,490		0.654	49,003	1.125
	3.40		226.6	127.0	28,772		0.661	51,867	1.191
	3.50		227.4	127.8	29,056		0.667	54,758	1.257
	3.60		228.2	128.6	29,340		0.674	57,678	1.324
	3.70		229.0	129.4	29,627		0.680	60,626	1.392
	3.80		229.8	130.2	29,914		0.687	63,603	1.460
	3.90		230.6	131.0	30,202		0.693	66,609	1.529
	4.00		231.4	131.8	30,492		0.700	69,644	1.599
	4.10		232.2	132.6	30,784		0.707	72,707	1.669
	4.20		233.0	133.4	31,076		0.713	75,800	1.740
	4.30		233.8	134.2	31,370		0.720	78,923	1.812
	4.40		234.6	135.0	31,665		0.727	82,074	1.884
	4.50		235.4	135.8	31,961		0.734	85,256	1.957
	4.60		236.2	136.6	32,259		0.741	88,467	2.031
	4.70		237.0	137.4	32,557		0.747	91,707	2.105
	4.80		237.8	138.2	32,858		0.754	94,978	2.180
	4.90		238.6	139.0	33,159		0.761	98,279	2.256
	5.00		239.4	139.8	33,462		0.768	101,610	2.333
Zone 2 (EURV)	5.05		239.8	140.2	33,613		0.772	103,287	2.371
	5.10		240.2	140.6	33,766		0.775	104,971	2.410
	5.20		241.0	141.4	34,071		0.782	108,363	2.488
	5.30		241.8	142.2	34,377		0.789	111,785	2.566
	5.40		242.6	143.0	34,685		0.796	115,239	2.646
	5.50		243.4	143.8	34,994		0.803	118,723	2.725
	5.60		244.2	144.6	35,305		0.810	122,238	2.806
	5.70		245.0	145.4	35,616		0.818	125,784	2.888
	5.80		245.8	146.2	35,929		0.825	129,361	2.970
	5.90		246.6	147.0	36,243		0.832	132,969	3.053
	6.00		247.4	147.8	36,559		0.839	136,610	3.136
	6.10		248.2	148.6	36,876		0.847	140,281	3.220
	6.20		249.0	149.4	37,194		0.854	143,985	3.305
	6.30		249.8	150.2	37,513		0.861	147,720	3.391
	6.40		250.6	151.0	37,834		0.869	151,487	3.478
	6.50		251.4	151.8	38,156		0.876	155,287	3.565
	6.60		252.2	152.6	38,479		0.883	159,119	3.653
	6.70		253.0	153.4	38,803		0.891	162,983	3.742
	6.80		253.8	154.2	39,129		0.898	166,879	3.831
	6.90		254.6	155.0	39,456		0.906	170,808	3.921
	7.00		255.4	155.8	39,784		0.913	174,770	4.012
	7.10		256.2	156.6	40,114		0.921	178,765	4.104
	7.20		257.0	157.4	40,445		0.928	182,793	4.196
	7.30		257.8	158.2	40,777		0.936	186,854	4.290
	7.40		258.6	159.0	41,110		0.944	190,949	4.384
	7.50		259.4	159.8	41,445		0.951	195,076	4.478
	7.60		260.2	160.6	41,781		0.959	199,238	4.574
	7.70		261.0	161.4	42,118		0.967	203,433	4.670
	7.80		261.8	162.2	42,457		0.975	207,661	4.767
	7.90		262.6	163.0	42,796		0.982	211,924	4.865
Zone 3 (100-year)	8.00		263.4	163.8	43,137		0.990	216,221	4.964

Figure 63. Stage-area-volume table created by UD-Detention program based on user inputs.

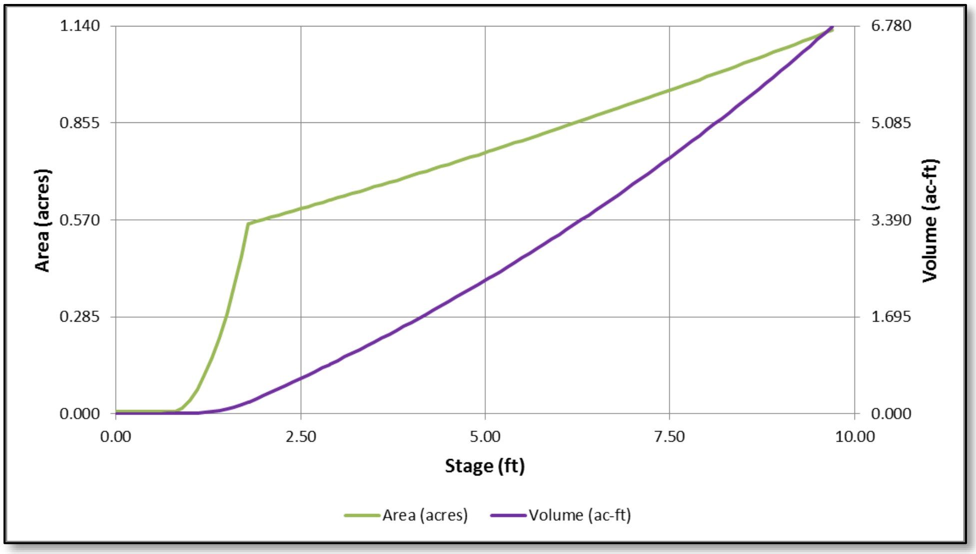


Figure 64. Graphical representation of tabulated data in Figure 63 prepared by UD-Detention program based on user inputs.

Once the required information has been entered in the Basin worksheet, the calculations automatically create the stage-area-volume table based on the required storage volume, the given maximum depth, basin slope, side slopes, and length-to-width ratio. There will be cases where no mathematical solution is available that can satisfy all of the given constraints. When this happens, the program will notify the user as shown in Figure 65. The user can then select "Yes" and allow the program to incrementally flatten the detention basin trickle channel slope until a mathematical solution is available, or the user can select "No" and manually change any of the aforementioned design constraint parameters.

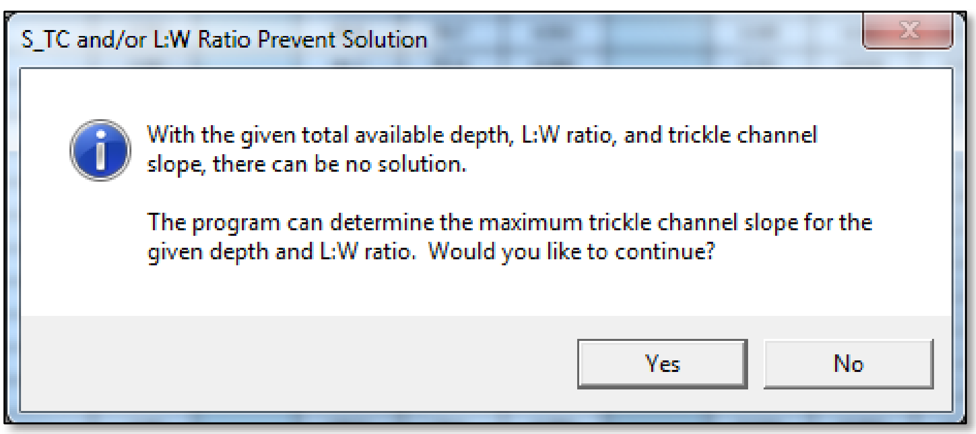


Figure 65. Example of built-in automation assists the user in sizing the storage volume.

5.42 Outlet Structure Worksheet

When the Basin worksheet user inputs have been satisfied and the program has been run, the user can proceed to the Outlet Structure worksheet. This worksheet is divided into 9 visible and 2 hidden (but optionally viewable) sections, including:

1. Basic information as to how the three zones will be drained,
2. Information specific to the EURV and/or WQCV orifice plate or elliptical slot weir,
3. Optional additional information regarding up to sixteen water quality drain orifices,
4. Optional vertical orifice information,
5. Overflow outlet weir and grate information,
6. 100-year (or other design event) orifice and restrictor plate information,
7. Emergency spillway information,
8. Routed hydrograph results,
9. Optional user-defined inflow hydrograph table,
10. Hidden (but optionally viewable) stage-storage-discharge result table, and
11. Hidden (but optionally viewable) Modified Puls reservoir routing table.

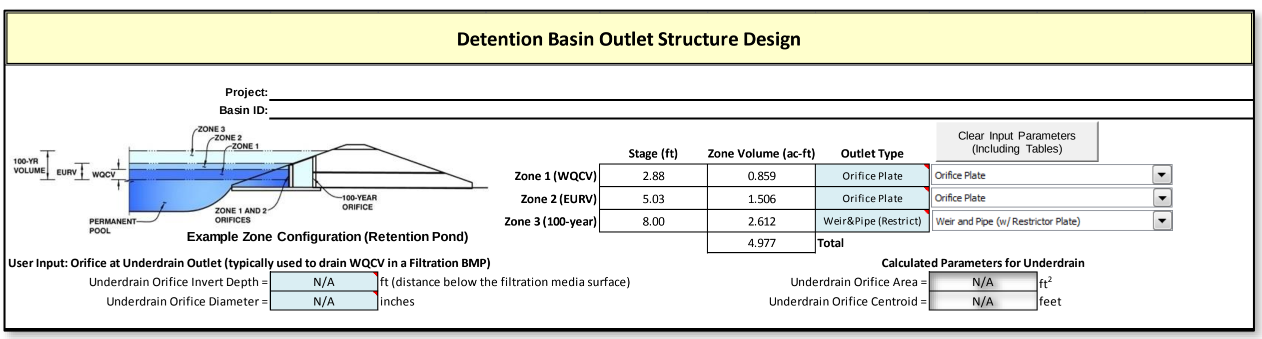


Figure 66. Outlet Structure Worksheet Section 1, showing user selections for Zones 1, 2, and 3.

Because EDB was selected as the BMP treatment method on the Basin worksheet in Figure 66, the underdrain user input (blue) cells are left blank.

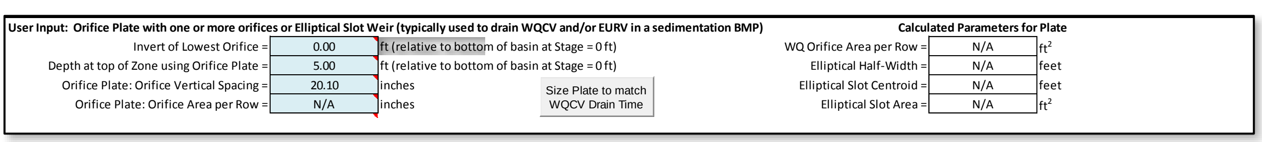


Figure 67. Outlet Structure Worksheet Section 2, showing user selections for water quality orifice placement and sizing in order to drain zones 1 and 2.

In Figure 67, previous selection of the elliptical slot weir in Section 1 had resulted in a slot with a gap of less than 0.375 inches, which would have been prone to clogging. The user then selected the orifice plate consisting of 3 orifices spaced 20.1 inches on center vertically. The Area per Row value is N/A because the user overrode the top orifice area as shown in Figure 68.

User Input: Stage and Total Area of Each Orifice Row (numbered from lowest to highest)

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	1.67	3.33					
Orifice Area (sq. inches)	4.19	4.19	12.00					

	Row 9 (optional)	Row 10 (optional)	Row 11 (optional)	Row 12 (optional)	Row 13 (optional)	Row 14 (optional)	Row 15 (optional)	Row 16 (optional)
Stage of Orifice Centroid (ft)								
Orifice Area (sq. inches)								

Figure 68. Outlet Structure Worksheet Section 3, showing the stage and area of the three water quality (EURV and WQCV) draining orifices.

In Figure 68, note that the user overrode the top orifice area in order to drain the storage volumes in compliance with the new Colorado statutory requirements (described in Section 6 of this report). Typically, only the first three rows will be used for the three required water quality orifices. The other 13 rows are solely for the purpose of analyzing an existing facility designed before the current recommendations became operative.

User Input: Vertical Orifice (Circular or Rectangular)

	Not Selected	Not Selected	
Invert of Vertical Orifice =	N/A	N/A	ft (relative to bottom of basin at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice =	N/A	N/A	ft (relative to bottom of basin at Stage = 0 ft)
Vertical Orifice Diameter =	N/A	N/A	inches

Calculated Parameters for Vertical Orifice

	Not Selected	Not Selected	
Vertical Orifice Area =	N/A	N/A	ft^2
Vertical Orifice Centroid =	N/A	N/A	feet

Figure 69. Outlet Structure Worksheet Section 4, showing the optional vertical orifice input and calculation cells.

Figure 69 shows Section 4, where the user can add a vertical orifice to assist in shaping the drawdown curve to meet the design intent. The vertical orifice is optional and for most EDBs will not be used.

User Input: Overflow Weir (Dropbox) and Grate (Flat or Sloped)

	Zone 3 Weir	Not Selected	
Overflow Weir Front Edge Height, Ho =	5.00	N/A	ft (relative to bottom of basin at Stage = 0 ft)
Overflow Weir Front Edge Length =	8.00	N/A	feet
Overflow Weir Slope =	4.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	8.00	N/A	feet
Overflow Grate Open Area % =	70%	N/A	%, grate open area / total area
Debris Clogging % =	50%	N/A	%

Calculated Parameters for Overflow Weir

	Zone 3 Weir	Not Selected	
Height of Grate Upper Edge, H_t =	7.00	N/A	feet
Over Flow Weir Slope Length =	8.25	N/A	feet
Grate Open Area / 100-yr Orifice Area =	9.22	N/A	should be $\geq$ 4
Overflow Grate Open Area w/o Debris =	46.18	N/A	ft^2
Overflow Grate Open Area with Debris =	23.09	N/A	ft^2

Figure 70. Outlet Structure Worksheet Section 5, showing the design parameter inputs and calculations for the overflow outlet and grate.

In Figure 70, the user inputs the design parameters for the overflow outlet and grate in the blue cells, while preliminary calculations are completed in the white cells. The work by Guo et al., as discussed in Section 4.2 of this report, provided the mathematical expressions to calculate the flow through an inclined overflow outlet grate.

User Input: Outlet Pipe w/ Flow Restriction Plate (Circular Orifice, Restrictor Plate, or Rectangular Orifice)

	Zone 3 Restrictor	Not Selected	
Depth to Invert of Outlet Pipe =	3.00	N/A	ft (distance below bottom of basin at Stage = 0 ft)
Outlet Pipe Diameter =	36.00	N/A	inches
Restrictor Plate Height Above Pipe Invert =	24.00		inches

Size Outlet Plate to match 90% of Predevelopment 100-year Peak Runoff Rate

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 3 Restrictor	Not Selected	
Outlet Orifice Area =	5.01	N/A	ft^2
Outlet Orifice Centroid =	1.12	N/A	feet
Restrictor Plate on Pipe =	1.91	N/A	radians

Figure 71. Outlet Structure Worksheet Section 6, showing the design parameter inputs and calculations for the 100-year (or other design event) orifice.

Figure 71 shows Section 6, where the user can size the 100-year (or other design event) orifice. An optional button can be clicked to run a sizing program that will automatically size the 100-year restrictor plate or orifice plate in order to meter the design flow at 90% of the estimated predeveloped flow rate. This estimated flow rate is explained in a technical memorandum titled "*Determination of Watershed Predeveloped Peak Unit Flow Rates as the Basis for Detention Basin Design*" and posted on the UDFCD web site at www.udfcd.org. This orifice is in the bottom of the outlet structure and acts as the final flow control to prevent downstream flooding during the design event.

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	9.10	ft (relative to bottom of basin at Stage = 0 ft)
Spillway Crest Length =	67.00	feet
Spillway End Slopes =	4.00	H:V
Freeboard above Max Water Surface =	1.00	feet

Size Emergency Spillway to pass Developed 100-yr Peak Runoff Rate

Calculated Parameters for Spillway

Spillway Design Flow Depth=	0.97	feet
Stage at Top of Freeboard =	11.07	feet
Basin Area at Top of Freeboard =	1.25	acres

Figure 72. Outlet Structure Worksheet Section 7, showing the design parameter inputs and calculations for the emergency spillway. A 500-year inflow hydrograph is supplied to be routed through this spillway.

