ARTICLE 5: REQUIREMENTS FOR STORMWATER MANAGEMENT

Introduction

Site stormwater management involves the control of **stormwater runoff** from **development sites** to minimize the potential for adverse impacts on adjacent and downstream properties. The goal of **site stormwater** management, as stated in §501.1 of the WMO, is to ensure that **development** does not:

- "Increase **flood** elevations or decrease **flood** conveyance capacity upstream or downstream of the area under the **ownership** or control of the **co-permittee**;
- Pose any increase in **flood** velocity or impairment of the hydrologic and hydraulic functions of streams and **floodplains** unless a **water resource benefit** is realized;
- Violate any provision of this **ordinance** either during or after construction; and
- Unreasonably or unnecessarily degrade surface or **groundwater** quality."

The objective of **stormwater** management is to balance **development** with **stormwater** drainage and design approaches that do not cause an increase in **flooding** and do not unreasonably or unnecessarily degrade water quality due to **development**. **Site development** and **stormwater** management standards are established in the WMO to meet the requirements of §501.1. **Site stormwater** management involves the control of the rate, volume, and quality of **stormwater runoff** associated with **development**. The WMO requires several practices to be incorporated into the **site** drainage design to address these impacts, including **detention** (rate control), **volume control practices** (volume reduction), and **flow-through** (treatment) requirements.

The WMO provides standards for **stormwater** management including:

- **Site Runoff** Requirements (§502);
- **Site Volume Control** Requirements (§503);
- **Site Detention** Requirements (§504); and
- Allowances for **Redevelopment** (§505).

The purpose of this section of the **TGM** is to provide guidance on how to demonstrate compliance with the WMO requirements, in order to ensure that the **development** or **redevelopment** will not result in a negative impact to downstream or adjacent properties.

Note: All bold terms contained in this document are defined terms in the WMO. Refer to Appendix A of the WMO or the TGM for the definition of each bold term.

GENERAL SITE DEVELOPMENT AND STORMWATER MANAGEMENT REQUIREMENTS (§501)

Site Stormwater Management and Development Categories

The WMO regulates **developments** based on the size and type of **development**. Table 5-1 (Table 2 in the WMO) summarizes which **site stormwater** management requirements must be met, depending on the size of **parcel** and type of **development**.

There are three categories of **site stormwater** regulations:

1. **Site Runoff** Requirements (§502);
2. **Site Volume Control** Requirements (§503); and
3. **Site Detention** Requirements (§504).

There are five types of **developments** specifically covered in the WMO:

1. **Single-family home**;
2. **Residential subdivision**;
3. **Non-residential** and multi-family **development**;
4. **Right-of-way development**; and
5. **Open space** (not part of a larger **development**).

Table 5-1 shows the **stormwater** regulation categories for the different classifications of **developments**. As defined in the WMO, **parcel** or **parcels** means "**contiguous** land area under single **ownership** or control, under an affidavit of **ownership**, or under a single legal description on record with the **Cook County** Recorder of Deeds Office." As defined in the WMO, **development** means:

"Any human-induced activity or change to real estate (including, but not limited to, grading, paving, excavation, dredging, fill, or mining; alteration, subdivision, change in land use or practice; **building**; or storage of equipment or materials) undertaken by private or public entities that affects the volume, flow rate, drainage pattern or composition of **stormwater**, or the **substantial improvement** of an existing **building** in a **Special Flood Hazard Area**. The term **development** shall include **redevelopment** and shall be understood to not include **maintenance**."

Table 5-1. Summary of Site Stormwater Management Requirements (Table 2 from WMO)[1]

Development Type (See Appendix A of the WMO for definitions.)	**§502**	**§503**	**§504**
	Runoff Requirements	**Volume Control Requirements[2]**	**Storage Requirements[2]**
Single-Family Home	Exempt	Exempt	Exempt
Residential Subdivision	Parcels ≥ 1 acre	Parcels ≥ 1 acre	Parcels ≥ 5 acres
Multi-Family Residential	Parcels ≥ 0.5 acre	Parcels ≥ 0.5 acre	Parcels ≥ 3 acres ‡
Non-Residential	Parcels ≥ 0.5 acre	Parcels ≥ 0.5 acre	Parcels ≥ 3 acres ‡
Right-of-Way	New Impervious Area ≥ 1 acre	New Impervious Area ≥ 1 acre †	New Impervious Area ≥ 1 acre †
Open Space	Parcels ≥ 0.5 acre	Not Applicable	Not Applicable

1 **Site stormwater** management requirements are not required for **maintenance activities** as defined in Appendix A of the WMO.

2 Requirements are applicable when a **Watershed Management Permit** is required under §201 of the WMO.

‡ Starting the effective date of the WMO, any new **development** on the **parcel** that totals either individually or in the aggregate to more than one-half (0.5) of an acre.

† Where practicable.

At the bottom of Table 5-1, it is important to note that **site stormwater** management requirements are not required for **maintenance**. The examples of **maintenance** are provided to explicitly state that in-kind replacement or repair of existing infrastructure or facilities are not regulated under the WMO.

"Where practicable" is included with **right-of-way** (roadway) **development** since it is understood that often roadway design is limited by public **right-of-way** constraints. In cases where the WMO **stormwater** requirements have not been met, the applicant must demonstrate how the **development** has met the **stormwater** requirements of the WMO to the maximum extent practicable.

Developments less than or equal to 0.1 acre that require a **Watershed Management Permit** are exempt from the runoff (§502), volume control (§503), and Legacy Sewer Permit requirements (§505). **Developments** greater than 0.1 acre are subject to these requirements when the ownership area exceeds the thresholds specified in Table 5-1.

SITE RUNOFF REQUIREMENTS (§502)

As described in Table 5-1, the **site runoff** requirements in §502 apply to:

1. **Residential subdivision development** on **parcels** that total one acre or more; and/or
2. **Non-residential** or **multi-family residential development** on **parcels** that total 0.5 acre or more; and/or
3. **Right-of-way development** that creates one acre or more of **new impervious area**, and/or
4. **Open space development** on **parcel**(s) that total 0.5 acre or more.

Site runoff requirements are not required for the **development** of **single family homes** on individual residential lots. The act of subdividing land distinguishes between a **single family home** and a **residential subdivision**, which is subject to the **site stormwater** requirements in Article 5. **Site runoff** requirements (§502) for **development** may include the:

1. Determination of **watershed** boundaries;
2. Design of minor **stormwater facilities**;
3. Design of major **stormwater facilities**;
4. Determination of **design runoff rate**s;
5. Consideration of existing sub-surface drainage;
6. Consideration of **upstream tributary flows** and bypass flows;
7. Protection of **depressional storage**;
8. Allowable flow depths on roadways and parking lots; and/or
9. **Building** protection standards.

Not all **development sites** will contain all of the **site** features discussed in §502. In addition, the applicant must procure all federal, state, and/or local permits associated with the **stormwater runoff** from the **site**.

Transfer Between Watersheds

Unless there are extenuating circumstances for a **development**, **stormwater runoff** should stay within the existing **watershed**. The WMO (§502.2) specifies that the transfer of water between **watersheds** is prohibited unless such transfers do not violate the provisions of §501.1 of the WMO. "**Watershed**" for this provision is defined as the **tributary area** to the **waterway** to which the **development** drains. "Transfer" refers to the diversion of **runoff** or stream flow via overland flow paths or **storm sewer** systems.

In order to demonstrate compliance with §502.2, the existing and proposed conditions **tributary areas** must be evaluated for the **site**. For the determination of the **development site's watershed** boundaries, topography from **Cook County** or the US Geological Survey (USGS) should be obtained. Some **sites**, prior to **development**, may contain a ridge line resulting in the **site** being tributary to multiple **watersheds**. The proposed grading plan for the **site** should preserve these natural drainage boundaries. For any portions of the **site** that drain to another **watershed**, it must be demonstrated that no negative impacts will result to any **watershed** to which the **site** was originally tributary. Negative impacts include increases in flowrates, velocities, and **flood** elevations. The maximum permissible impact is a 0.1-ft increase in **flood** elevations and a 10% increase in velocities. Computation of flows must be completed utilizing the methodology required for **major stormwater systems**.

Minor Stormwater Systems

Minor stormwater systems are typically designed to collect and convey events less than the 100-year **storm event** and consist of **storm sewers**, inlets/catch basins and other **stormwater** collection appurtenances. **Minor stormwater systems** prevent water from ponding on roadways, sidewalks, and properties during the more frequent **storm events**.

§502.5 requires that the **minor stormwater systems** be sized to convey **runoff** from the **tributary area** under fully developed conditions consistent with the design requirements of the local jurisdiction or existing **stormwater** system. Minor **stormwater facilities**, when not regulated by the local community, should be designed to convey a minimum of the 10-year event by gravity, with no pressure flow in the conduits.

Minor **stormwater facilities** should consider all **watershed** areas upstream of the point of design under fully developed conditions to ensure that they are not undersized. In some cases, the fully developed conditions of the upstream area may be the current land. Regardless, the upstream land use conditions must be assessed. In cases where the upstream **tributary areas** are still subject to **development**, the municipal or County land use plans should be evaluated. If the upstream **tributary area** is subject to further **development**, then any special considerations of the **runoff** rate must be pre-approved by the **District** or an **authorized municipality**.

Rational Method

The Rational Method can be utilized to determine the peak flow from a particular **tributary area** for the sizing of minor **stormwater facilities**, primarily consisting of inlets and **storm sewers**. The Rational Method is defined as:

$$Q = C \cdot i \cdot A$$

Where: Q = peak rate of flow (cubic feet/second)

C = Composite **runoff** coefficient , based on Table 5-2

i = intensity of precipitation (inches/hour)for a duration equal to time of concentration, t_c, and a given return period (Table 5-3)

A = area (acres)

For each **drainage area**, the composite **runoff** coefficient (C) must be computed using the values shown in Table 5-2.

Table 5-2. Runoff Coefficients (C Values) for the Rational Method

Surface Type	Runoff Coefficient, C
Impervious area (Roads, roofs, sidewalks, etc.)	0.90
Pervious Area	0.45
Gravel (loose, unbound)	0.75
Water Surface (open water)	1.00
Native Plantings	0.15
Wetlands	0.79
Synthetic Turf Fields	0.75
Green Infrastructure:	
Pervious Surfaces (Porous Asphalt, Pervious Concrete, Permeable Pavers)	0.75
Bioswale	0.10
Rain Garden	0.10
Green Roof	(Refer to Table 5-9)

The duration used for the rainfall intensity, i, is equal to the time of concentration, t_c, for the **drainage area**. An example t_c calculation is provided in Example 5.3. **Bulletin 70** sectional rainfall intensities are to be used for Rational Method calculations; these rainfall intensities are provided in Table 5-3 below. The rainfall intensities provided in Table 5-3 can be used to generate intensity-frequency-duration (IDF) curves for **storm sewer** sizing applications. For example, Figure 5.1 shows the IDF curves generated by the *Hydraflow* computer software based on the user-specified **Bulletin 70** rainfall intensities.

Table 5-3. *Bulletin 70* Northeast Sectional Rainfall Intensities

Duration	Intensity (in/hr)						
	1-Year	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
5 min	3.60	4.32	5.52	6.48	7.92	9.36	10.92
10 min	3.30	4.02	5.04	5.88	7.26	8.52	10.02
15 min	2.72	3.28	4.12	4.84	5.96	7.00	8.20
30 min	1.86	2.24	2.82	3.30	4.08	4.78	5.60
1 hour	1.18	1.43	1.79	2.10	2.59	3.04	3.56
2 hour	0.74	0.90	1.12	1.32	1.63	1.91	2.24
3 hour	0.53	0.65	0.81	0.95	1.18	1.38	1.62
6 hour	0.31	0.38	0.48	0.56	0.69	0.81	0.95

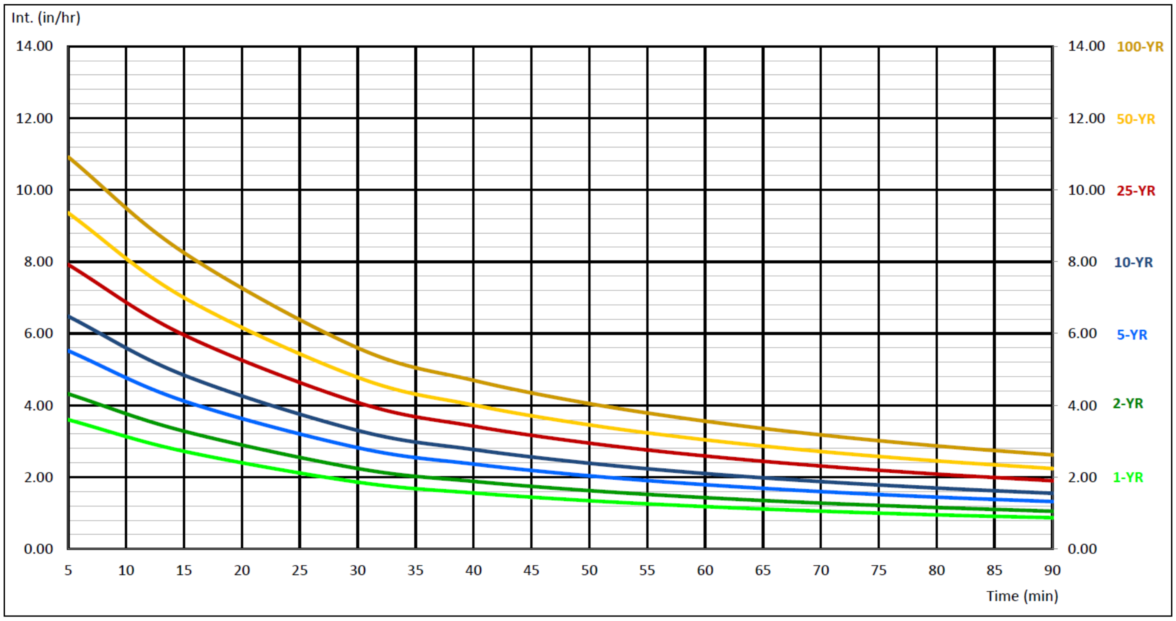


Figure 5.1. *Bulletin 70* Northeast Sectional IDF Curves (Generated by *Hydraflow*)

Once the peak flowrate has been established, Manning's equation can be used to size the pipe or open channel through an iterative process. This must be done until design capacity from Manning's equation is found to exceed the required peak flow design rate as determined in the Rational Method. Manning's equation is as follows:

$$Q = \frac{1.49}{n} \cdot A \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$$

Where: Q = design flowrate (cubic feet/second)

n = the roughness coefficient of the pipe or channel (dimensionless)

A = the cross sectional area of the pipe or channel (square feet)

R = the hydraulic radius of the pipe or channel which is the area (square feet) divided by the wetted perimeter (feet)

S = the slope of the pipe or channel (foot/foot)

Example 5.1 – Sizing of a Minor Stormwater System

For the proposed drainage schematic shown below, determine the required **storm sewer** sizes to convey the 10-year **storm event**. The area, **runoff** coefficient, and time of concentration are provided for each subbasin. Also provided are the rim elevations of each proposed **structure** and the distances between **structures**. Use Manning's equation to determine the required pipe sizes for the proposed drainage system.