Figure 72 shows Section 7, the input cells and preliminary calculations for the emergency spillway. This spillway is typically sized to pass the undetained 100-year inflow hydrograph at a depth of one foot. An optional button can be clicked to run a sizing program that will automatically size the spillway to meet these design constraints.

Routed Hydrograph Results									
Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	0.95	1.34	1.64	2.02	2.32	2.61	3.29
Calculated Runoff Volume (acre-ft) =	0.859	2.365	1.869	3.283	4.469	6.262	7.540	9.005	12.091
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.859	2.365	1.868	3.283	4.469	6.256	7.540	9.002	12.086
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.02	0.32	0.54	1.02	1.29	1.61	2.24
Predevelopment Peak Q (cfs) =	0.0	0.0	0.8	16.1	27.1	50.9	64.4	80.5	112.0
Peak Inflow Q (cfs) =	19.1	52.4	41.4	73.1	99.8	139.9	168.7	201.3	269.8
Peak Outflow Q (cfs) =	0.4	1.1	0.9	8.4	25.1	54.8	72.5	75.5	114.0
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.5	0.9	1.1	1.1	0.9	1.0
Structure Controlling Flow =	Plate	Plate	Plate	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Outlet Plate 1	Outlet Plate 1	Spillway
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	0.2	0.5	1.2	1.5	1.6	1.7
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	40	66	60	69	69	69	69	69	69
Time to Drain 99% of Inflow Volume (hours) =	**40**	66	60	69	69	70	70	70	70
Maximum Ponding Depth (ft) =	2.79	4.85	4.21	5.58	6.15	6.77	7.16	7.93	9.40
Area at Maximum Ponding Depth (acres) =	0.62	0.76	0.72	0.81	0.85	0.90	0.93	0.99	1.11
Maximum Volume Stored (acre-ft) =	0.798	2.220	1.748	2.794	3.268	3.810	4.176	4.904	6.441

Figure 73. Outlet Structure Worksheet Section 8, the final output table showing the design parameter inputs and calculations for the emergency spillway. A 500-year inflow hydrograph is supplied to be routed through this spillway.

In Figure 73, all of the results from the preliminary calculations and the hidden Modified Puls reservoir routing tables are reported in Section 8 for the user's analysis. If the results are satisfactory, the work is complete. If the results are not satisfactory, the user can go back to any of the preceding sections and modify those inputs to adjust the results in this table.

Detention Basin Outlet Structure Design

Reset hydrographs to default values from workbook

Outflow Hydrograph Workbook Filename:

☐ Use relative path name

Export Outflow Hydrographs to a blank workbook for later use in a downstream UD-Detention Workbook

Storm Inflow Hydrographs

The user can override the calculated inflow hydrographs from this workbook with inflow hydrographs developed in a separate program.

	SOURCE	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK	WORKBOOK
Time Interval	TIME	WQCV [cfs]	EURV [cfs]	2 Year [cfs]	5 Year [cfs]	10 Year [cfs]	25 Year [cfs]	50 Year [cfs]	100 Year [cfs]	500 Year [cfs]
5.00 min	0:00:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0:05:00	0.03	0.08	0.06	0.10	0.13	0.17	0.19	0.21	0.26
Hydrograph	0:10:00	1.00	2.43	2.00	3.17	4.01	5.15	5.89	6.66	8.14
Constant	0:15:00	2.48	6.38	5.13	8.61	11.35	15.29	18.00	20.99	27.03
1.000	0:20:00	6.85	17.39	14.05	23.29	30.47	40.69	47.68	55.36	70.85
	0:25:00	17.26	44.13	35.58	59.30	77.90	104.50	122.77	142.86	183.58
	0:30:00	19.10	52.42	41.38	73.09	99.79	139.94	168.69	201.30	269.79
	0:35:00	16.33	45.83	36.01	64.22	88.20	124.65	151.13	181.68	246.68
	0:40:00	13.20	37.40	29.34	52.44	72.11	102.09	123.87	148.91	202.28
	0:45:00	10.58	29.91	23.48	41.89	57.55	81.42	98.71	118.53	160.68
	0:50:00	8.33	23.64	18.54	33.17	45.65	64.71	78.54	94.39	128.17
	0:55:00	6.60	18.71	14.68	26.21	36.06	51.12	62.06	74.59	101.32
	1:00:00	5.43	15.18	11.95	21.18	29.10	41.26	50.08	60.18	81.72
	1:05:00	3.91	11.20	8.76	15.81	21.96	31.46	38.41	46.42	63.57
	1:10:00	2.93	8.32	6.53	11.66	16.07	22.84	27.78	33.47	45.67
	1:15:00	2.04	5.88	4.59	8.31	11.54	16.53	20.18	24.38	33.38
	1:20:00	1.48	4.21	3.30	5.91	8.15	11.60	14.11	17.01	23.21
	1:25:00	1.15	3.26	2.55	4.56	6.29	8.93	10.86	13.06	17.77
	1:30:00	0.94	2.64	2.08	3.69	5.07	7.18	8.71	10.47	14.20
	1:35:00	0.84	2.32	1.83	3.22	4.41	6.23	7.55	9.05	12.24
	1:40:00	0.80	2.21	1.74	3.07	4.19	5.88	7.10	8.49	11.43
	1:45:00	0.78	2.16	1.70	3.00	4.08	5.72	6.90	8.25	11.09
	1:50:00	0.78	2.16	1.70	3.00	4.08	5.71	6.88	8.21	11.03
	1:55:00	0.78	2.16	1.70	3.00	4.08	5.71	6.88	8.21	11.03
	2:00:00	0.51	1.49	1.16	2.13	2.99	4.32	5.29	6.43	8.86
	2:05:00	0.30	0.88	0.68	1.24	1.73	2.49	3.05	3.71	5.13
	2:10:00	0.17	0.50	0.39	0.72	1.00	1.45	1.78	2.16	2.99
	2:15:00	0.09	0.27	0.21	0.39	0.54	0.78	0.96	1.17	1.61
	2:20:00	0.04	0.13	0.10	0.19	0.27	0.40	0.50	0.61	0.86
	2:25:00	0.01	0.04	0.03	0.06	0.09	0.15	0.19	0.23	0.34
	2:30:00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03	0.05
	2:35:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Figure 74. Outlet Structure Worksheet Section 9, the optional user-input inflow hydrograph table.

Figure 74 shows Section 9, the optional user-input inflow hydrograph table. If the user chooses not to allow the program to select from the hidden library of over 16,000 inflow hydrographs, custom inflow hydrographs may be entered in this table. A limitation with this option is that all of the hydrographs entered must have a common time interval. The heading at the top of each column changes from “Workbook” to “User” when the hydrograph in that column does not exactly match the hydrograph from the hidden library.

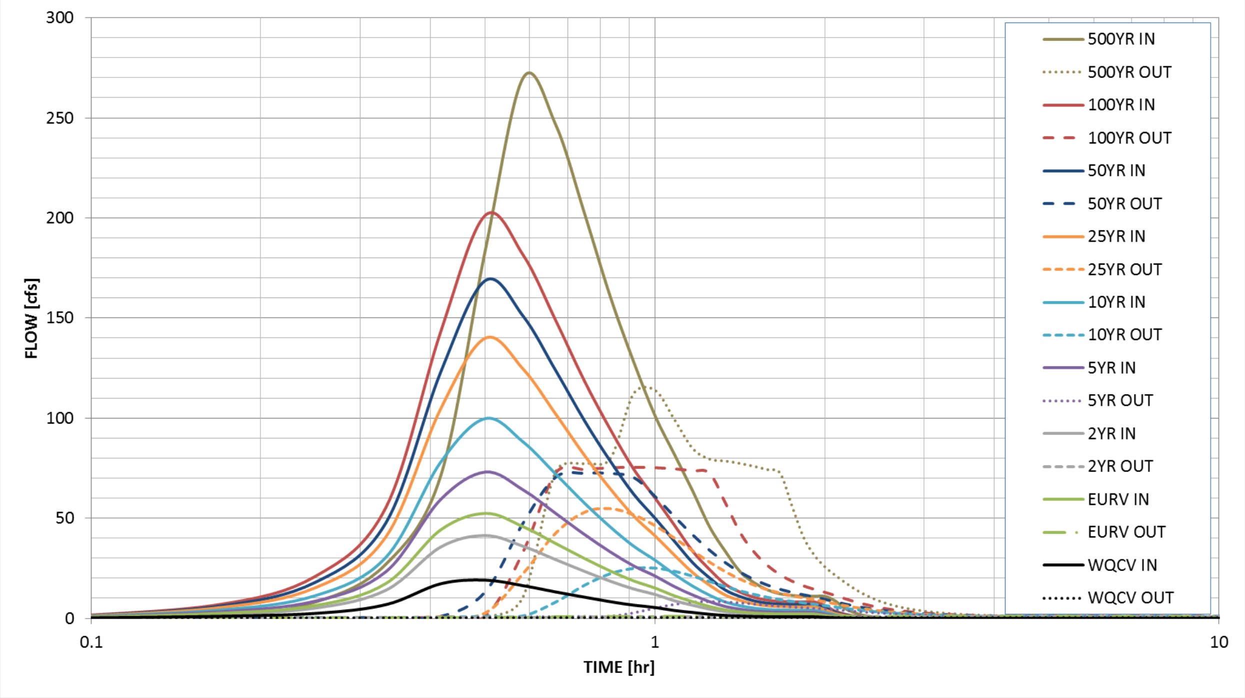


Figure 75. The Outlet Structure Worksheet includes graphs detailing the performance of the stormwater management facility, such as this graph depicting the inflow hydrographs and the resulting detained outflow hydrographs from the facility. Note that the abscissa axis scale is logarithmic.

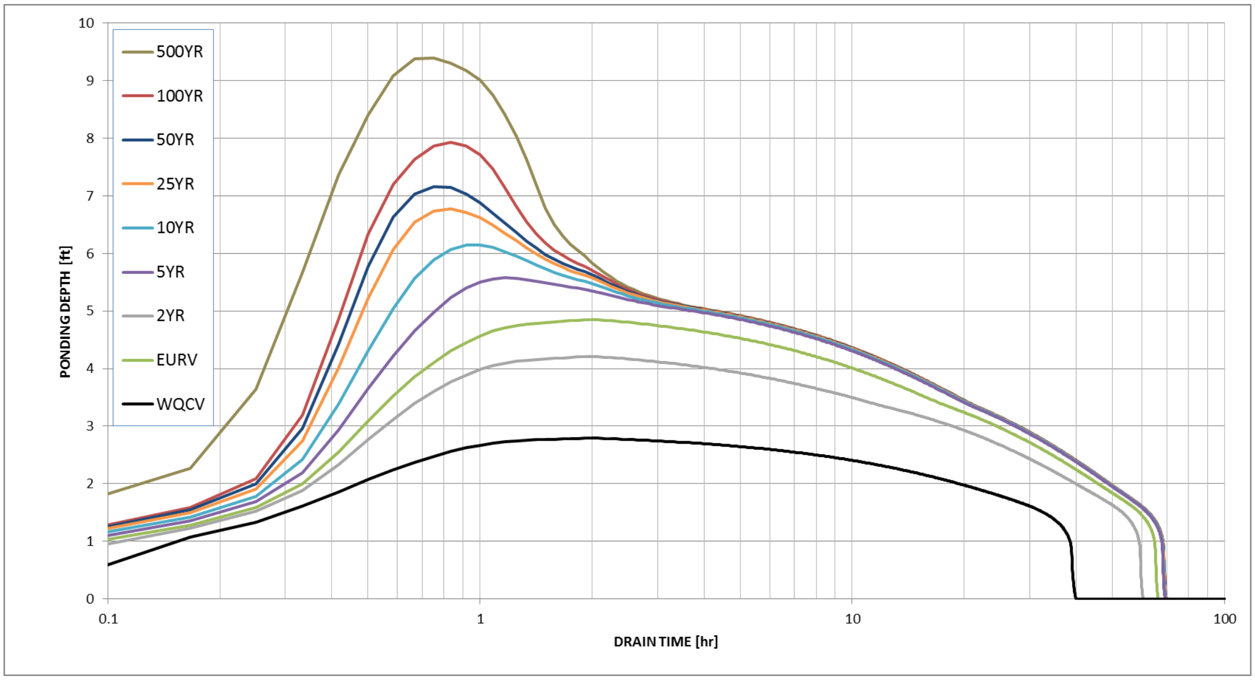


Figure 76. This Outlet Structure Worksheet graph depicts the ponding depth over time in the stormwater management facility for seven recurrence intervals plus the WQCV- and EURV-sized storms. Note that the abscissa axis scale is logarithmic.

Due to the complexity and magnitude of Section 10 (the stage-storage-discharge table), and Section 11 (the Modified Pulse reservoir routing tables), they cannot be shown as Figures in this report. While by default, the figures are hidden in the Outlet Structure Worksheet. The are buttons below Section 8 (the routed results table) can allow the user to click and to make them visible for inspection and/o9r exporting to another application. Figure 75 shows the built-in graphing of the final inflow and outflow hydrographs, while Figure 76 shows the built-in graphing of the stormwater management facility's ponding depth over time—information critical to demonstrate compliance with the new Colorado state statute as described in the next section.

6. RELEVENT NEW STATUTORY REQUIREMENTS

Senate Bill 15-212 was signed into law by Governor Hickenlooper in May 2015 and became effective on August 5, 2015 as Colorado Revised Statute (CRS) §37-92-602 (8). This statute provides legal protection for any regional or individual site stormwater detention and infiltration facility in Colorado, provided the facility meets the following criteria:

1. It is owned or operated by a governmental entity or is subject to oversight by a governmental entity (e.g., required under an MS4 permit).
2. It continuously releases or infiltrates at least 97% of all of the runoff from a rainfall event that is less than or equal to a 5-year storm within 72 hours after the end of the event.
3. It continuously releases or infiltrates as quickly as practicable, but in all cases releases or infiltrates at least 99% of the runoff within 120 hours after the end of events greater than a 5-year storm.
4. It operates passively and does not subject the stormwater runoff to any active treatment process (e.g., coagulation, flocculation, disinfection, etc.).
5. If it is in the Fountain Creek (tributary to the Arkansas River) watershed it must be required by or operated in compliance with an MS4 permit.

The statute specifies that runoff treated in stormwater detention and infiltration facilities shall not be used for any other purpose by the owner/operator/overseer (or that entity's assignees), shall not be released for subsequent diversion or storage by the owner/operator/overseer (or that entity's assignees), and shall not be the basis for a water right or credit.

There are specific notification requirements that apply to all new stormwater detention and infiltration facilities, including individual site facilities built by private parties as a development requirement. For any stormwater detention and infiltration facility constructed after August 5, 2015 and seeking protection under the new statute, the "entity that owns, operates, or has oversight for" shall, prior to operation of the facility, provide notice to all parties on the substitute water supply plan notification email list maintained by the State Engineer. This notice must include the following:

1. The location.
2. The approximate surface area at design volume.
3. Data that demonstrate that the facility has been designed to comply with the release rates described in Items 2 and 3 above.

The Colorado Division of Water Resources (DWR) maintains seven email lists, one for each of the seven major watersheds in Colorado (these coincide with the seven DWR Divisions). UDFCD worked with DWR and the Colorado Stormwater Council to develop a simple data sheet and an online map-based compliance portal website that will allow all municipalities and counties in Colorado to easily upload this required notification information. The website application will then automatically send email notifications to the proper recipients, relieving public works staff of the emailing burden while also minimizing the volume of email going out to the email list recipients.

The notification requirement applies only to new stormwater facilities (constructed after August 5, 2015), which the statute provides a "rebuttable presumption" of non-injury to water rights. This rebuttable presumption is contestable but only by comparison to the runoff that would have been generated from the undeveloped land condition prior to the development necessitating the stormwater facility.