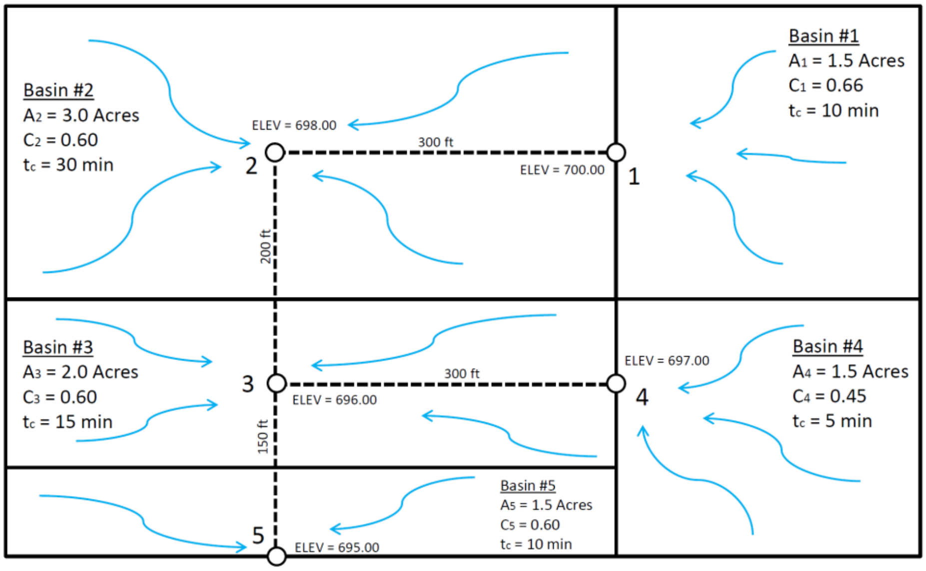
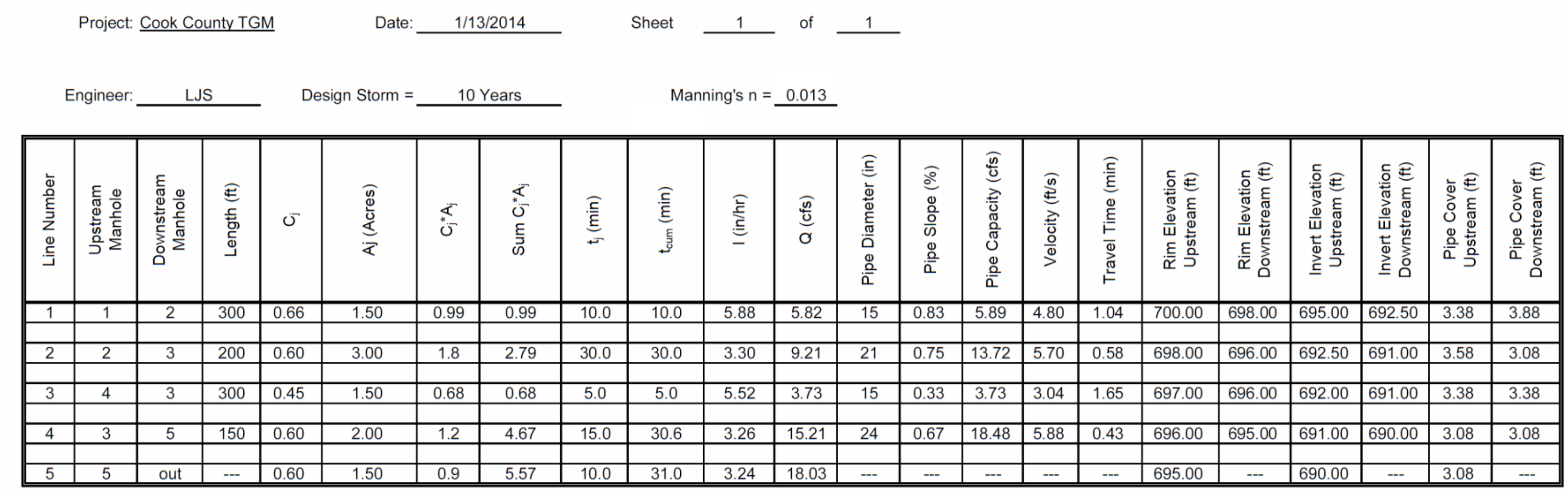


Figure 5.2. Proposed Drainage Schematic for Storm Sewer Sizing

In this example, a spreadsheet based on Manning's equation is used to size the **proposed storm sewers**. The completed spreadsheet with the required **storm sewer** sizes is provided as Figure 5.3. Referring to Figure 5.3, the following items should be noted:

- Rainfall intensities are calculated from the IDF curves provided as Figure 5.1 and are based on **Bulletin 70** northeast sectional rainfall depths.
- Manning's equation is used to verify the full-pipe capacity is greater than the design 10-year peak flowrate.

- A Manning’s equation spreadsheet is an acceptable alternative to a **storm sewer** sizing computer program such as *Hydraflow*, *HYDRA*, and *StormCAD*. If an applicant is using computer software to design a **storm sewer** network, the hydraulic grade line (HGL) of the system must be plotted to ensure the **storm sewer** system is not under pressure flow for the 10-year design. Because Manning’s equation assumes full-pipe conditions, the HGL is equal to the crown of the pipe and therefore no other HGL calculations are required using this methodology.



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Engineer: LJS Design Storm = 10 Years Manning's n = 0.013

Line Number	Upstream Manhole	Downstream Manhole	Length (ft)	C_j	Aj (Acres)	C_j*A_j	Sum C_j*A_j	t_j (min)	t_{cum} (min)	I (in/hr)	Q (cfs)	Pipe Diameter (in)	Pipe Slope (%)	Pipe Capacity (cfs)	Velocity (ft/s)	Travel Time (min)	Rim Elevation Upstream (ft)	Rim Elevation Downstream (ft)	Invert Elevation Upstream (ft)	Invert Elevation Downstream (ft)	Pipe Cover Upstream (ft)	Pipe Cover Downstream (ft)
1	1	2	300	0.66	1.50	0.99	0.99	10.0	10.0	5.88	5.82	15	0.83	5.89	4.80	1.04	700.00	698.00	695.00	692.50	3.38	3.88
2	2	3	200	0.60	3.00	1.8	2.79	30.0	30.0	3.30	9.21	21	0.75	13.72	5.70	0.58	698.00	696.00	692.50	691.00	3.58	3.08
3	4	3	300	0.45	1.50	0.68	0.68	5.0	5.0	5.52	3.73	15	0.33	3.73	3.04	1.65	697.00	696.00	692.00	691.00	3.38	3.38
4	3	5	150	0.60	2.00	1.2	4.67	15.0	30.6	3.26	15.21	24	0.67	18.48	5.88	0.43	696.00	695.00	691.00	690.00	3.08	3.08
5	5	out	---	0.60	1.50	0.9	5.57	10.0	31.0	3.24	18.03	---	---	---	---	---	695.00	---	690.00	---	3.08	---

Figure 5.3. Storm Sewer Design Sheet – Rational Method, Example 5.1

Major Stormwater Systems

Major stormwater systems are drainageways that convey flows from major storms when the capacity of **minor stormwater systems** is exceeded. Generally, the **minor stormwater system** is designed to carry the 10-year design **runoff** event and the **major stormwater system** is designed to carry the additional flow for the 100-year design **runoff** event. The WMO (§502.6) requires that the **major stormwater system** be designed to convey the **design runoff rate** of the 100-year **storm event** using a **critical duration analysis** and an event hydrograph method. A **critical duration analysis** is required only for the following:

1. Large **developments**, where:
    a. Residential > 10 acres, and
    b. Commercial > 5 acres
2. Smaller sites with an offsite flow area greater than the **development** area; and
3. Clear conveyance issues that may contribute to onsite flooding.

The **design runoff rate** for **major stormwater systems** must include the calculated flows from all the **tributary areas** upstream of the point of design without increasing **flood** or **erosion** damages downstream or on adjacent properties. A **major stormwater system** consists of the overland flow routes and channels that convey **stormwater runoff** that exceeds the **storm sewer** capacity downstream, to the **site detention facility**.

In general, the **minor stormwater system** consists of a **storm sewer** system (designed for the 10-year return interval), and the **major stormwater system** consists of an overland flow path (designed for the 100-year return interval). Overland flow paths can consist of roadways or side/rear yard swales and can be sized using Manning's equation. As shown in Table 5-4 below, only two Manning's n values should be used for the design of the overland flow route, depending on whether the proposed channel is paved or unpaved.

Table 5-4. Manning's n Values for Design of Overland Flow Routes (Source: Chow, 1959)

Surface Type	Manning's n Value
Paved Channels (asphalt or concrete roadways)	0.013
Unpaved Channels (grassed)	0.035

For all projects, the direction of flow for **major stormwater systems** should be clearly shown on the appropriate plan sheets and exhibits (grading plan and drainage exhibit). Flow arrows should be provided that show the direction of overland flow, and if storm sewers are sized to convey the 100-year design runoff rate, this should be indicated on the utility plan.

Event Hydrograph Methods

The maximum flowrate determined in the 100-year **critical duration analysis** will establish the **design runoff rate**. As with the **minor stormwater system** design, the major **stormwater facilities** should consider all **watershed** areas upstream of the point of design under fully developed conditions to ensure that they are not undersized. Peak discharge for conveyance of **stormwater** and the design of **stormwater facilities** must be based on the **critical duration analysis** using the appropriate rainfall distribution. The analysis should include the 1-, 2-, 3-, 6-, 12-, 18-, 24- and 48-hour storm durations to determine the critical storm duration for the **watershed**.

The following event hydrograph methods are allowed for the determination of the **design runoff rate** and the sizing of **major stormwater facilities**:

1. HEC-1 (SCS **runoff** method);
2. HEC-HMS (SCS **runoff** method); and
3. TR-20.

Other modeling programs that are not listed may be used with the approval of the **District**. Event hydrographs must incorporate the following:

1. Antecedent Moisture Condition II. The antecedent moisture condition is the measure of the soil conditions with respect to **runoff** potential before a storm. “I” is dry, “II” is average and “III” represents saturated soils conditions; the CN values for all **development** should be based on the values provided in Table 5-7, which are based on an AMC II.
2. Huff Rainfall Distribution. The Huff rainfall distribution is a measure of the time distribution of rainfall for **storm events** of various durations. The appropriate Huff quartile distribution to be used for each associated rainfall duration is found in Table 5-5.

Table 5-5. Huff Quartile Distributions

HUFF QUARTILE DISTRIBUTIONS												
CUMUL. STORM PERCENT	AREA < 10 SM				AREA > 10 & AREA < 50				AREA > 50 & AREA < 400			
	HUFF QUARTILE				HUFF QUARTILE				HUFF QUARTILE			
	1st	2nd	3rd	4th	1st	2nd	3rd	4th	1st	2nd	3rd	4th
05	16	03	03	02	12	03	02	02	08	02	02	02
10	33	08	06	05	25	06	05	04	17	04	04	03
15	43	12	09	08	38	10	08	07	34	08	07	05
20	52	16	12	10	51	14	12	09	50	12	10	07
25	60	22	15	13	62	21	14	11	63	21	12	09
30	66	29	19	16	69	30	17	13	71	31	14	10
35	71	39	23	19	74	40	20	15	76	42	16	12
40	75	51	27	22	78	52	23	18	80	53	19	14
45	79	62	32	25	81	63	27	21	83	64	22	16
50	82	70	38	28	84	72	33	24	86	73	29	19
55	84	76	45	32	86	78	42	27	88	80	39	21
60	86	81	57	35	88	83	55	30	90	86	54	25
65	88	85	70	39	90	87	69	34	92	89	68	29
70	90	88	79	45	92	90	79	40	93	92	79	35
75	92	91	85	51	94	92	86	47	95	94	87	43
80	94	93	89	59	95	94	91	57	96	96	92	54
85	96	95	92	72	96	96	94	74	97	97	95	75
90	97	97	95	84	97	97	96	88	98	98	97	92
95	98	98	97	92	98	98	98	95	99	99	99	97

The distributions are expressed as percentages of cumulative rainfall depth as a percentage of storm duration. The 1st quartile distribution is applied to storm durations less than or equal to 6 hours, the 2nd quartile distribution is applied to storm durations greater than 6 hours but less than 12 hours, the 3rd quartile distribution applies to storm durations greater than 12 hours but less than or equal to 24 hours, and the 4th quartile distribution applies to storm durations greater than 24 hours. There are three separate Huff distributions that represent the three different sizes of **drainage areas** utilized in the study: **drainage areas** less than 10 square miles (left), **drainage areas** greater than 10 square miles but less than 50 square miles (center), and **drainage areas** between 50 and 400 square miles (right). The majority of projects will be using values for areas less than 10 square miles.

3. Illinois State Water Survey **Bulletin 70** Northeast Sectional Rainfall Statistics. **Bulletin 70** rainfall data provide the expected rainfall amounts for selected storm durations and return periods. Table 5-6 provides the **Bulletin 70** rainfall depths for the various storm durations and return intervals.

Table 5-6. Illinois State Water Survey Bulletin 70 Rainfall Depths for Northeast Sectional (inches)

Duration	**Storm event Frequency**						
	1-year	**2-year**	**5-year**	**10-year**	**25-year**	**50-year**	**100-year**
5 min	0.30	0.36	0.46	0.54	0.66	0.78	0.91
10 min	0.55	0.67	0.84	0.98	1.21	1.42	1.67
15 min	0.68	0.82	1.03	1.21	1.49	1.75	2.05
30 min	0.93	1.12	1.41	1.65	2.04	2.39	2.80
1 hour	1.18	1.43	1.79	2.10	2.59	3.04	3.56
2 hour	1.48	1.79	2.24	2.64	3.25	3.82	4.47
3 hour	1.60	1.94	2.43	2.86	3.53	4.14	4.85
6 hour	1.88	2.28	2.85	3.35	4.13	4.85	5.68
12 hour	2.18	2.64	3.31	3.89	4.79	5.62	6.59
18 hour	2.30	2.79	3.50	4.11	5.06	5.95	6.97
24 hour	2.51	3.04	3.80	4.47	5.51	6.46	7.58
48 hour	2.70	3.30	4.09	4.81	5.88	6.84	8.16
72 hour	2.93	3.55	4.44	5.18	6.32	7.41	8.78
120 hour	3.25	3.93	4.91	5.70	6.93	8.04	9.96
240 hour	4.12	4.95	6.04	6.89	8.18	9.38	11.14

Calculating Runoff Curve Number

The **runoff** curve number (CN) is a hydrologic parameter used to estimate the potential for **stormwater runoff** from a particular area. Factors affecting CN values include land cover type (including hydrologic conditions), hydrologic soil group, and antecedent moisture condition. The CN is used to calculate **runoff** volumes and flows for the **development site** as well as to size the **site's detention facility**. CN values must be calculated for both existing and proposed conditions of the **development site**.

For calculating CNs of proposed **developments**, the land uses and soil types should be taken from those listed in Table 5-7. This table is a modified version of Table 2-2a from the *Urban* ***Hydrology*** *for Small* ***Watersheds****, TR-55* (TR-55 Manual), published by **NRCS**, for determining curve numbers. The table also includes appropriate CNs for **volume control practices**. For further guidance, refer to the TR-55 Manual, which is available through the Certified Professional in **Erosion** and **Sediment** Control (CPESC) website at: http://www.cpesc.org/reference/tr55.pdf.

Table 5-7.Runoff Curve Numbers for Urban Areas

Cover Type and Hydrologic Conditions	Curve Numbers for Hydrologic Soil Group	
	C	D
Impervious area (roads, roofs, sidewalks, etc.)	98	98
Pervious area (open space, mostly grassed areas)	74	80
Gravel (railroad yards, roads, parking lots)	89	91
Water surface (open water)	100	100
Newly graded areas (pervious areas only, no vegetation)	91	94
Native Plantings	70	77
Wetlands	91	94
Synthetic Turf Fields	91	91
Green Infrastructure:		
Non-compacted gravel areas	91	91
Porous/permeable pavement	91	91
Bioswale	63	70
Rain Garden	63	70
Green Roof	Refer to Table 5-9	

The **development**s and **redevelopments** that will be permitted under the WMO will be mostly residential and commercial **developments** that have significant amounts of **impervious area**. Due to the general uniformity of **developments**, the land use types used in the CN calculation will be limited to those shown in Table 5-7. The majority of **Cook County** consists of either native poorly-drained soils (HSG C and D) or became that way due to **development**. The areas of the County that contain well-drained soils (HSG A and B) are extremely limited. And even though an area may be labeled as having well-drained soils on the soil survey, if the area has been developed, the soils have likely lost their infiltration capacity. Based on this information, the calculation of CNs for proposed **developments** should be based on HSG C and D type soils. The use of A and B soils in calculating CNs would only be allowed for those **sites** where native soils are currently intact and a soil test is performed to verify the infiltration capacity. The applicant would also have to demonstrate that these soils would be preserved under the developed conditions. Soils information for **Cook County** is available on-line through the **NRCS** at: websoilsurvey.nrcs.usda.gov/.