Stormwater facilities in existence before August 5, 2015 are defined in the statute as materially non-injurious to water rights and do not require notification. Additionally, the State issued a memorandum on February 11, 2016 indicating that construction BMPs and non-retention BMPs do not require notice pursuant to SB-212 and are allowed at the discretion of the Division Engineer, and that green roofs are allowable as long as they intercept only precipitation that falls within the perimeter of the vegetated area and do not intercept or consume concentrated flow nor store water below the root zone. The DWR Statement can be found here:
http://water.state.co.us/DWRIPub/Documents/DWR%20Storm%20Water%20Statement.pdf
The compliance portal can be found here:
https://maperture.digitaldataservices.com/gvh/?viewer=cswdif
A tutorial YouTube video and a list of frequently asked questions (FAQs) can also be accessed from that website. UDFCD has worked closely with CDOT's water quality staff toward making this process as streamlined as possible.

7. CONCLUSION

The purpose of this study was to examine alternative outlet designs for extended detention, particularly the concept of the elliptical slot weir. Traditional design of water quality outlets involved orifice plates with small orifices spaced four inches on center vertically. While the four-inch spacing was initially promulgated in the 1999 Urban Storm Drainage Criteria Manual Volume 3, this was intended to be a minimum dimension and not a standard spacing. But as frequently happens with design criteria, what were intended to be minimums become standards. To protect these small orifices, a well screen was recommended with a large open area compared to the sum of all the water quality orifices. Unfortunately, the well screen was not much better with regard to clogging than were the unprotected orifices.

Through the work with ARCADIS and subsequent work at the Colorado State University Hydraulics Laboratory, design parameters and mathematical equations were created, predicting the flow rate through the elliptical slot weir as a function of ponding depth. At CSU, additional qualitative testing was done to demonstrate the debris handling characteristics of the elliptical slot weir. The admittedly subjective observation of this qualitative testing was that the elliptical slot weir handles debris (particularly plastic bags and straw) better than do orifice plates of equal flow capacity.

The elliptical slot weir is very efficient. With a flow pattern characterized by higher flows at greater ponding depths and lower flows at lower ponding depths performed efficiently as compared to the traditional orifice plate. This is hypothesized to result in better sediment (and associated adsorbed pollutants of concern) removal since it matches more closely the sediment gradation-based settling velocities as defined by Stokes Law. However, the demonstration of this hypothesis was not included in the scope of this effort and it would likely take years of intensive water quality sampling to confirm or disprove this.

What this study did determine is that, while the elliptical slot weir drains and handles debris as well or better than does its orifice plate counterpart, it is too efficient for smaller detention basins, oftentimes resulting in a very narrow and clog-prone slot. The qualitative debris handling investigation in the CSU hydraulics laboratory and the two-year field testing at the Northfield and U.S. Postal Service detention basins made it clear that while the elliptical slot weir handles debris very well when the slot is wide (say greater than one inch), debris clogging becomes an issue as the slot grows more narrow. Based on these investigations, UDFCD does not recommend an elliptical slot weir having a slot width of less than 3/8-inch. This equates roughly to a WQCV of one acre-ft or larger, assuming a 40-hour drain time; or an excess urban runoff volume (EURV, refer to the USDCM Volume 3 for details on the EURV concept) of 1.6 acre-ft or larger, assuming a 60-hour drain time. Of course, the depth of the storage volume also plays a significant role in the width of the slot.

Upon making these conclusions, the focus of the study turned to answering the question of how best to gravity drain the water quality volume when the elliptical slot width did not meet the minimum criterion. This resulted in the recommendation of limiting the number of water quality orifices to no more than three. These orifices are to be spaced at stage zero, H/3, and 2H/3, where H is the maximum ponding depth of the EURV or the WQCV. When the number of orifice is limited to three (as opposed to the traditional three per foot of depth), the size of each orifice becomes larger and therefore less prone to clogging. This also facilitates the application of a bar grate instead of a well screen, which is also less prone to clogging.

This study also took advantage of the availability of the CSU and the USBR hydraulics laboratories to evaluate the stage-discharge characteristics of the overflow outlet structure. Previously, the researchers and CDOT had worked with Dr. James Guo of the University of Colorado on a physical modeling study of CDOT Type C and D grated inlets used in highway medians. With this modeling, Guo developed mathematical expressions to define the stage-discharge characteristics of those inlets. Guo’s work was extended to this study as the same grates are commonly used to pass flow through the overflow outlet portion of the detention basin outlet structure. Questions did remain to whether the results from the previous study were truly transferrable to this study so additional work with the USBR confirmed Guo’s previous work with some modifications to the orifice and weir coefficients for grate slopes of zero (horizontal), 3:1 (H:V), and 4:1 (H:V).

In order to standardize the elements learned through this research, the development of new design software was undertaken, including research to:

1. Create a mathematical model of a detention basin,
2. Create equations to approximate runoff volumes and required storage volumes,
3. Create a method to shape inflow hydrographs based on the watershed slope and shape factor.

This work in turn led to the creation of three new design workbooks, namely:

1. SDI-Design-Data,
2. UD-FSD,
3. UD-Detention.

The first workbook, SDI-Design-Data.xlsm, is simply a tool that can be used in conjunction with the new compliance website for Colorado Revised Statute (CRS) 37-92-602(8) to demonstrate compliance with the statute. The second workbook, UD-FSD.xlsm facilitates the design of full spectrum detention basins only. For the design of all other stormwater management facilities, the UD-Detention workbook is a very powerful and easy to use design aid that will help the design engineer complete a preliminary volume sizing and outlet configuration to drain the various recurrence interval inflow hydrographs appropriately. UD-Detention can also be used with a grading plan to complete the final analysis of the performance of the facility.
UDFCD is committed to the maintenance and upkeep of these three design aid workbooks and is currently in the process of creating tutorial videos that will be made freely available at www.udfcd.org.

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Appendix F – Estimating Flow Through a Partially Submerged Vertical Orifice

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TECHNICAL MEMORANDUM

FROM: Ken A. MacKenzie, P.E., UDFCD Master Planning Program Manager

SUBJECT: Estimating Flow through a Partially Submerged Vertical Orifice

DATE: December 31, 2015

In detention basin design, the question has been asked as how to model flow through the 100-year orifice (or through the water quality orifices or any other vertical orifice) when the ponding depth is less than the top of that orifice.

A peer-reviewed technical paper titled *Flow Through Partially Submerged Orifice* has been submitted to the ASCE *Journal of Irrigation and Drainage Engineering* by James CY Guo, Ryan Stitt, and David Mays. While still in draft form, the paper provides a sound mathematical approach to estimating flow through a partially submerged vertical circular orifice.

The reader is encouraged to read *Flow Through Partially Submerged Orifice* by Guo et al. Because of the complexity of setting up the mathematical steps in the approach by Guo et al. however, this technical memorandum suggests a simplified equation that closely matches the results produced by their method.

First, determine the orifice flow resulting when the ponding depth is equal to the top of the circular orifice as:

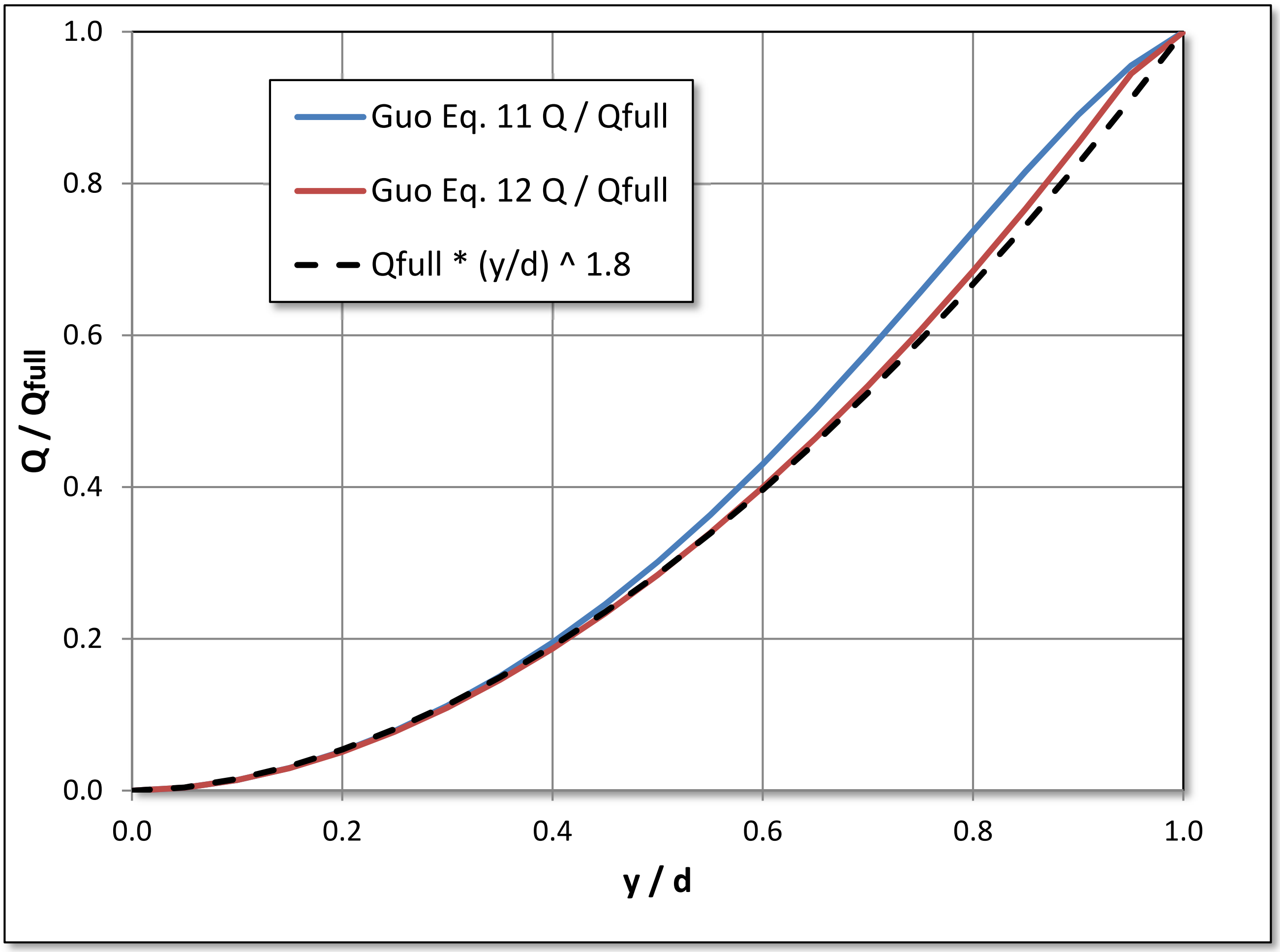
$$Q_{full} = C_d A_o \sqrt{2gd} \tag{1}$$

Where Q_{full} is the orifice flow through a just-full orifice, C_d is the coefficient of discharge (recommended by Guo et al. as 0.53), A_o is the area of the orifice, g is the gravitational constant, and d is the diameter of the orifice (the ponding depth y is in this case equal to d).

Next, calculate the flow through the orifice where the ponding depth y is less than the diameter d as:

$$Q = Q_{full}\left({}^{y}/_{d}\right)^{1.8} \tag{2}$$

The paper by Guo et al. suggests either of two equations (Eq. 11 and Eq. 12) will produce a good approximation of orifice flow, and this was verified in the University of Colorado Hydraulics lab. The results of Equations 1 & 2 in this technical memorandum are compared to Guo's Eq. 11 and Eq. 12 in the following graph.


Guo Eq. 11 Q / Qfull
Guo Eq. 12 Q / Qfull
Qfull * (y/d) ^ 1.8
Q / Qfull
y / d
0.0
0.2
0.4
0.6
0.8
1.0

Appendix G – Flow Through Partially Submerged Orifice

FLOW THROUGH PARTIALLY SUBMERGED ORIFICE

James C.Y. Guo[1], Ryan P. Stitt[2], and David C. Mays[3]

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ABSTRACT

Vertically mounted circular orifices have been extensively used for flow measurements. The operation of a side orifice is not always under a high headwater above the crown of the orifice, as a result, the prediction of partially submerged flow is characterized as a mixing flow between weir and orifice flows. The general equations of weir and orifice need to be tailored to predict partially submerged orifice flows. The purpose of this study is to derive a new method for calculating the discharge flow rate through partially submerged circular orifice. The method applies a weighting factor to construct a reliable rating curve for the transitional flow from a weir flow when the flow depth is shallow to an orifice flow when the flow depth becomes deep. The proposed method is further normalized in form of Froude Number using the diameter of the orifice as the characteristic length and gravitational acceleration as the characteristic time. For engineering practices, the best-fitted orifice discharge coefficient was determined to be 0.53 using the least squared error method.

INTRODUCTION

A side circular orifice is often installed as a flow entrance into an outlet structure. For instance, the entrance of a culvert is treated as an orifice. An orifice is depicted with its cross sectional shape and size. In practice, the performance of a culvert is under either inlet or outlet control, depending on the required headwater depth for a given design flow, whichever higher dictates (HDS5 in 1985). A complete orifice flow only occurs when the crown of the side orifice is submerged under a headwater deeper than at least one orifice diameter (Bos 1989). As a rule of thumb for culvert designs, the headwater to diameter ratio is recommended to be between 1.5 and 2.0 (HDS5 in 1985). Obviously this protocol is not applicable when the operation of a side orifice is under a shallow headwater.

Accurate determination of flow rates through a hydraulic structure is critically important to its operation and maintenance. The latest development in stormwater management has been shifted from runoff flow control for extreme events to runoff volume reduction for all events. Under the concept of green approach, the operation of a detention facility shall be designed to manage all events. Although orifices and outlet structures are often sized to pass the extreme events under a submerged condition, more than 90% of their operations are in fact under partially submerged conditions (Guo and Urbonas 1996). Therefore, how to estimate the small, frequent flows released through partially submerged orifices are urgently important for implementing the green concept into stormwater management. Laboratory experimentations have provided an empirical weir flow equation that calculates flow discharge between the invert and the center of a circular orifice. However, the discharge flows under a headwater depth between the center and the crown remains unclear (Greve 1924). Similarly, investigations on the flow interception capacities for curb-opening inlet and grate inlet have revealed that the transitional process from a shallow-water weir flow to a deep-water orifice is a mixing flow that can be weighted using the geometric means of orifice and weir flows (Guo, MacKenzie, and Mommandi 2009). In this study, a weighting factor was derived using the ratio of headwater to diameter as a basis to estimate the mixing flow. It was confirmed that this weighting method produces good agreements with the observed data for both shallow-water weir flows and deep-water orifice flows. This approach is further normalized to apply to all cases. It is believed that this study improves the understanding of low-flow hydraulics via a circular orifice when the green approach is introduced to managing runoff flows of full spectrum.