Example 5.2 provides a sample CN calculation for a 3-acre, **non-residential development** for proposed conditions, as shown in Figure 5.4. The CN calculation spreadsheet for this example is provided as Figure 5.5.

Example 5.2 – Example CN Calculation

Figure 5.4 shows a proposed **development** that consists of a **building**, a permeable pavement parking lot, **open space**, and a **stormwater** detention basin. Using **NRCS** TR-55 methodology, determine the CN for the **site**.

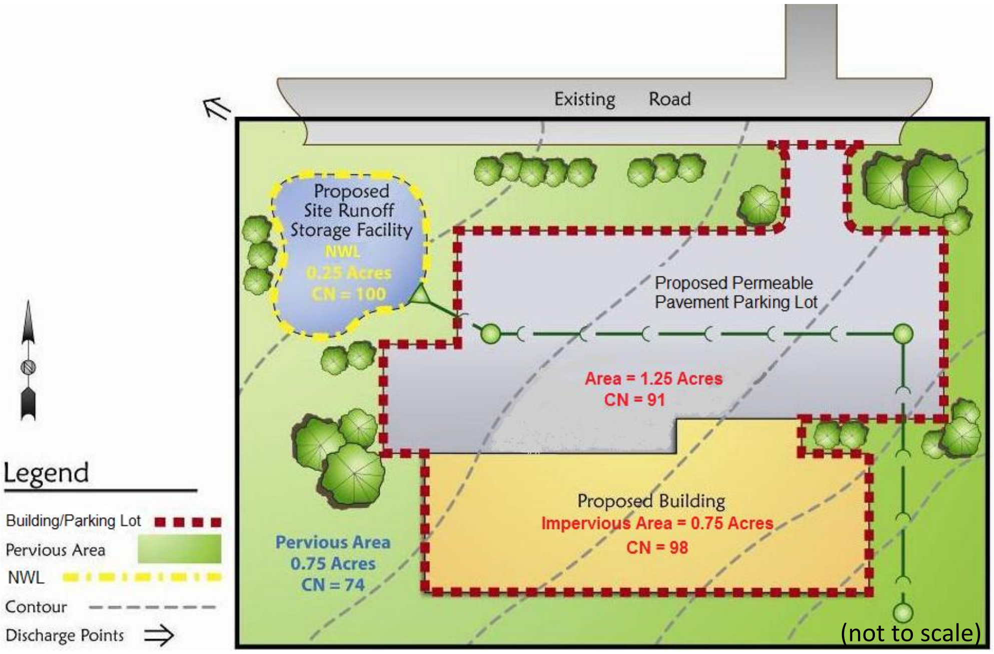
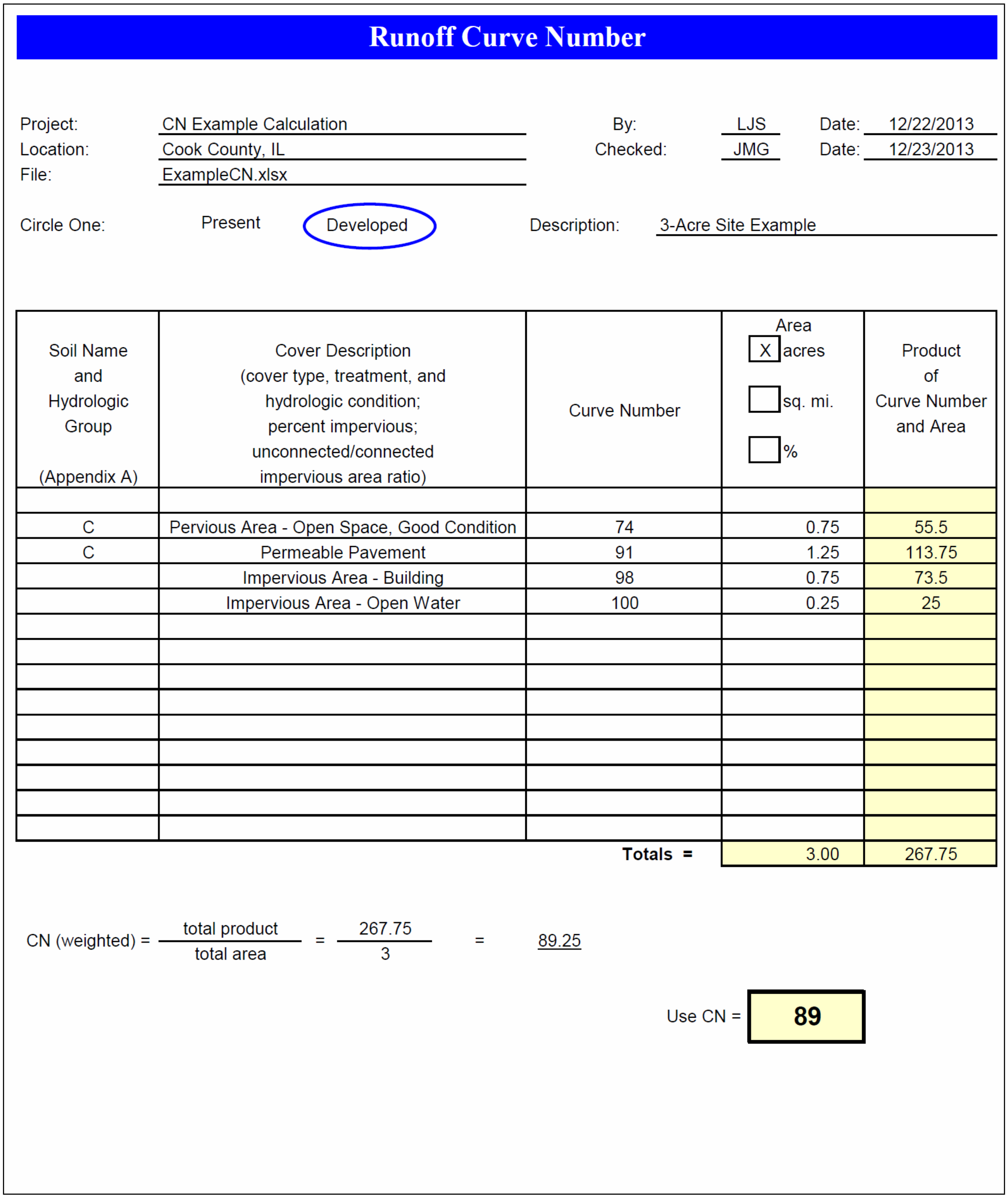


Figure 5.4. Example of Calculation of Runoff Curve Numbers

In this example, there are two types of pervious area onsite: (1) the **open space** around the perimeter of the **development** that is assumed to be in good condition (grass cover is >75%) and HSG C-type soils and (2) the permeable pavement parking lot. From the values provided in Table 5-7, the **open space** CN is 74 and the CN for permeable pavement is 91. A distinction is made between the two types of **impervious areas** on the proposed **site**: the **building** is assigned a CN of 98, while the open water of the wet-bottom detention basin (at the normal water level (NWL)) is assigned a CN of 100.

Using the CN worksheet, as shown in Figure 5.5, a composite CN of 89 is computed for the **development**.



Runoff Curve Number

Project: CN Example Calculation
Location: Cook County, IL
File: ExampleCN.xlsx

By: LJS Date: 12/22/2013
Checked: JMG Date: 12/23/2013

Circle One: Present [✓ Developed]

Description: 3-Acre Site Example

Soil Name and Hydrologic Group (Appendix A)	Cover Description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	Curve Number	Area ☒ acres ☐ sq. mi. ☐ %	Product of Curve Number and Area
C	Pervious Area - Open Space, Good Condition	74	0.75	55.5
C	Permeable Pavement	91	1.25	113.75
	Impervious Area - Building	98	0.75	73.5
	Impervious Area - Open Water	100	0.25	25
		Totals =	3.00	267.75

$$\text{CN (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{267.75}{3} = \underline{89.25}$$

Use CN = **89**

Figure 5.5. CN Calculation Worksheet for Example 5.2

Calculating the Time of Concentration

The time of concentration (t_c) is the time it takes for **stormwater runoff** to travel from the most hydraulically distant point in a **watershed** to the **watershed** outlet. Based on the **NRCS** TR-55 methodology, the t_c for a **watershed** is a combination of sheet flow, shallow concentrated flow, and open channel flow.