PARTIALLY SUBMERGED CIRCULAR ORIFICE FLOW

Studies of orifice flow can be traced back to Evangelista in 1643. A side circular orifice is a vertically installed circular opening on a metal plate which is placed perpendicular to the entrance of a straight culvert or channel (Bos 1989). As recommended, the design condition for a side orifice includes: (a) a headwater depth at the entrance to be at least one diameter over the invert of the orifice, and (b) a full contraction of the discharge jet under the negligible tailwater effects from the downstream channel. Details of submerged orifice hydraulics can be found elsewhere (HDS 5 in 1985). As illustrated in Fig 1, all normalized flow parameters associated with a partially submerged orifice flow are directly related to the diameter and central angle as (Guo 2015):

$$\frac{y}{d} = \frac{1}{2}(1 - \cos\theta) \tag{1}$$

$$\frac{A}{d^2} = \frac{1}{4}(\theta - \sin\theta\cos\theta) \tag{2}$$

$$\frac{R}{d} = \frac{\theta - \sin\theta\cos\theta}{4\theta} \tag{3}$$

$$\frac{T}{d} = \sin\theta \tag{4}$$

$$\frac{Y_h}{d} = \frac{y}{d} - \frac{1}{2} + \frac{2(\sin\theta)^3}{3[2\theta - \sin(2\theta)]} \tag{5}$$

$$Q_o^{\ *} = \frac{Q_o}{d^{2.5}\sqrt{g}} = \sqrt{2}C_d\frac{A}{d^2}\sqrt{\frac{y}{d} - \frac{Y_h}{d}} \tag{6}$$

in which A = flow area in [L^2], d = diameter of pipe in [L], R = hydraulic radius in [L], θ = central angle in radians varied from zero to π shown in Fig. 1, y= flow depth in [L], Q_0 = design discharge in [L^3/T], Q_0^*= normalized orifice flow, g= gravitational acceleration in [L/T^2], and Y_h = depth from water surface to centroid of flow area in [L].

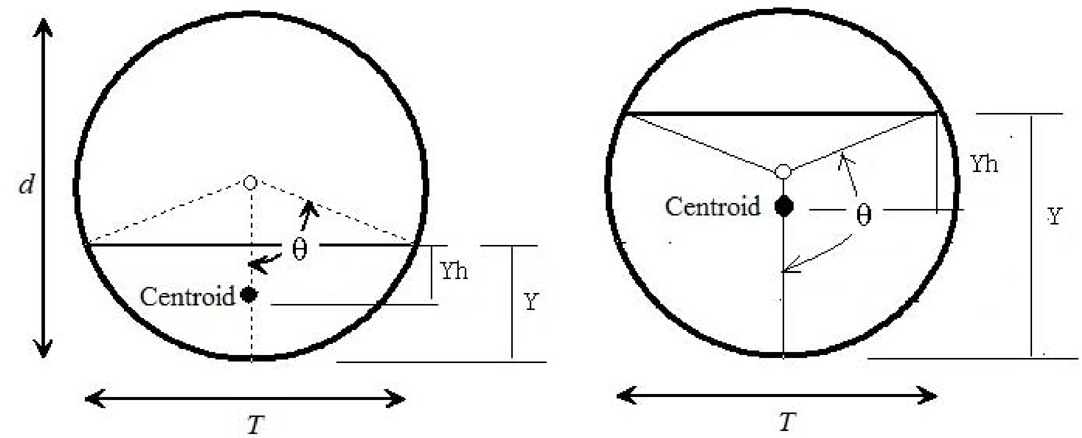

Fig. 1 Partially Submerged Flow in Circular Orifice

Noted that when the orifice is submerged to its crown, y=d, $\theta=\pi$, Y_h=d/2, and T=0, the orifice flow formula is recued to:

$$\frac{Q_o}{d^{2.5}\sqrt{g}} = \frac{\sqrt{2}\pi}{4} C_d \tag{7}$$

Eq (7) agrees with the conventional orifice formula.

PARTIALLY SUBMERGED CIRCULAR WEIR FLOW

Partially submerged weir flows were studied using multiple rectangular slots to represent the circular sharp-crested weir. Flow through each slot was estimated as a rectangular weir flow. The sum of all slotted flows provides the total weir flow (Balachandar et al. 1991), (Brandes et al. in 2013). The analyses of laboratory data collected from partially submerged circular weir flows show that the rating curves appear to be straight lines on a logarithmic graphic paper. Therefore, the correlation between weir flows and shallow headwater depths was derived from the best fitted lines as (Greve 1924, 1932):

$$W_w = 179 d^{0.637} y^{1.87} \tag{8}$$

Where W_w = weight of weir flow rate in [pound/sec], and y = weir flow depth in feet above the invert of the orifice in [L]. In this study, Eq (8) is converted to its normalized form as:

$$Q_w^* = \frac{Q_w}{\sqrt{g}d^{2.5}} = \frac{179}{\gamma_w\sqrt{g}}(\frac{y}{d})^{1.87} = \frac{\frac{2}{3}\gamma_w\sqrt{2g}C_d}{\gamma_w\sqrt{g}}(\frac{y}{d})^{1.87} = \frac{2}{3}\sqrt{2}C_d(\frac{y}{d})^{1.87} \qquad (9)$$

Where Q_w^* = normalized weir flow in form of Froude number, Q_w = weir flow in $[L^3/T]$, γ_w = specific weight of water or 62.4 pounds/ft^3. The value of 179 in Eq (8) is a lumped parameter that includes specific weight of water, discharge coefficient, and a factor of 2/3 in the conventional weir flow formula.

PARTIALLY SUBMERGED CIRCULAR TRANSITIONAL FLOW

Eq 7 is applicable to water depths above the crown of the circular orifice, while Eq 8 is verified for the headwater depths less than the center of the circular orifice. In this study, the flows for the range from y=0 to y=d are formulated as:

$$w = (\frac{y}{d})^2 \qquad (10)$$

$$Q^* = \frac{Q}{\sqrt{g}d^{2.5}} = (1-w)\frac{Q_w}{\sqrt{g}d^{2.5}} + w\frac{Q_o}{\sqrt{g}d^{2.5}} \qquad (11)$$

Where w= weighing factor, Q^*= normalized transitional flow, and Q= transitional flow in $[L^3/T]$. The weighing factor in Eq (10) reflects the flow area ratio which is equivalent to the squared length ratio (Guo, MacKenzie and Mommandi 2009), (Vatankhah 2010).

Table 1 summarizes the detailed calculation for various water depths and flows. Fig. 2 presents the comparison between Eq (11) and the observed data

Table 1 Calculation of Normalized Partially Submerged Circular Orifice Flows

Central Angle degree	Angle radian	Depth y/d	Area A/d^2	Centroid Yh/d	Orifice Qo*	Weir Qw*	Min (Qo*,Qw*)	Weighting w	Factor 1-w	Weighted Q* cfs	Obs Q* cfs
0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.00	1.00	0.00000	0.00000
30.00	0.52	0.067	0.023	0.027	0.004	0.003	0.003	1.00	0.00	0.00331	--
60.00	1.05	0.250	0.154	0.103	0.046	0.039	0.039	0.94	0.06	0.03925	0.03901
90.00	1.57	0.500	0.393	0.212	0.164	0.142	0.142	0.75	0.25	0.14736	--
113.48	1.98	0.699	0.587	0.309	0.285	0.266	0.266	0.51	0.49	0.27512	0.24365
120.00	2.09	0.750	0.632	0.336	0.316	0.303	0.303	0.44	0.56	0.31042	0.32231
150.00	2.62	0.933	0.763	0.447	0.414	0.455	0.414	0.13	0.87	0.41915	--
180.00	3.14	1.000	0.785	0.500	0.432	0.519	0.432	0.00	1.00	0.43197	--

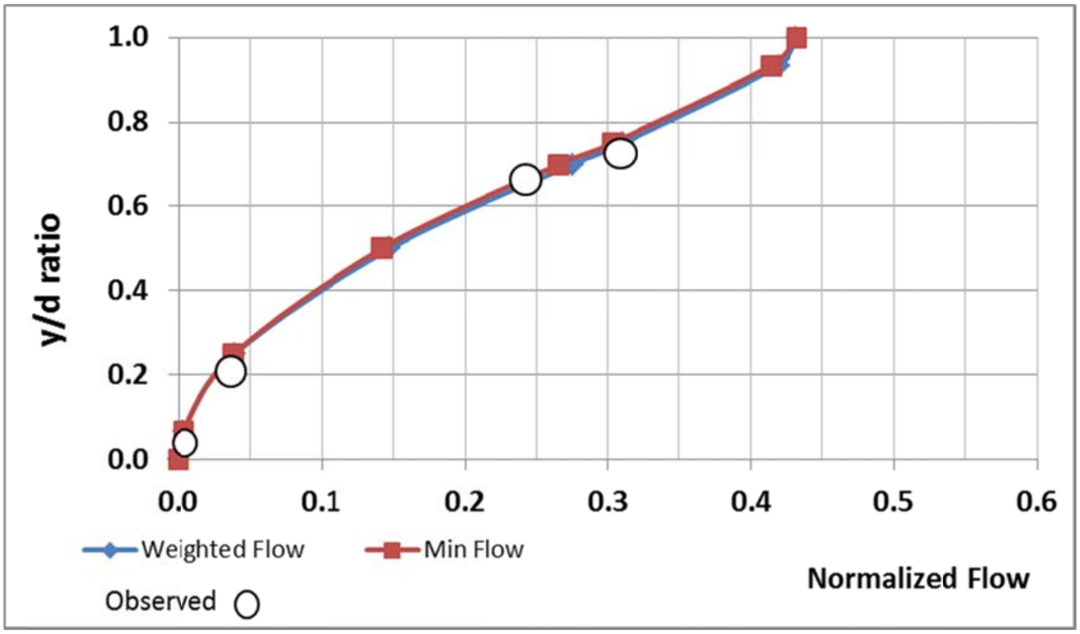

Fig. 2 Normalized Partially Submerged Orifice Flows with Observed Data

The method of least squared error was used to minimize the total squared difference between the predicted and observed flows. The best value of C_d is found to be 0.53 which agreed well with the laboratory data (Greve 1932).

CONCLUSIONS

As a partially submerged flow, both orifice and weir flows are operated in an open flow system which is dictated by the gravitational acceleration. The dimensional analysis performed in this study recommends that Eq (11) be normalized in form of Froude number and the calibration parameter be the discharge coefficient. With the available data sets, the best fitted value for discharge coefficient is found to be 0.53.

As reported (Guo 2006), the transitional flow can be approximated to be the smaller one between weir and orifice flows as:

$$Q^* = \min(Q_w^*, Q_o^*) \qquad (12)$$

Although Eq (12) was not sufficiently verified, in this study, a comparison with the laboratory data was also performed and plotted in Fig. 2. In general, Eq (12) agrees with observed flows well. The best fitted discharge coefficient for Eq (12) is found to be 0.55. In practice, both Eq's (11) and (12) are recommended for engineering designs.

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Appendix H – Physical Modeling of Overflow Outlets for Extended Detention Stormwater Basins

RECLAMATION

Managing Water in the West

Hydraulic Laboratory Technical Memorandum PAP-1105

Physical Modeling of Overflow Outlets for Extended Detention Stormwater Basins

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Hydraulic Investigations and Laboratory Services Group
Denver, Colorado

September 2014

Hydraulic Laboratory Technical Memorandum PAP-1105

Physical Modeling of Overflow Outlets for Extended Detention Stormwater Basins

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U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
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Denver, Colorado

September 2014

Mission Statements

The U.S. Department of the Interior protects America's natural resources and heritage, honors our cultures and tribal communities, and supplies the energy to power our future.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Hydraulic Laboratory and PAP Reports

The Hydraulic Laboratory Report and PAP series is produced by the Bureau of Reclamation's Hydraulic Investigations and Laboratory Services Group (Mail Code 85-846000), PO Box 25007, Denver, Colorado 80225-0007. At the time of publication, this report was also made available online at **http://www.usbr.gov/pmts/hydraulics_lab/pubs/**.

Disclaimer

Funding for this project was provided by Urban Drainage and Flood Control District

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Background

The Urban Drainage and Flood Control District (UDFCD) was established by the Colorado legislature in 1969 for the purpose of assisting local governments in the Denver metropolitan area to address multi-jurisdictional drainage and flood control challenges in order to protect people, property, and the environment. The District covers an area of 1608 square miles and includes Denver, parts of the 6 surrounding counties, and all or parts of 32 incorporated cities and towns. There are about 1600 miles of "major drainage ways" which are defined as draining at least 1000 acres (Urban Drainage, 2014).

The UDFCD provides design guidance on many different types of stormwater and water quality infrastructure that are used throughout the District. One of these structures is the extended detention basin (EDB), which is a sedimentation basin designed to detain stormwater for many hours after the end of storm runoff events. EDBs utilize a small outlet that extends the emptying time of the more frequently occurring runoff events to facilitate pollutant removal and reduce the peak runoff that would enter a storm water system. Figure 1 provides an overview of some of the main features of an EDB (UDFCD, 2010) which include:

- a <u>basin length to width ratio</u> of at least 2:1
- <u>side slopes</u> not steeper than 3:1
- <u>inlet structure</u> that can dissipate flow energy at the concentrated points of inflow
- <u>forebay</u> to allow larger particles to settle quickly
- <u>trickle channel</u> which conveys low flows from the forebay to the micropool
- <u>micropool</u> which creates a small permanent defined pool directly upstream from the basin outlet. The micropool prevents large shallow puddles that produce unwanted mosquito habitat.
- <u>outlet structure</u> (Figure 2) located in the embankment containing water quality orifices, a 10-yr orifice, a sloped weir overflow with trash rack, and a 100-yr orifice downstream from the trash rack.

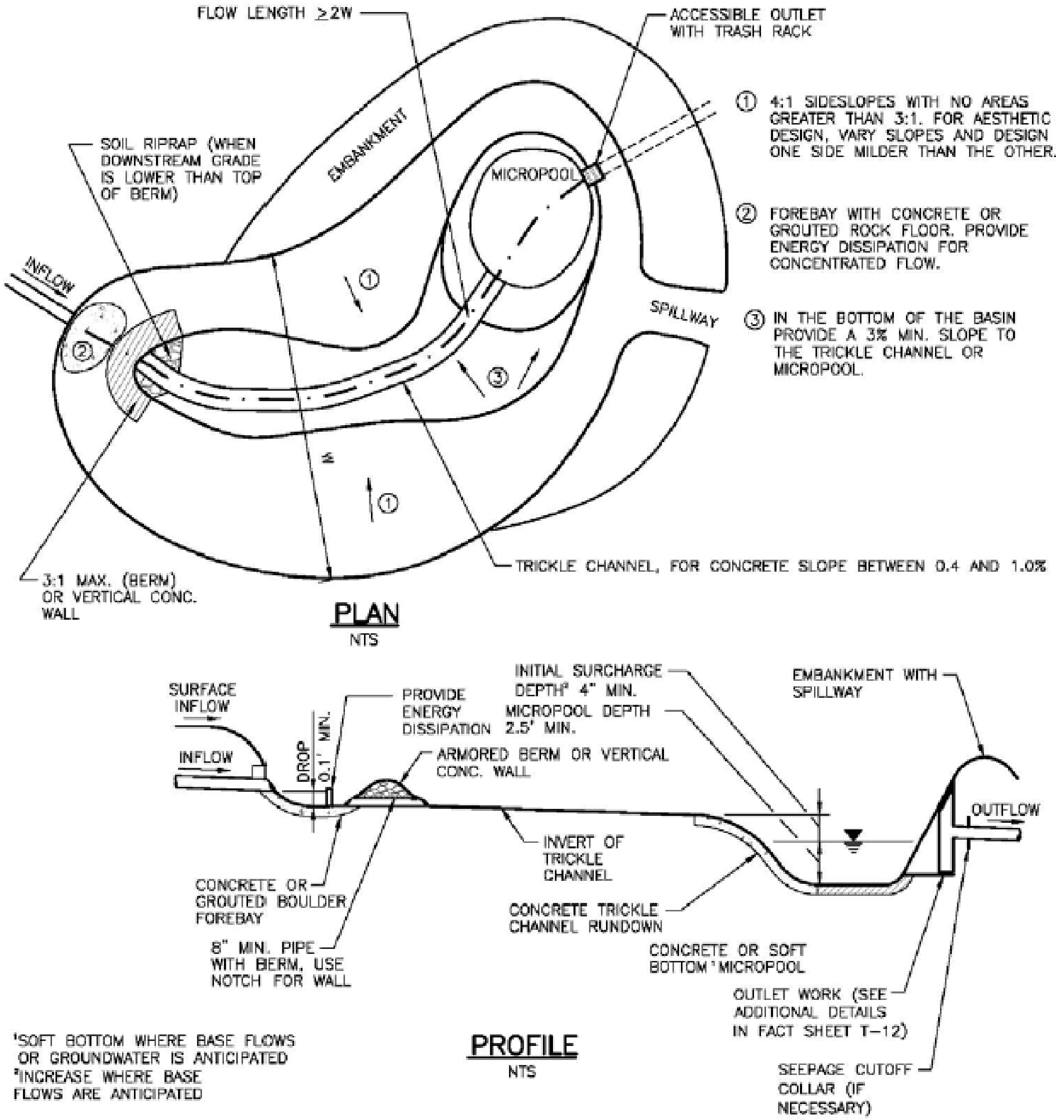

Figure 1 - Basic description of an extended detention basin (EDB) (UDFCD, 2010)

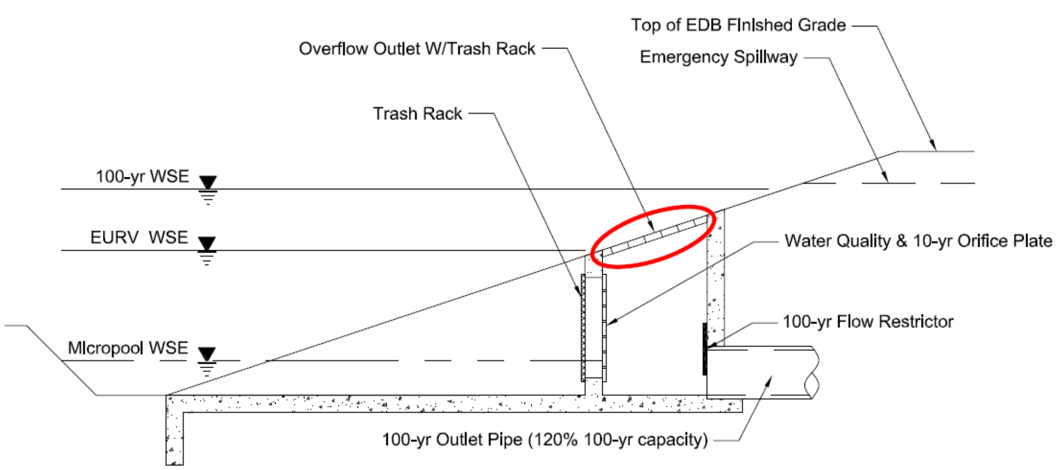

Figure 2 - Typical outlet structure for an extended detention basin (EDB)

The Urban Drainage and Flood Control District contacted the Bureau of Reclamation (Reclamation) in March 2012 to request assistance in resolving some questions regarding the calculation of flow passing through the overflow outlet portion of the outlet structure (circled in red in Figure 2). The flow through the outlet structure is used to regulate storm runoff events through detention basins. An accurate estimate of the flow passing through the overflow outlet portion of the structure will provide better regulation of extreme storm runoff events. UDFCD requested that Reclamation build and test a 1:3 scale physical model of the sloped overflow outlet (not the water quality or 10-yr orifice plate portion) in Reclamation's hydraulics laboratory to determine the head-discharge rating of the structure and evaluate previously developed rating equations.

Previous Work and Provided Information

Dr. James Guo at the University of Colorado Denver campus derived equations to represent flow through the overflow outlet based on a physical model of roadway median inlets (Guo, 2012). Guo collected data from 96 configurations of a 1:3 (model:prototype) scale physical model at the Colorado State University Hydraulics Laboratory. Two types of grates were tested at slopes varying from 0 to 30 degrees. Table 1 provides the equations to calculate flow through the median inlets based on discharge coefficients (C_d) determined from the physical model (Guo, 2012). Variables used in the equations in Table 1 are as follows (see Figure 3):

Q = Flow (ft^3/sec)
C_d = Discharge coefficient (Typically 0.62)
n = Open area ratio for the grate (typically between 0.3 and 0.7)
H = Headwater depth above bottom weir crest
H_b = Depth from bottom weir crest to the top of the upper edge of the grate
B = Bottom weir crest length
L = Horizontal grate length (not parallel to the inclined grate)
θ = Angle of inclined grate

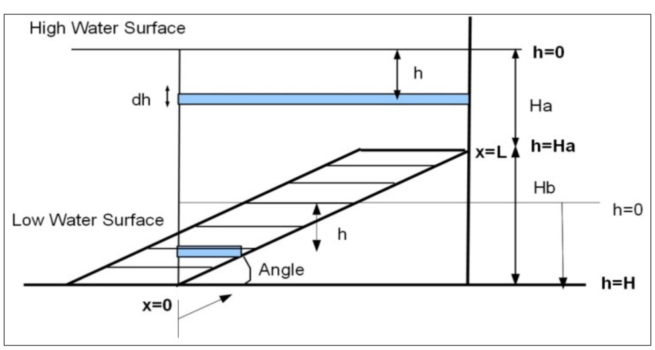

Figure 3 - Diagram of inclined grate with some variables specified (Guo, 2012)

Table 1 - Dr. Guo's equations for calculating discharge through median inlets (Guo, 2012).

Flow Type	Flow Overtopping Two Sides of Inclined Grate	Flow overtopping the Lower Base Width	Condition
Orifice	$Q_o = \frac{2}{3} nC_d BHCot\ \theta\sqrt{2gH} = \frac{2}{3} nC_d BXCos\ \theta\sqrt{2gH}$ Subject to: $X = \frac{H}{Sin\theta} < L$		$H<H_b$ Un-submerged
Weir	$Q_{WS} = \frac{4}{15} nC_d\sqrt{2g}Cot\theta H^{\frac{5}{2}} = \frac{4}{15} nC_d XCos\theta\sqrt{2g}H^{\frac{3}{2}}$ subject to: $X = \frac{H}{Sin\theta} < L$ $Q_W = 2Q_{WS} + Q_{WB}$	$Q_{WB} = \frac{2}{3} nC_d\sqrt{2g}BH^{3/2}$	$H<H_b$ Un-submerged
Orifice	$Q_o = \frac{2}{3} nC_d BLCos\theta\sqrt{2gH}[\frac{H^{\frac{3}{2}}}{H_b\sqrt{H}} - \frac{(H-H_b)^{\frac{3}{2}}}{H_b\sqrt{H}}]$ In case of θ=0 and Hb=0, then $Q_o = \frac{2}{3} nC_d BL\sqrt{2gH}\ if\ \theta = 0$		$H \geq H_b$ Submerged
Weir	$Q_{WS} = \frac{4}{15} nC_d\sqrt{2g}LCos\theta H^{\frac{3}{2}}[\frac{H^{\frac{5}{2}}}{H^{\frac{3}{2}}H_b} - \frac{(H-H_b)^{\frac{5}{2}}}{H^{\frac{3}{2}}H_b}]$ In case of θ=0 and Hb=0, then $Q_{WS} = \frac{2}{3} nC_d L\sqrt{2g}H^{\frac{3}{2}}$ $Q_W = 2Q_{WS} + Q_{WB}$	$Q_{WB} = \frac{2}{3} nC_d\sqrt{2g}BH^{3/2}$	$H \geq H_b$ Submerged

Jim Wulliman from Muller Engineering developed the equations in Table 2 for calculating flow through the inclined grate by deriving weir equations across a side sloping weir. Variables used in the equations contained in Table 2 are as follows:

Q = Flow (ft^3/sec)
C_w = Weir Coefficient (Muller used 2.8)
n = Open area ratio for the grate (typically between 0.3 and 0.7)
H = Headwater depth above bottom weir crest
H_b = Depth from bottom weir crest to the top of the upper edge of the grate
B = Bottom weir crest length
L = Horizontal grate length (not parallel to the inclined grate)
Z = Side slope (Z:1 = H:V)

Table 2- Equations developed by Jim Wulliman from Muller Engineering for an inclined weir

Flow Type	Two Sides of Grate	Lower Base and Top of Grate
Un-Submerged Weir ($H < H_b$)	$Q_{WS} = \frac{2}{5} C_w Z n \left(H^{\frac{5}{2}} \right)$ $\boldsymbol{Q_W = 2Q_{WS} + Q_{WB}}$	$Q_{BW} = C_w B n \left(H^{\frac{3}{2}} \right)$
Submerged Weir ($H \geq H_b$)	$Q_{WS} = \frac{2}{5} C_w Z n \left(H^{\frac{5}{2}} - (H - H_b)^{\frac{5}{2}} \right)$ $\boldsymbol{Q_W = Q_{WB} + 2Q_{WS} + Q_{TOP}}$	$Q_{WB} = \frac{2}{3} n C_w \sqrt{2g} B H^{\frac{3}{2}}$ $Q_{TOP} = C_w B n (H - H_b)^{\frac{3}{2}}$

ARCADIS Engineering performed an analysis of flow through the overflow outlet using computational fluid dynamics (CFD) modeling (Figure 4). They modeled the structure with a 3:1 (Horizontal:Vertical) slope and did not include any reduction for grate clogging.

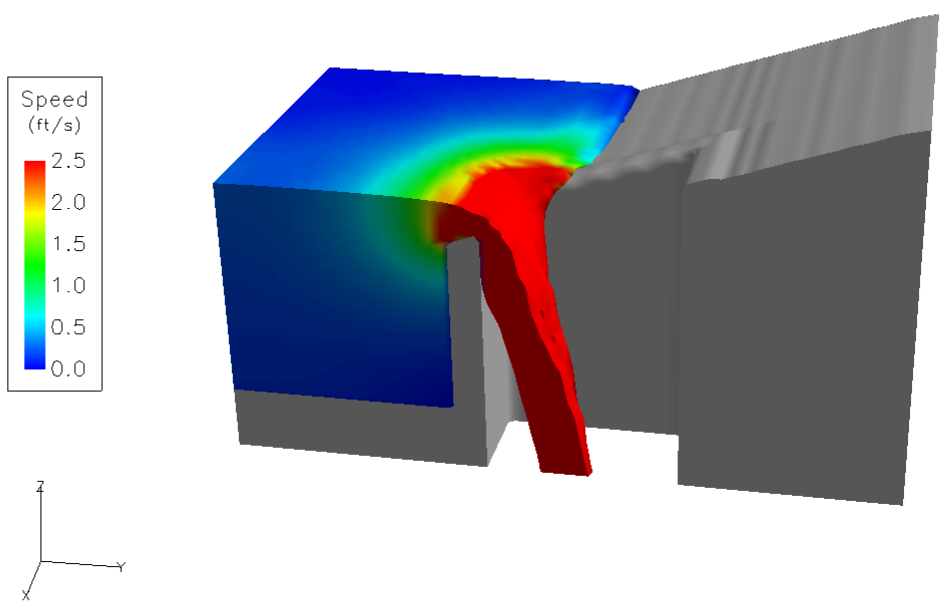

Figure 4 - ARCADIS Engineering CFD model of a 3:1 sloped overflow outlet structure

Figure 5 compares each of the previously mentioned equations and methods to each other. No two methods align very well across the full spectrum. Due to the large disagreement between each of the methods, UDFCD requested that Reclamation conduct a 1:3 scale physical model study to determine which equation best represents the flow through the overflow outlet structure.

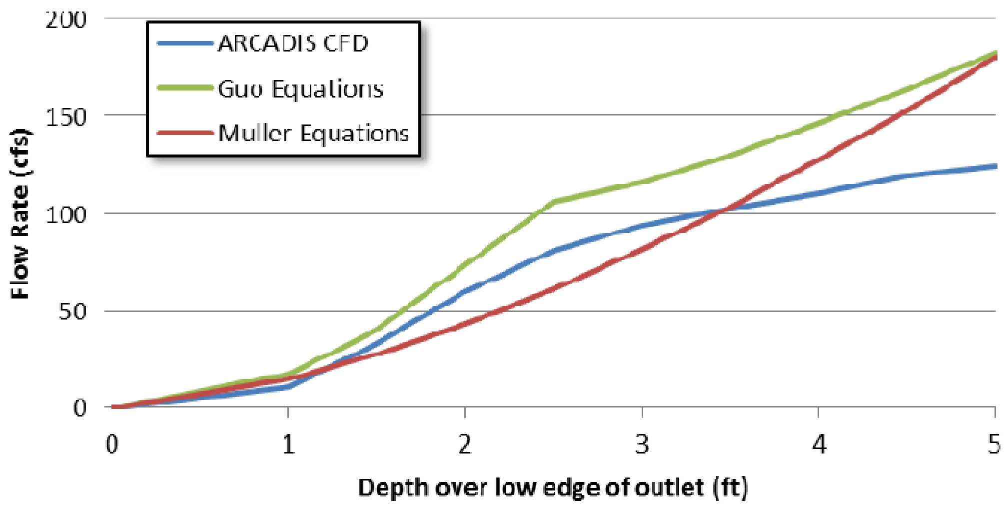

Figure 5 - Comparison of the three different methods to calculate flow through an overflow outlet structure with a 3:1 (H:V) slope (no reduction for grating or debris)

MODEL SETUP

The physical model was constructed in the Bureau of Reclamation's Hydraulics Laboratory in Denver CO, USA. A model box approximately 25-ft wide, 45-ft long and 4-ft deep was configured to simulate an extended detention basin (EDB) (Figure 6). One end of the box contained a 12-in. diameter inlet pipe and a 6-in. thick rock baffle to evenly distribute the flow entering the model. The opposite end of the box contained several configurations of the overflow outlet structure with and without grating.

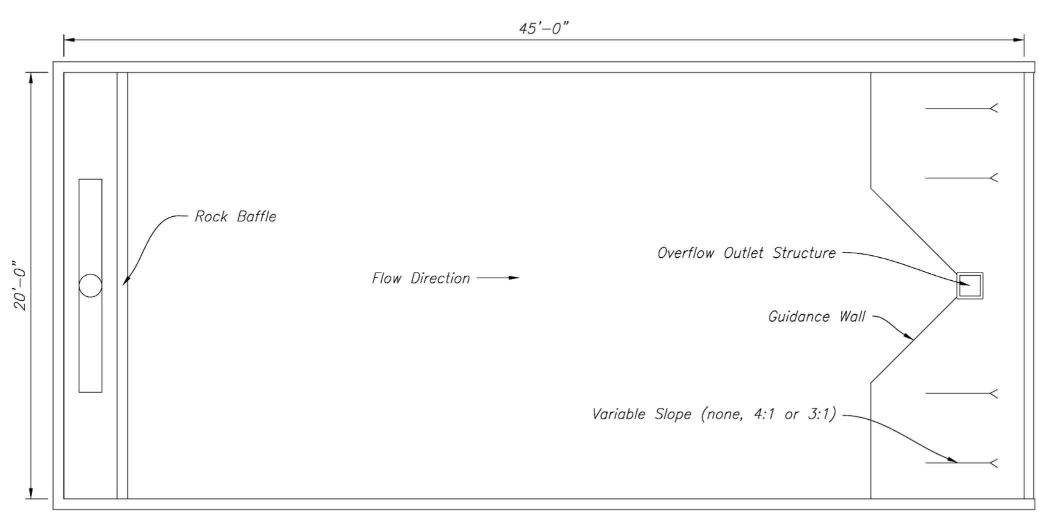

Figure 6 - Physical model layout of an extended detention basin (EDB) (model scale)

The outlet structure was modeled at a geometric scale of 1:3, which means model dimensions are one-third of the prototype dimensions. Since hydraulic performance for open channel flow depends primarily on gravitational and inertial forces, Froude law scaling was used to establish a relationship between the model and prototype. Froude law scaling causes the ratio of gravitational to inertial forces to be equal in the model and prototype; stated in another way, the Froude numbers of the model and prototype are kept equal to one another. Froude law similitude produces the following relationships between model (m) and prototype (p):

Length Ratio: $L_r = L_m/L_p = 1{:}3$
Velocity Ratio: $V_r = V_m/V_p = L_r^{1/2} = 1{:}1.732$
Discharge Ratio: $Q_r = Q_m/Q_p = L_r^{5/2} = 1{:}15.59$

Three different grates were tested (Colorado Department of Transportation Standard Plan No. M-604-10): a Standard CDOT Type C (Figure 7) grate which is approximately 40.5-in. by 26.75-in. with four 2.67-in. wide members on 8-in. centers creating an open area of 68.6 percent, a CDOT close-mesh (Figure 8) grate which is approximately 40.4-in. by 33.5-in. with 0.375-in. wide members on 2.375-in. centers creating an open area of 79.8 percent, and None (Figure 9) or no grate which is a rectangular opening approximately 41-in. by 35-in. and has a 3-in. lip on two edges to hold each grate in position. Each grate was tested at slopes of 3:1 (H:V)(Figure 10), 4:1 (Figure 11), and 1:0 horizontal (no slope).