Sheet flow is the first segment of a flow path and consists of shallow flow (less than 0.1 ft) over plane surfaces. Sheet flow should be limited to a length of 100 feet, and roughness coefficients (Manning's n values) for sheet flow over different types of surfaces should be taken from Table 5-8 (Table 3-1 from the **NRCS** TR-55 manual). To determine the travel time for sheet flow, Manning's kinematic (Overton and Meadows) solution should be used.

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}s^{0.4}}$$

Where: T_t = travel time (hr)
n = Manning's roughness coefficient (Table 5-8)
L = flow length (ft)
P_2 = 2-year, 24-hour rainfall depth (3.04 in)
s = slope of hydraulic grade line (land slope, ft/ft)

Table 5-8. Manning's n Values for Sheet Flow (Table 3-1 from NRCS TR-55)

Surface Description	Manning's n Value
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated Soils	
Residue Cover ≤ 20%	0.06
Residue Cover ≥ 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:	
Light underbrush	0.40
Dense underbrush	0.80

After 100 ft, sheet flow will transition to shallow concentrated flow. The travel time for shallow concentrated flow is based on the equation (TR-55 equation 3-1) shown below.

$$T_t = \frac{L}{3600V}$$

Where: T_t = travel time (hr)
L = flow length (ft)
V = average velocity (ft/s)
3600 = conversion factor from seconds to hours

The velocity term in the equation is calculated separately based on the type of surface. If the surface is "unpaved," the velocity is calculated using the following equation:

$$V_{unpav} = 16.1345(s)^{0.5}$$

Where: s = slope of the ground (ft/ft)

If the surface is "paved," the following equation should be used:

$$V_{pav} = 20.3282(s)^{0.5}$$

Water may also move through a **watershed** as open channel flow. Creeks, ditches, and **storm sewers** are all examples of open channel conveyance systems in a **watershed**. The travel time for open channel flow can be determined using TR-55 Equation 3-1, with flow velocities based on Manning's equation (assuming bankfull conditions). For open channel flow in **storm sewer**s, it is reasonable to assume an average velocity of 2 ft/s.

In general, urbanization decreases the t_c in **watersheds**, as flow over smooth, impervious surfaces – along with the installation of efficient conveyance systems such as **storm sewers** – speeds up the flows and decreases the t_c. The t_c affects the shape of the **runoff** hydrograph for a **watershed**: a shorter t_c results in a steep **runoff** hydrograph with a higher peak flow, whereas a longer t_c will flatten the shape of the **runoff** hydrograph and result in a lower peak flow. In many cases, particulary in small urban areas, the t_c will be small, and a minimum value of 10 minutes should be used.

Example 5.3 – Example Time of Concentration Calculation

The figure below shows the flow path from the most hydraulically distant point in a **watershed** (A) to the **watershed** outlet (D). Using **NRCS** TR-55 methodology, determine the time of concentration (t_c) for this **watershed** assuming the following:

Segment AB: Sheet flow; dense grass; slope = 0.03 ft/ft; length = 100 ft
Segment BC: Shallow concentrated flow; paved; slope = 0.02 ft/ft; length = 650 ft
Segment CD: Open channel flow; length = 900 ft; velocity = 2 ft/s (from Manning's equation)

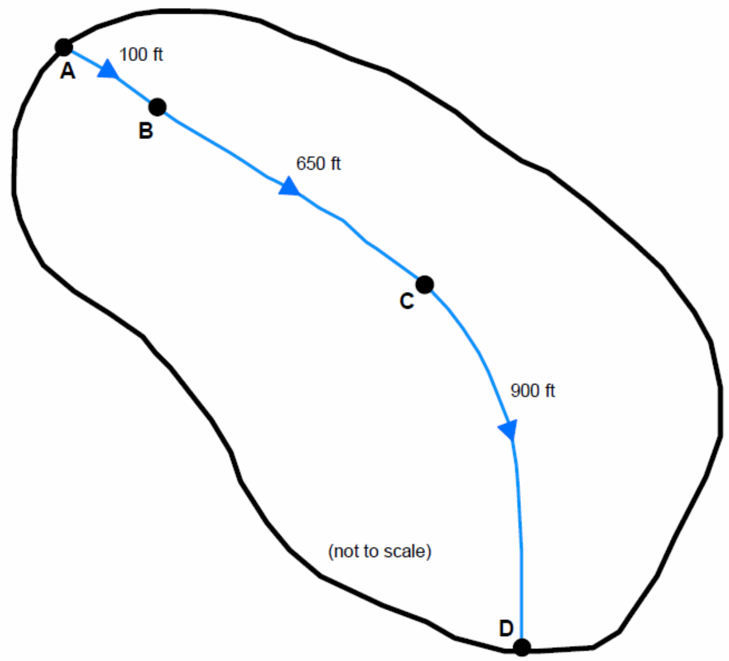
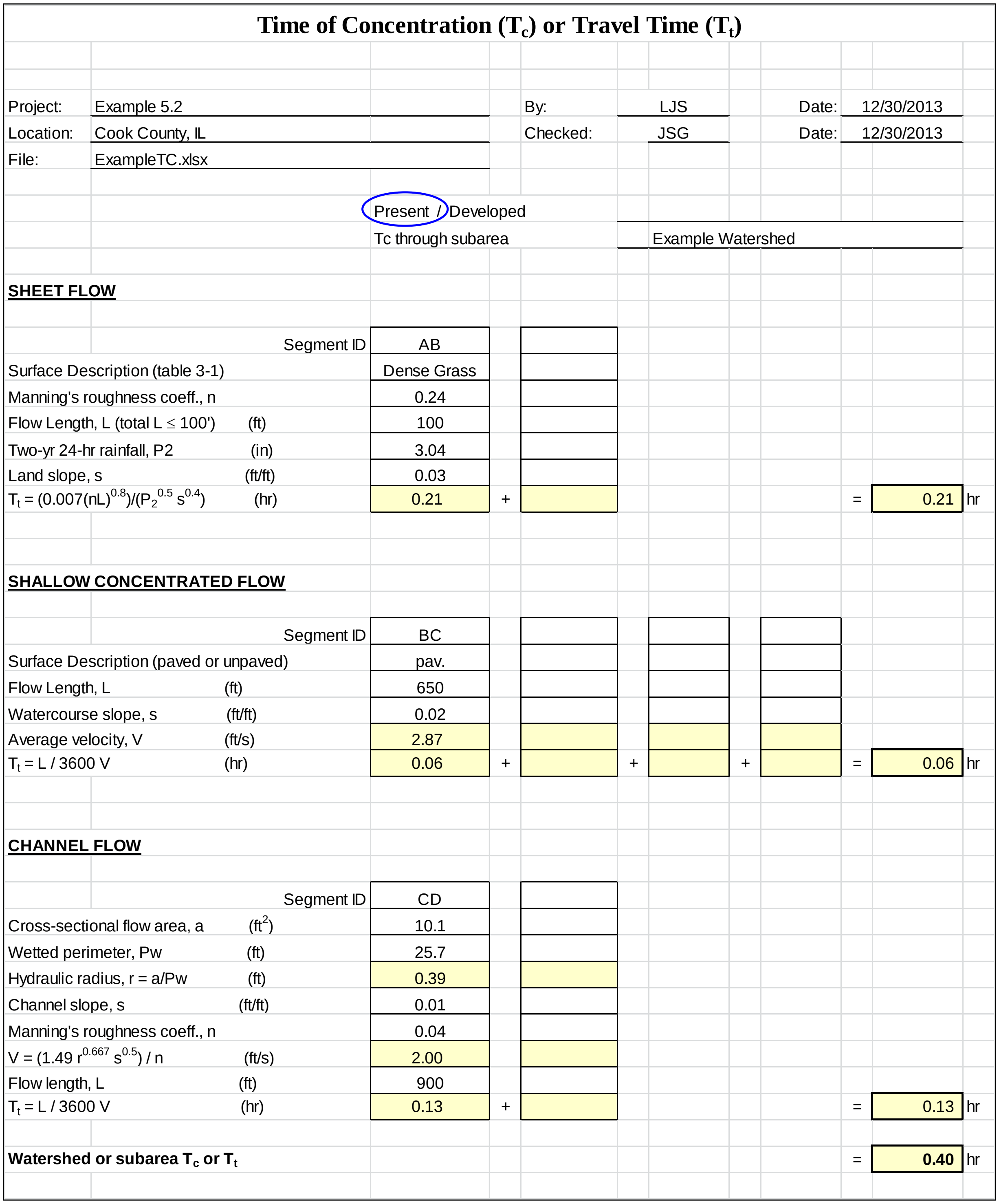


Figure 5.6. Time of Concentration Flow Path for Example 5.3

As shown in the calculation worksheet (Figure 5.7), the t_c for this **watershed** is 0.40 hours.



Time of Concentration (T_c) or Travel Time (T_t)

Project: Example 5.2 By: LJS Date: 12/30/2013
Location: Cook County, IL Checked: JSG Date: 12/30/2013
File: ExampleTC.xlsx

[✓ Present] / Developed
Tc through subarea Example Watershed

SHEET FLOW

Segment ID		AB					
Surface Description (table 3-1)		Dense Grass					
Manning's roughness coeff., n		0.24					
Flow Length, L (total L ≤ 100')	(ft)	100					
Two-yr 24-hr rainfall, P2	(in)	3.04					
Land slope, s	(ft/ft)	0.03					
$T_t = (0.007(nL)^{0.8})/(P_2^{0.5}\ s^{0.4})$	(hr)	0.21	+		=	0.21	hr

SHALLOW CONCENTRATED FLOW

Segment ID		BC									
Surface Description (paved or unpaved)		pav.									
Flow Length, L	(ft)	650									
Watercourse slope, s	(ft/ft)	0.02									
Average velocity, V	(ft/s)	2.87									
$T_t = L / 3600\ V$	(hr)	0.06	+		+		+		=	0.06	hr

CHANNEL FLOW

Segment ID		CD					
Cross-sectional flow area, a	(ft^2)	10.1					
Wetted perimeter, Pw	(ft)	25.7					
Hydraulic radius, r = a/Pw	(ft)	0.39					
Channel slope, s	(ft/ft)	0.01					
Manning's roughness coeff., n		0.04					
$V = (1.49\ r^{0.667}\ s^{0.5}) / n$	(ft/s)	2.00					
Flow length, L	(ft)	900					
$T_t = L / 3600\ V$	(hr)	0.13	+		=	0.13	hr

Watershed or subarea T_c or T_t = **0.40** hr

Figure 5.7. Time of Concentration Worksheet for Example 5.3

Example 5.4 – Sizing of a Major Stormwater System

Based on **Cook County** topography, a proposed **development** has an offsite **drainage area** of 40 acres. Using **NRCS** TR-55 methodology, the CN was calculated to be 87 and the t_c was determined to be 1.2 hrs. Determine the **design runoff rate** that must be bypassed through the **site** and size a grassed swale to safely convey the flow.

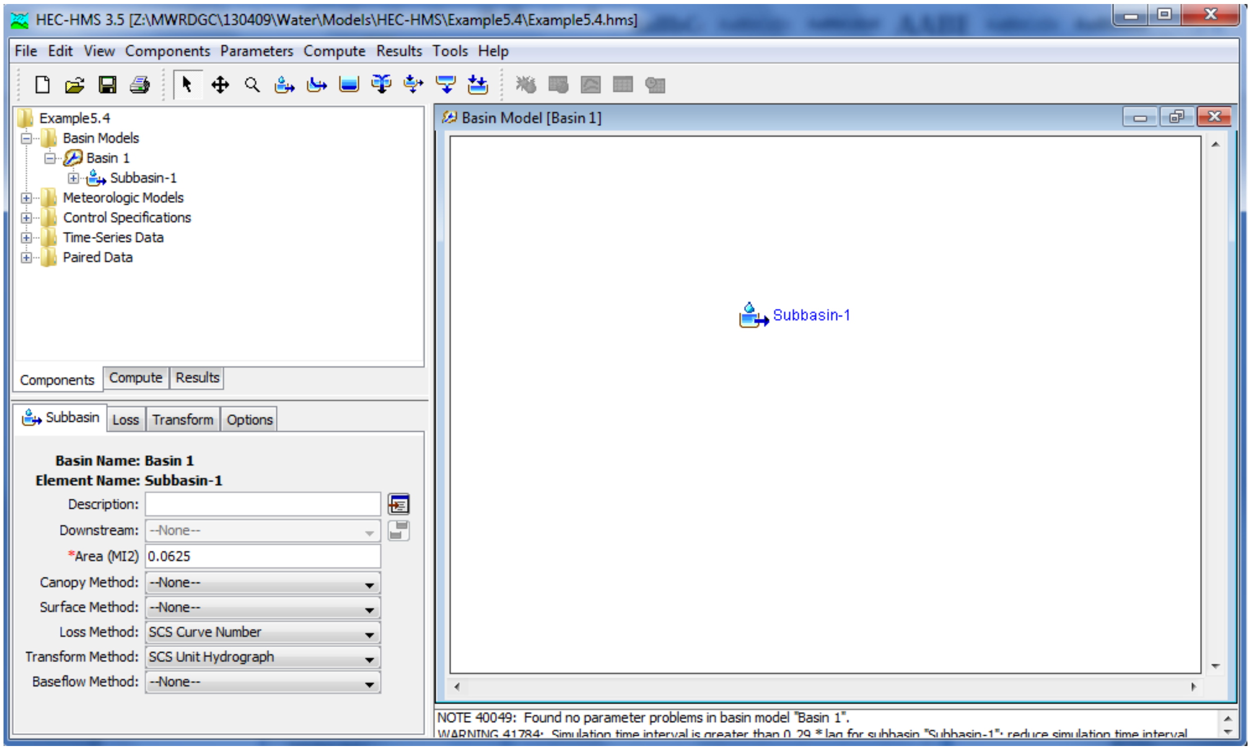
Solution: Use HEC-HMS to perform a **critical duration analysis** to determine the 100-year **design runoff rate**. Once the design flowrate is known, Manning's equation can be used to size the overland flow path.

Step 1: A one-subbasin HEC-HMS model was developed to represent the offsite area. The subbasin (named Subbasin-1) is the only component of the *Basin Model*.

For Subbasin-1, enter the information for the project **site**:

- Area = 0.0625 square miles (40 acres)
- CN = 87
- Lag time = 0.72 hrs (0.6 * t_c)
- SCS CN and Unit Hydrograph Methodology

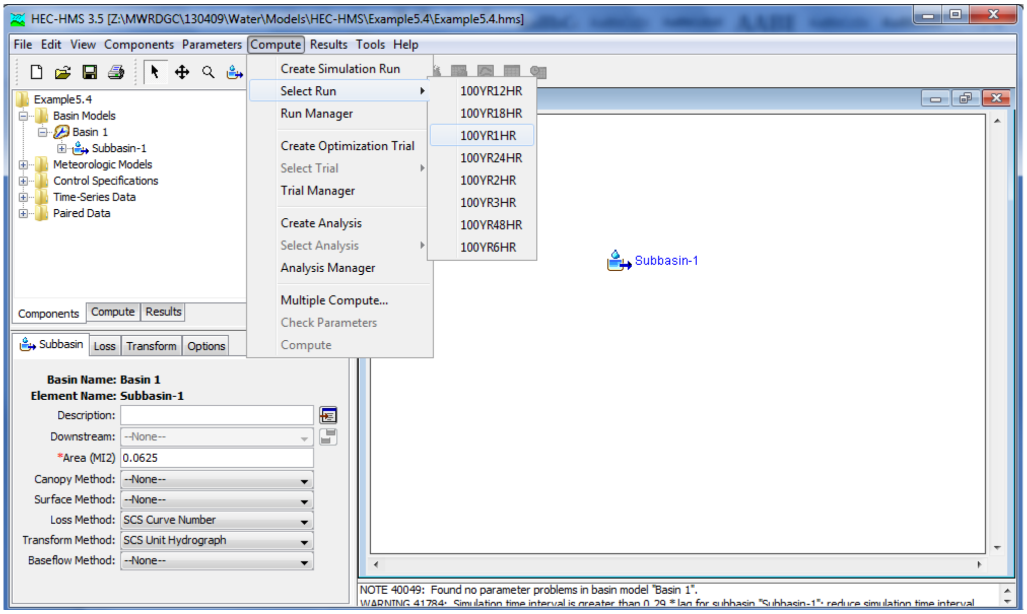
The *Meteorological Model* contains the rainfall depth information, which are the 100-year, 1-hour through 100-year, 48-hour depths from Table 5-6. The *Time-Series Data* contains the time distribution of rainfall, which includes the Huff 1st through 4th quartile distributions for various storm durations.