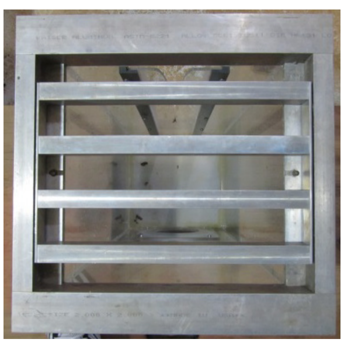

Figure 7 - Plan view of CDOT Type C grate

Figure 8 - Plan view of CDOT close-mesh grate

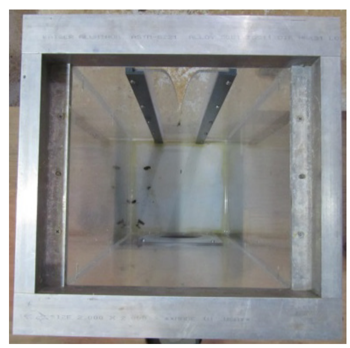

Figure 9 - Plan view of no grate

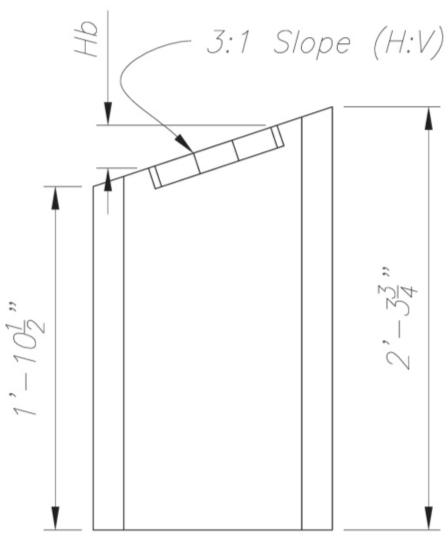

Figure 10 - 3:1 sloped weir box with grate H_b= 0.307 ft (model scale)

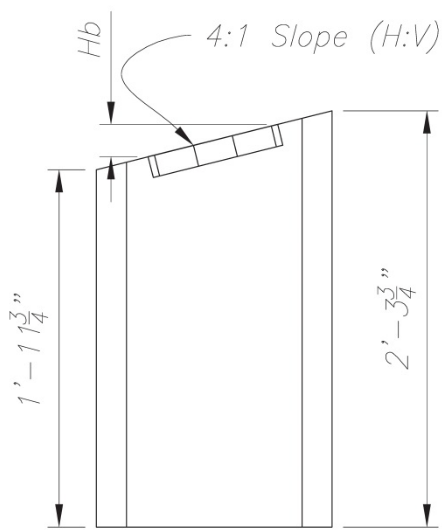

Figure 11 - 4:1 sloped weir box with grate H_b= 0.236 ft (model scale)

Table 3 contains a summary of the test configurations modeled and indicates where surrounding topography was set at the same slope as the overflow outlet structure and grate (Figure 12).

Table 3 - Summary of test configurations that were modeled

Slope	**Grate**	**Surrounding Topography**
3:1 (H:V)	Standard CDOT Type C	YES
3:1 (H:V)	CDOT Close Mesh	YES
3:1 (H:V)	None	YES
4:1 (H:V)	Standard CDOT Type C	YES
4:1 (H:V)	CDOT Close Mesh	YES
4:1 (H:V)	None	YES
None	Standard CDOT Type C	NO
None	CDOT Close Mesh	NO
None	None	NO

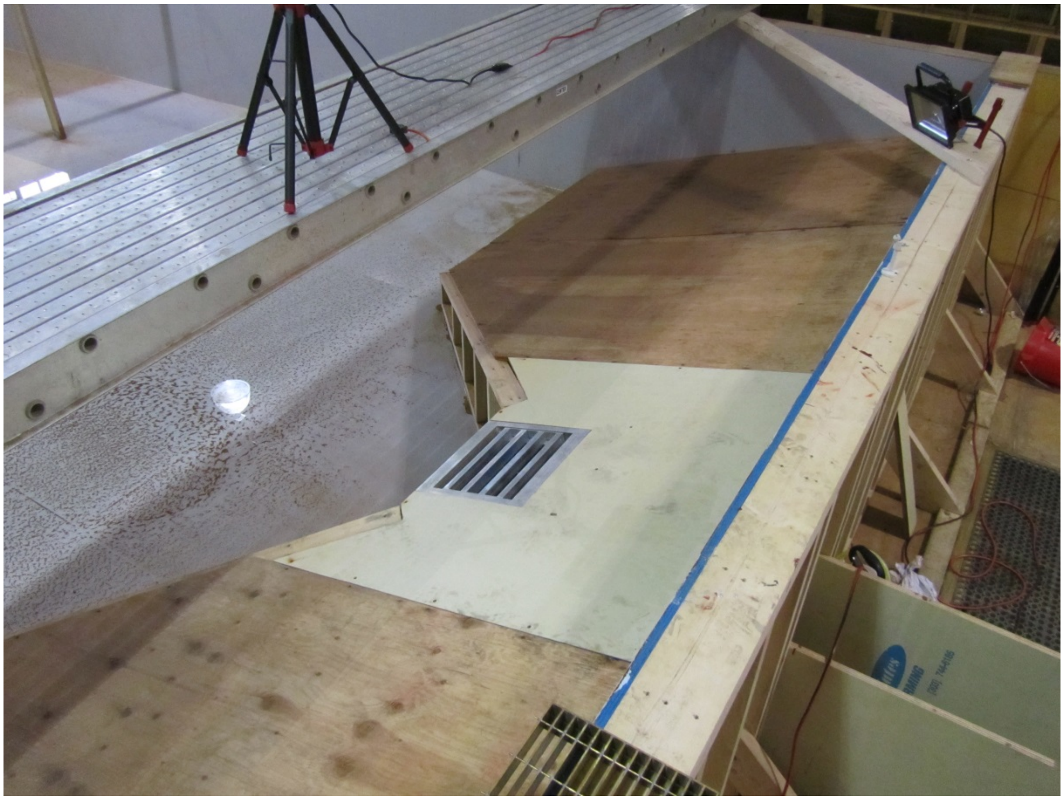

Figure 12 - 3:1 (H:V) slope showing the surrounding topography set at the same slope as the inlet grate

Most test configurations modeled the flow passing through the overflow outlet portion of the outlet works. One final configuration was modeled that tested no slope with no topography and included a complete outlet structure with micropool (Figure 13), water quality orifice plate and 100-yr orifice (Figure 14) restricting flow downstream of the overflow outlet. The water quality orifice plate was modeled as both the standard configuration with a series of orifice holes and as an alternative elliptical weir (Figure 15).

Figure 13 - Complete outlet structure including micropool, water quality orifice plate, horizontal overflow outlet and 100 year controlling orifice

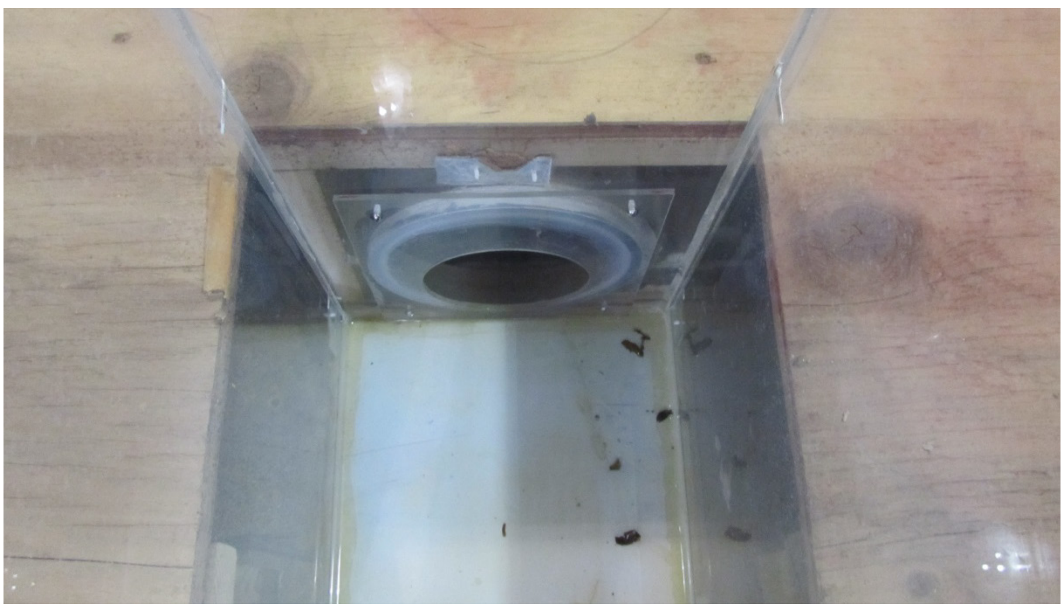

Figure 14 - 100 year controlling outlet orifice (inside outlet structure downstream of overflow)

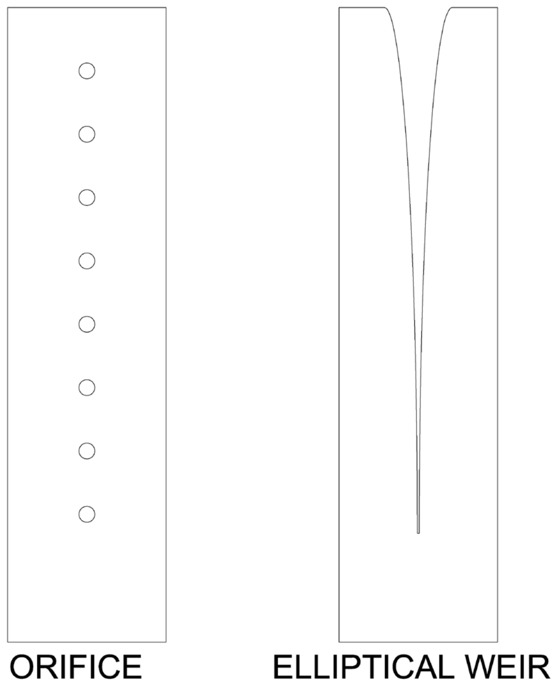

Figure 15 - Water quality orifice plate configurations tested in the complete EDB model

Test Procedure

Each model configuration was tested by completing the following steps:

1. Establish a specific flow rate measured by a calibrated venturi meter accurate to ±0.25 percent (USBR 1989) into the model box.
2. Allow the flow to stabilize for the necessary amount of time so that no change in water surface in the EDB is noticed for at least 5 minutes.
3. Obtain the water surface elevation (stage) above the lower edge of the inlet using both a calibrated laboratory ultrasonic sensor and a point gauge (redundant measurements for consistency).
4. Record both the stage and flow.
5. Repeat steps 1-4 to create a complete rating curve that identifies any transitions between weir and orifice flow.

Inflow and stage were recorded and plotted to generate stage-discharge relationships for each configuration. Collected data were then compared to the provided rating equations in Table 1 and Table 2.

RESULTS

All results presented in this section are reported in prototype dimensions. Figure 17 shows data collected at the 1:0 (H:V) (no slope) configuration for each of the three tested grates. Figure 18 shows data collected at the 4:1 (H:V) slope configuration for each of the three tested grates. Figure 18 shows data collected at the 3:1 (H:V) slope configuration for each of the three tested grates. Each figure plots stage above the lowest edge of the overflow outlet structure in ft on the x-axis and discharge through the overflow outlet in ft^3/sec on the y-axis.

Figure 19 provides data collected on the complete EDB with micropool, water quality orifice, horizontal overflow outlet and 100-year controlling orifice. This plot also shows stage (ft) above the lowest edge of the overflow outlet structure on the x-axis and discharge through the overflow outlet in ft^3/sec on the y-axis. All three grates were tested with a series of orifice holes in the water quality plate. One test was conducted with the orifice holes being replaced with an elliptical weir which releases a significantly larger discharge for a given head.

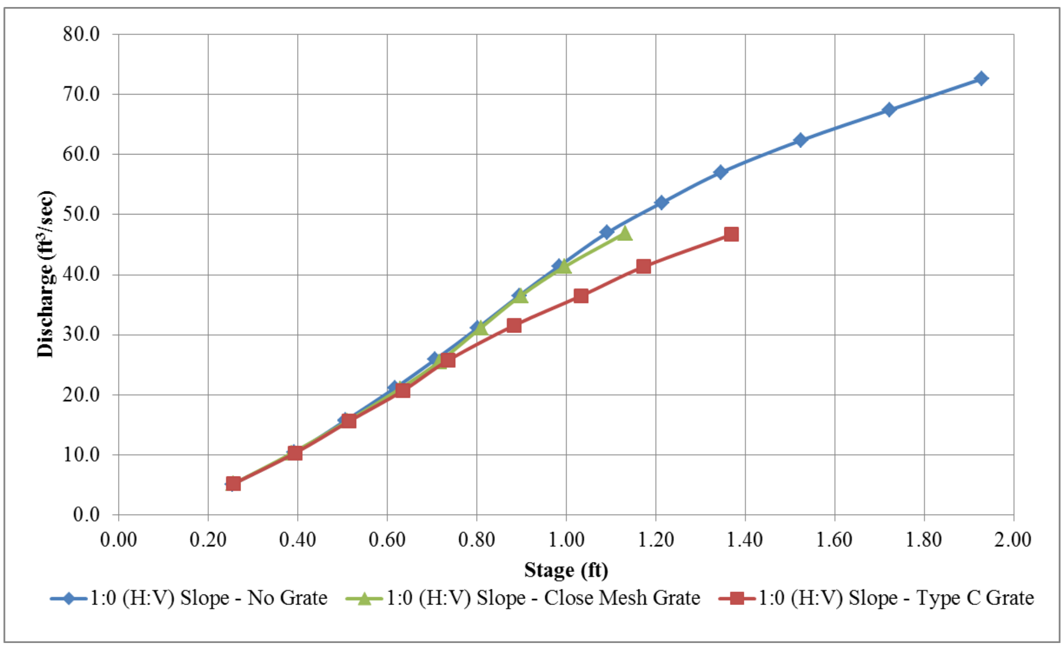

Figure 16 - Data collected in the 1:0 (H:V) slope configuration for each grate (prototype dimensions)

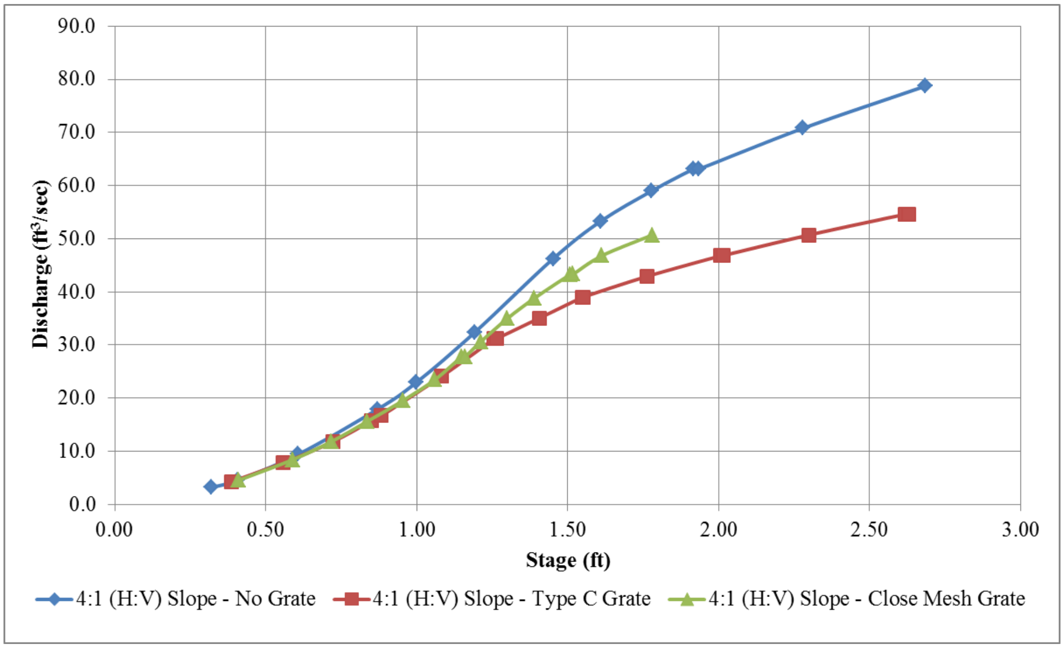

Figure 17 - Data collected in the 4:1 (H:V) slope configuration for each grate (prototype dimensions)

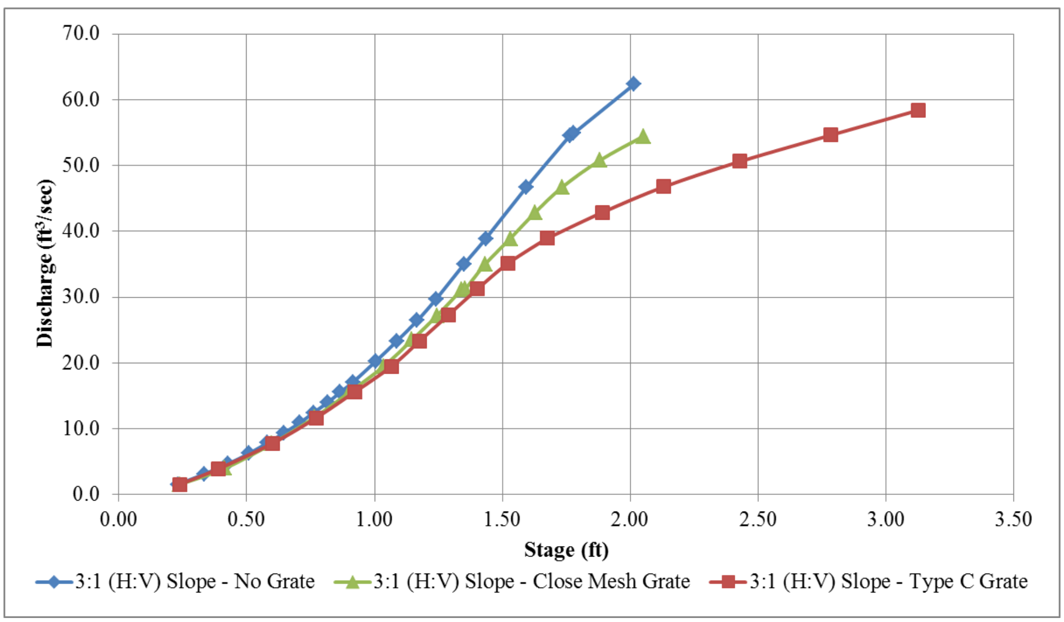

Figure 18 - Data collected in the 3:1 (H:V) slope configuration for each grate (prototype dimensions)

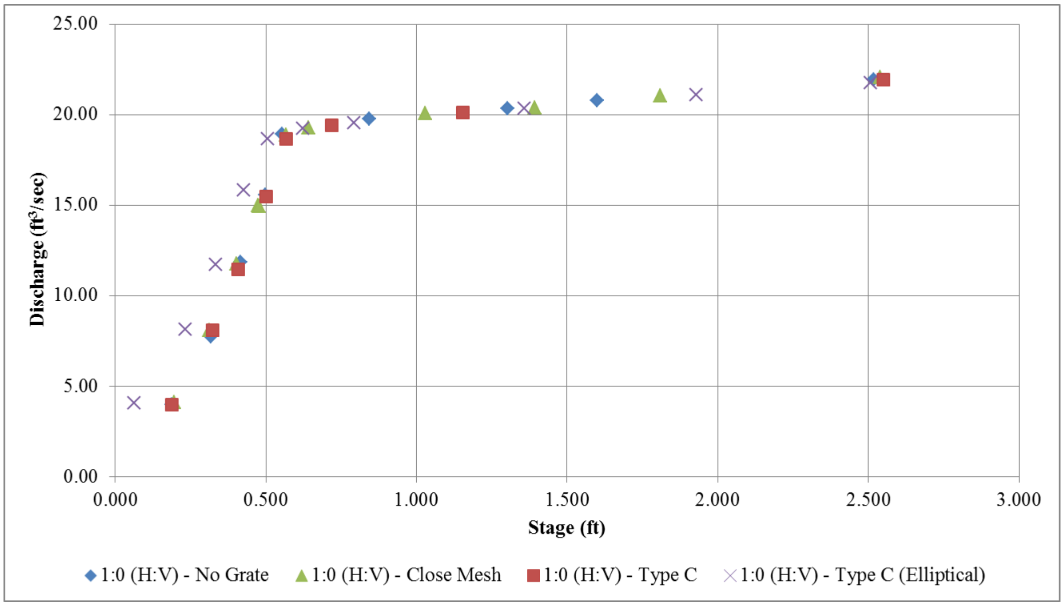

Figure 19 - Data collected on the complete EDB with micropool and 1:0 (H:V) slope overflow outlet structure. Water quality plates and the 100-year controlling orifice were installed for each configuration tested.

ANALYSIS

Each scenario was compared to the equations provided in Table 1 and Table 2 to determine if any of the equations generated rating curves consistent with the physical model. Figure 20 provides a sample plot with all of the equations and the laboratory data. These plots were created for each of the model configurations. Only one plot (4:1 (H:V) slope with a Type C grate installed) is presented in this report to give a representative sample of the data comparisons. Minor differences between generated plots occurred, but all looked similar, with inconsistencies existing between the model data and computed equations.

Two lines to pay particular attention to in Figure 20 are the “Model Data” and the “UDFCD ss” lines. The “Model Data” line is the model data taken in the laboratory. The “UDFCD ss” line is the set of equations adopted by the UDFCD for design purposes, which uses simple logic to determine which flow regime the overflow outlet structure is in and then uses the respective equations developed by Guo to calculate the flow. Table 4 shows tabulated values from Figure 20. The absolute difference was calculated by subtracting the model value from the equation value and the percent difference was determined by dividing the absolute difference by the model value and multiplying by 100. Equation values ranged from -26% to 493% different from the model data, and this was typical across all configurations tested. Values of NA in the table were not calculated because the equations were unable to calculate flows at those stages.

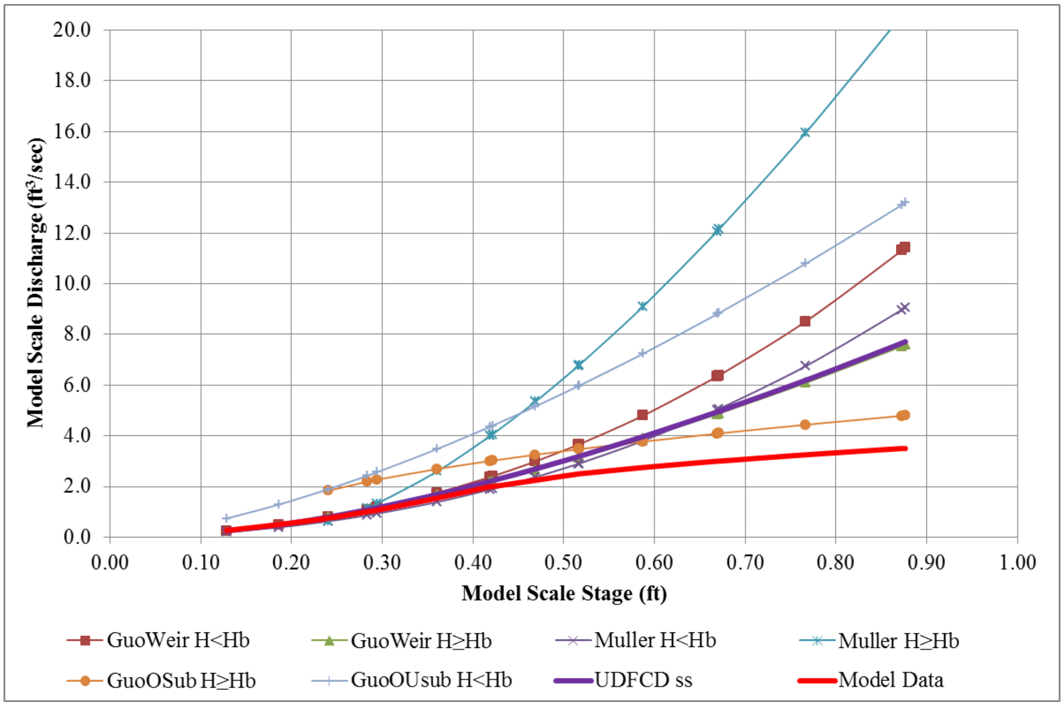

Figure 20 - 4:1 (H:V) slope with Type C grate model data compared to all equations presented in Tables 1 and 2 calculated using the same configuration information.

Table 4 - Data comparison between equation and model data presented in absolute difference [equation data - model data] and percent difference [absolute difference/model data X 100].

GuoWeir $_{H<Hb}$	GuoWeir $_{H\geq Hb}$	Muller $_{H<Hb}$	Muller $_{H\geq Hb}$	GuoOSub $_{H\geq Hb}$	GuoOUsub $_{H<Hb}$	UDFCD $_{ss}$
-0.02 (-7%)	NA	-0.07 (-26%)	NA	NA	0.47 (174%)	-0.02 (-7%)
-0.01 (-2%)	NA	-0.11 (-22%)	NA	NA	0.79 (158%)	-0.01 (-2%)
0.04 (6%)	0.03 (4%)	-0.12 (-16%)	-0.11 (-15%)	1.08 (144%)	1.14 (152%)	0.04 (6%)
0.08 (8%)	0.06 (6%)	-0.14 (-14%)	0.14 (14%)	1.18 (118%)	1.42 (141%)	0.08 (8%)
0.1 (10%)	0.08 (7%)	-0.14 (-13%)	0.26 (24%)	1.2 (112%)	1.5 (140%)	0.1 (9%)
0.21 (14%)	0.12 (8%)	-0.15 (-10%)	1.05 (68%)	1.15 (74%)	1.94 (126%)	0.15 (10%)
0.38 (19%)	0.18 (9%)	-0.12 (-6%)	1.99 (100%)	1.01 (51%)	2.36 (118%)	0.21 (11%)
0.4 (20%)	0.2 (10%)	-0.09 (-5%)	2.06 (103%)	1.02 (51%)	2.4 (120%)	0.24 (12%)
0.74 (33%)	0.41 (18%)	0.12 (6%)	3.1 (138%)	1.01 (45%)	2.92 (130%)	0.45 (20%)
1.15 (46%)	0.63 (25%)	0.4 (16%)	4.26 (170%)	0.97 (39%)	3.46 (138%)	0.68 (27%)
1.16 (46%)	0.64 (26%)	0.4 (16%)	4.29 (171%)	0.97 (39%)	3.48 (139%)	0.69 (28%)
2.05 (74%)	1.15 (42%)	1.05 (38%)	6.35 (231%)	1.02 (37%)	4.49 (163%)	1.22 (44%)
3.33 (111%)	1.86 (62%)	2.02 (67%)	9.06 (302%)	1.09 (36%)	5.8 (193%)	1.94 (65%)
3.38 (113%)	1.89 (63%)	2.06 (69%)	9.15 (305%)	1.1 (36%)	5.85 (195%)	1.97 (66%)
5.26 (162%)	2.84 (87%)	3.5 (107%)	12.7 (391%)	1.19 (36%)	7.54 (232%)	2.94 (90%)
7.81 (223%)	4.02 (115%)	5.47 (156%)	17.08 (488%)	1.29 (37%)	9.61 (274%)	4.15 (119%)
7.92 (226%)	4.08 (117%)	5.56 (159%)	17.26 (493%)	1.3 (37%)	9.7 (277%)	4.21 (120%)

The shape of the head-discharge curve observed in the model makes it apparent that flow control varies from weir flow at low heads to transitional (mixed flow) at intermediate heads, and finally orifice flow at high heads. Approximate bounds of these zones are illustrated in Figure 21. Zones will change slightly depending on the geometry and configuration of the outlet structure and overflow weir.

When flows were in the mixed flow zone they became unstable and the stage in the EDB would fluctuate significantly with a constant inflow. Figure 22 shows this phenomenon, which was present at all configurations. Data was collected for each configuration until the stage oscillations were noticed. As can be seen in Figure 16 through Figure 18 oscillations occurred at different head and discharge for each configuration.

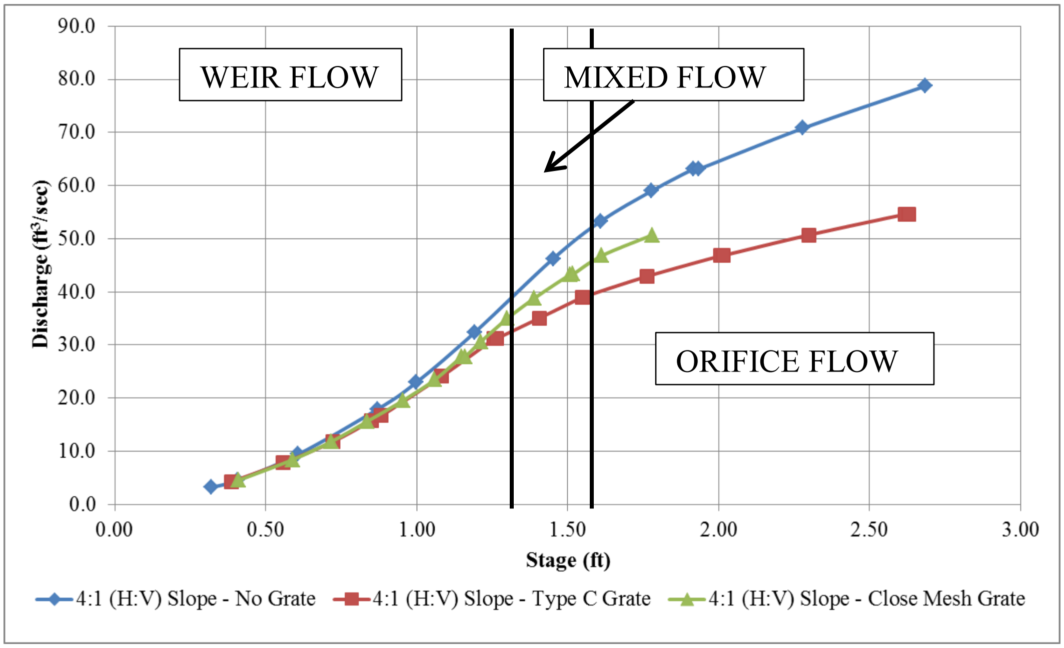

Figure 21 - Approximate boundary zones for weir flow, mixed flow and orifice flow

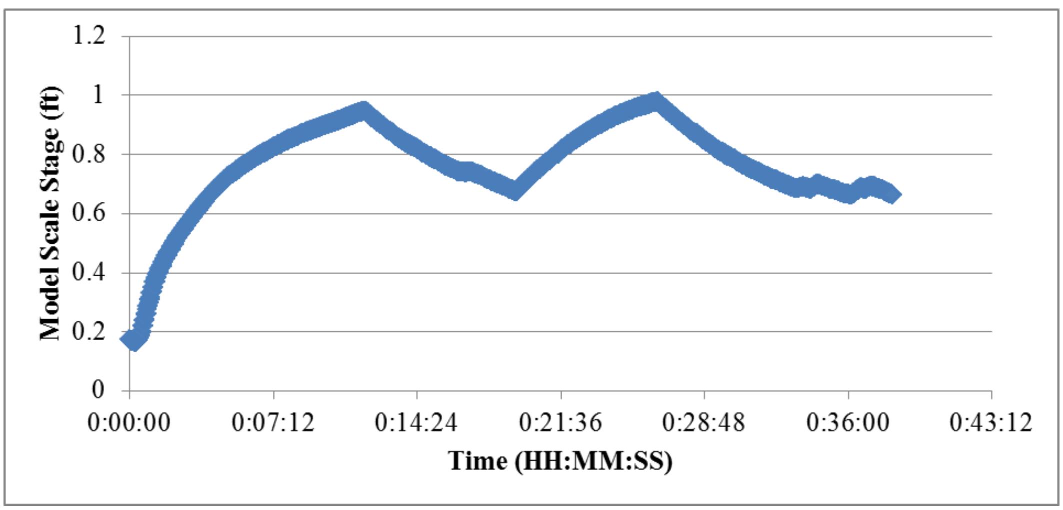

Figure 22 - Sample flow oscillations that occurred when flows entered mixed zone for the 4:1 case with standard type c grate.

Reclamation analyzed the data to determine if a single new equation or set of equations of consistent form could be generated that would accurately describe the flow through the overflow outlet works for all structure configurations. Reclamation plotted the data in TableCurve 2D and TableCurve 3D utilizing different dependent and independent variables. No single relationship was found that accurately described the overflow outlet discharge for all configurations tested. Reclamation determined that it would be difficult to develop a new equation that would accurately describe the flow through the overflow outlet in all zones (weir, mixed and orifice) for any slope, especially with the limited amount of data that was collected during this modeling effort. If more slopes and flows were tested it may be possible to generate a more uniform equation.

Reclamation determined that calculating the discharge through the overflow outlet in all three zones (weir, mixed and orifice) was unnecessary from a practical perspective, because when installed, the outlet works is required to have a 100-yr orifice that restricts the flow through the overflow outlet and prevents the outlet from ever functioning as the flow control in the transitional or orifice mode. After discussing this with UDFCD it was determined that modeling a complete EDB would verify how the 100-yr orifice controls the flow. As shown in Figure 19, the complete model of the EDB confirmed that flow would be restricted by the 100-yr orifice prior to the overflow outlet entering the mixed flow or orifice flow zones; the overflow outlet is in the weir flow zone for the entire range in which it controls the flow.

The 100-yr orifice installed downstream of the overflow outlet performs several valuable functions for the EDB. First, the flow rate from the EDB must be limited to the 100-yr flow so that piping systems downstream of the EDB outlet are not overwhelmed. Second, the 100-yr orifice makes calculating the flow from the overflow outlet less complicated because the flow would remain primarily in the weir flow zone. Discharge calculations from the EDB would transfer to using the 100-yr orifice before utilizing the overflow outlet as an orifice. Third, the 100-yr orifice would prevent the overflow outlet from reaching an unstable oscillating water surface with associated unstable outflows that could not be accurately calculated from the EDB stage.

Flows entering the outlet structure become very turbulent between the overflow outlet and the 100-yr orifice. Reclamation questioned if using a standard orifice discharge coefficient of 0.61 would yield accurate discharge calculations from the 100-yr orifice. Data from the physical model were used to determine that the coefficient in the model was 0.60. When calculating flow from the 100-yr orifice, head relative to the center of the orifice was used.

When calculating flow through an overflow outlet, UDFCD was utilizing a clogging factor which was a reduction factor to represent typical clogging plus the reduction in area caused by the grates. Reclamation determined that it would be more appropriate to use a discharge coefficient to account for the reduction in flow caused by the grate and have a separate clogging factor to account for debris clogging. By creating custom discharge coefficients from the physical model data for each grate and slope, Reclamation was able to match the physical model data utilizing the weir equations provided by Guo in Table 1. Discharge coefficients for each slope and grate can be found in Table 5. These discharge coefficients are used in the equations presented in Table 6 (adapted from Guo's) to calculate the flow from the overflow outlet structure; variable locations are shown in Figure 23.

Table 5 - Discharge coefficients for each slope and grate

100-yr Orifice Coefficient	
0.60	100-yr orifice
Overflow Outlet Coefficient, C_d	
0.64	1:0 (H:V) Slope - No Grate
0.62	1:0 (H:V) Slope - Close Mesh
0.60	1:0 (H:V) Slope - Type C
0.68	4:1 (H:V) Slope - No Grate
0.63	4:1 (H:V) Slope - Close Mesh
0.62	4:1 (H:V) Slope - Type C
0.68	3:1 (H:V) Slope - No Grate
0.60	3:1 (H:V) Slope - Close Mesh
0.58	3:1 (H:V) Slope - Type C

Table 6 - Equations to determine discharge from the overflow section of an extended detention basin.

Flow Type	**Equation**
100-yr orifice	$Q_O = C_o A_o \sqrt{2gH}$
Flat Weir	$Q_{Flat} = \frac{2}{3} n C_d (2B + 2L)\sqrt{2g} H^{\frac{3}{2}}$
Sloped Un-Submerged Weir ($H < H_b$)	$Q_{WS} = \frac{4}{15} n C_d \sqrt{2g} \cot(\theta)\, H^{\frac{5}{2}}$ $Q_{WB} = \frac{2}{3} n C_d \sqrt{2g} B H^{\frac{3}{2}}$ $\boldsymbol{Q_W = 2Q_{WS} + Q_{WB}}$
Sloped Submerged Weir ($H \geq H_b$)	$Q_{WS} = \frac{4}{15} n C_d \sqrt{2g} L \cos(\theta) \left[\frac{H^{\frac{5}{2}} - (H - H_b)^{\frac{5}{2}}}{H_b}\right]$ $Q_{WB} = \frac{2}{3} n C_d \sqrt{2g} B H^{\frac{3}{2}}$ $\boldsymbol{Q_W = 2Q_{WS} + Q_{WB}}$

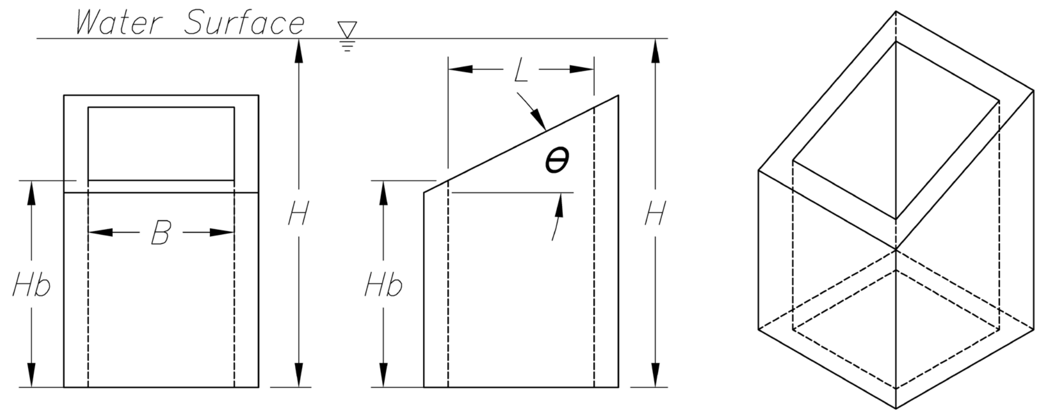

Figure 23 - Variable Locations for Equations in Table 6

Guo's weir-flow equations calculate flow into only three sides of the overflow outlet (flow over the top edge is considered negligible because the head acting on this section is limited by the overland flow across the ground surface). For the 1:0 (H:V) no slope case, this is not realistic because flow can enter equally from all four sides since these outlets typically are not installed in the bank of the EDB and do not have surrounding topography.

Reclamation used the information gathered from the physical model to develop a new spreadsheet for UDFCD to utilize when calculating the discharge from the overflow outlet. Visual Basic programming was used to logically determine, based on the outlet configuration and the stage, which equations should be used to determine the flow. The entire Visual Basic program can be found in Appendix A. The spreadsheet calculations were compared to all physical model data to verify that it accurately calculates the flow through the discharge structure. Figure 24 is a plot directly from the spreadsheet that shows the physical model data overlaid on top of the spreadsheet stage discharge relationship. The stage sharply increases where the 100-yr orifice begins controlling the flow through the outlet structure.

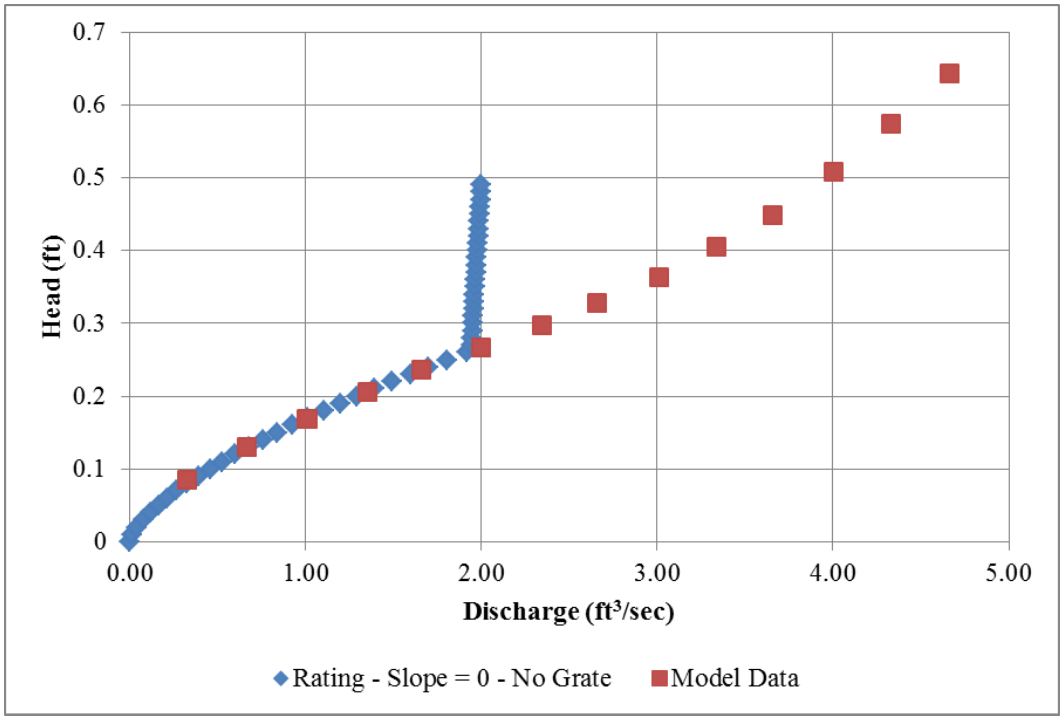

Figure 24 - Final spreadsheet stage discharge plot showing the rating calculated from the spreadsheet in blue and the model data for a 1:0 (H:V) slope with no grate in red.

When modeling the complete EDB, two different water quality orifice options were tested, a series of orifice holes and an elliptical weir configuration. The elliptical weir configuration is desirable from a debris standpoint because the orifice holes have a tendency to clog when floating debris enters the EDB. Given the same stage, Figure 19 shows that the elliptical weir will release more flow from the EDB than the orifice configuration.

UDFCD wished to know if the Flow-3D commercially available computational fluid dynamics (CFD) software package was capable of accurately determining the discharge through the overflow outlet. FLOW-3D is developed by Flow Science Inc. and was chosen because of its ability to accurately model free-surface flows. FLOW-3D utilizes the Reynolds-averaged Navier-Stokes (RANS) equations to solve for fluid flow. Reclamation modeled a single configuration using Flow-3D at a 4:1 (H:V) slope with no grate. Figure 25 confirms that Flow-3D can be used to accurately model the discharge through the overflow section of the outlet works. The CFD model was set up and run at multiple discharges and differences between the physical and numerical model were only compared graphically. Differences between the physical and CFD model were minimal and could most likely be improved by doing a mesh resolution analysis on the CFD model to determine if the resolution of the model was as accurate as possible. This type of analysis was not pursued.

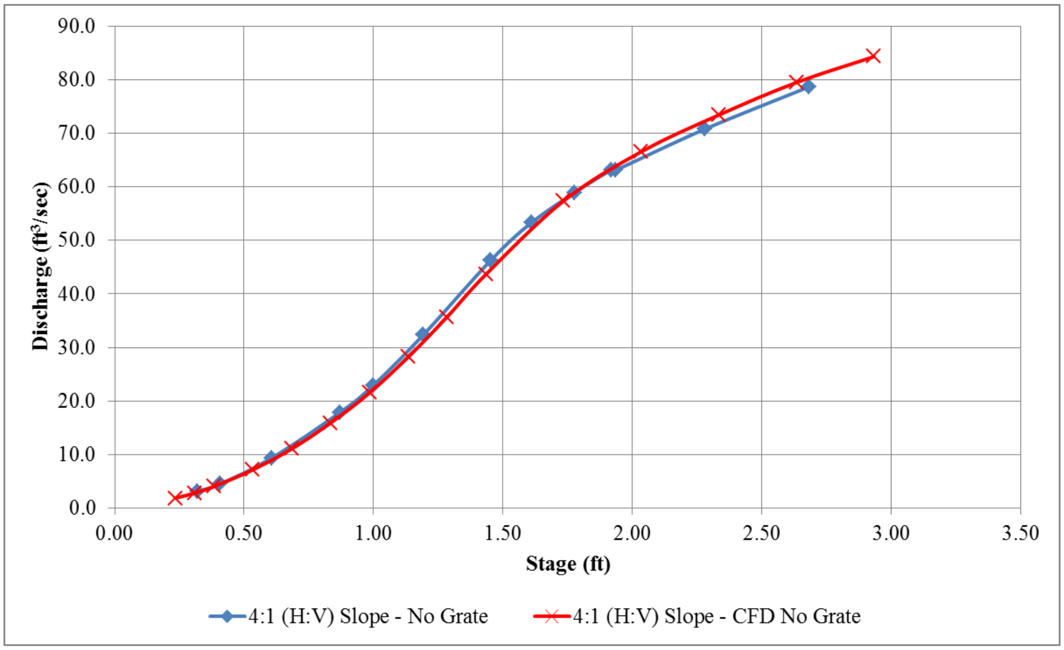

Figure 25 - Computational Fluid Dynamics (CFD) and physical model comparison of the 4:1 (H:V) slope configuration with no grate.

RECOMMENDATIONS

Reclamation recommends utilizing the spreadsheet created in conjunction with this study to calculate the flow through extended detention basin overflow outlets (EDBs). The results of the spreadsheet were compared to the physical model data and good agreement was confirmed between model data and spreadsheet results for all configurations tested.

The spreadsheet has some limitations.

- It has only been verified against the data collected in the physical model, which was limited to three slopes and three grate configurations.
- If grates are used in parallel (side by side or end to end to increase area) the spreadsheet calculations may not be accurate.
- The spreadsheet does not calculate the flow through the water quality orifice plates at the front of the outlet structure. This calculation could be added if desired.
- Results of the spreadsheet are dependent on accurately inputting the correct dimensions and discharge coefficients from Table 5.

If no 100-yr orifice is installed in the EDB outlet structure, calculating flow from the overflow outlet based on stage is difficult because oscillating stage with a constant inflow is possible when the weir flow limit is exceeded. Flow through the 100-yr orifice should be calculated using an orifice discharge coefficient of 0.60 and a head referenced from the center of the orifice.

Designs that are unique and push the limits of what was tested in the physical model can likely be modeled successfully using computational fluid dynamics. Reclamation successfully matched model test data using Flow-3D, and other CFD modeling programs might render similar results.

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