Basin Name: **Basin 1**
Element Name: **Subbasin-1**
Description:
Downstream: --None--
*Area (MI2) 0.0625
Canopy Method: --None--
Surface Method: --None--
Loss Method: SCS Curve Number
Transform Method: SCS Unit Hydrograph
Baseflow Method: --None--

NOTE 40049: Found no parameter problems in basin model "Basin 1".
WARNING 41784: Simulation time interval is greater than 0.29 * lag for subbasin "Subbasin-1"; reduce simulation time interval

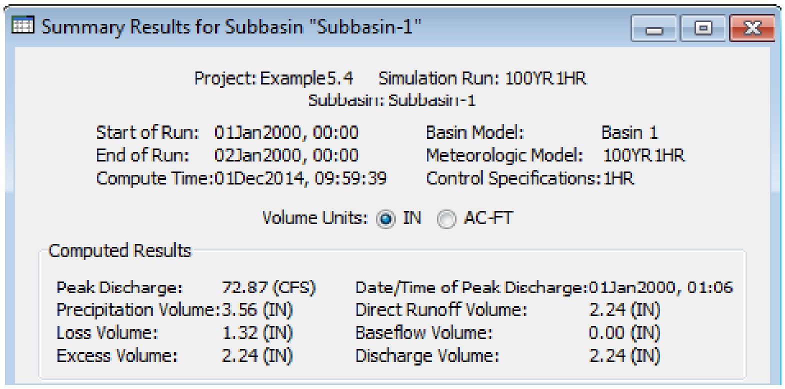
Create and compute simulation runs for the 100-year, 1-hour through 100-year, 48-hour storm durations.



Basin Name: Basin 1
Element Name: Subbasin-1
Description:
Downstream: --None--
*Area (MI2) 0.0625
Canopy Method: --None--
Surface Method: --None--
Loss Method: SCS Curve Number
Transform Method: SCS Unit Hydrograph
Baseflow Method: --None--

```
NOTE 40049:  Found no parameter problems in basin model "Basin 1".
WARNING 41784:  Simulation time interval is greater than 0.29 * lag for subbasin "Subbasin-1"; reduce simulation time interval
```

The figure below shows the summary output for the 100-year, 1-hour **storm event**.



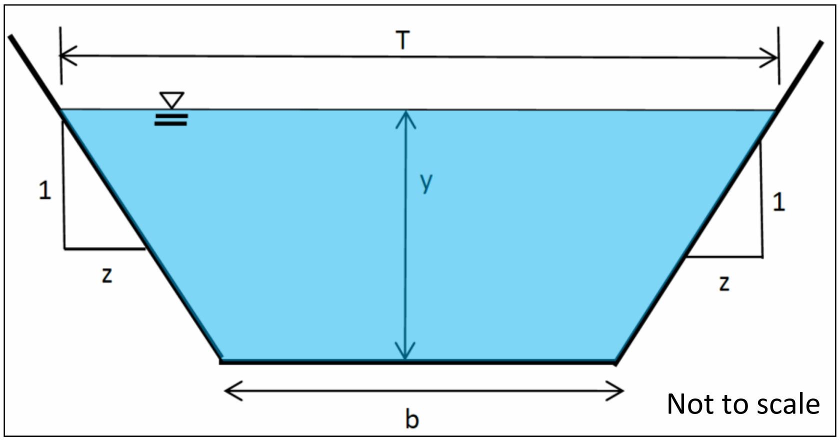
```
Summary Results for Subbasin "Subbasin-1"

            Project: Example5.4     Simulation Run: 100YR1HR
                           Subbasin: Subbasin-1

     Start of Run:    01Jan2000, 00:00        Basin Model:          Basin 1
     End of Run:      02Jan2000, 00:00        Meteorologic Model:   100YR1HR
     Compute Time:01Dec2014, 09:59:39         Control Specifications:1HR

                    Volume Units: ● IN   ○ AC-FT

Computed Results

  Peak Discharge:        72.87 (CFS)     Date/Time of Peak Discharge:01Jan2000, 01:06
  Precipitation Volume:3.56 (IN)         Direct Runoff Volume:       2.24 (IN)
  Loss Volume:           1.32 (IN)       Baseflow Volume:            0.00 (IN)
  Excess Volume:         2.24 (IN)       Discharge Volume:           2.24 (IN)
```

Because there is no summary output table for the various simulations, the results of the **critical duration analysis** have been summarized in the table below.

Storm event	Peak Flowrate (cfs)
100-Year, 1-Hour	73
100-Year, 2-Hour	**77**
100-Year, 3-Hour	70
100-Year, 6-Hour	56
100-Year, 12-Hour	42
100-Year, 18-Hour	35
100-Year, 24-Hour	29
100-Year, 48-Hour	17

As shown in the table above, the 100-year, 2-hour **storm event** yields the highest peak flowrate (77 cfs) for the offsite area.

Step 2: Using the peak 100-year flowrate computed from HEC-HMS, use Manning's equation to determine the geometry of the overland flow path to safely convey the offsite flow. From the **site** grades, the slope of the channel can be no more than 1%. Based on the elevations of adjacent homes, there can be no more than one foot of depth in the channel. The depth must be kept under one foot to provide the two feet of freeboard for the 100-year critical duration **storm event**.



Referring to the figure above, for a trapezoidal channel,

The flow area, A, is calculated by: $A = (b + zy)y$

The wetted perimeter, P_w, is calculated using: $P_w = b + 2y \times (1+z^2)^{0.5}$

The hydraulic radius, R, is calculated by: $R = A/P$

Since this is a grassed channel, the Manning's n value should be 0.035, as determined from Table 5-4. The slope s, is 1%, or 0.01 ft/ft and the flow depth, y, is equal to 1 ft. Side slopes, z, for the overland flow route should be equal to 3:1. This only leaves the bottom width, b, which can be adjusted until the capacity of the channel is greater than the 100-year design flowrate of 77 cfs. Solving Manning's equation iteratively, the required bottom width is 12 ft which yields a channel capacity of 79 cfs.

Minor and Major Stormwater System Design Considerations

Upstream flows must be considered when developing a **site**, as stated in the WMO (§502.9). Flows from the upstream **tributary areas** to the **site** should be computed under fully developed conditions to ensure the proposed **stormwater facilities** are not undersized. Upstream offsite flows cannot be blocked. Upstream flows can be routed around the **development** via **storm sewer** or swales and must be designed to convey the **base flood** event. Otherwise, the upstream flows can be routed into and through the **site stormwater facility** (including **site storm sewer**, swales, **site detention facilities**, etc.).

All systems should drain by gravity. Hydraulic grade line (HGL) computations must be provided to verify **buildings** and **structures** are properly protected from **flooding**. HGL computations must take into account appropriate tailwater conditions at the most downstream point of the proposed **stormwater** system (HWL, **BFE**, etc.). The rim elevations of all **storm sewer** manholes (catch basins, inlets, trench drains, area drains, etc.) must be at or above the high water elevation of the **site detention facility** or if an emergency overflow weir/spillway is provided, no lower than six (6) inches above the weir/spillway crest elevation.

Existing and proposed low-entry points and **buildings** should be considered when designing **major stormwater facilities**. A minimum of one foot must be provided from the maximum designed water surface elevation to the low-entry point of any **building**. This includes all **major stormwater facilities** (overland flow paths and **detention facilities**).

Design of drainageways should have:

1. Sufficient energy dissipation at the outlet to prevent scouring of the streambank, bed, or downstream land. Armoring of the stream channel should not be considered in lieu of energy dissipation. Energy dissipation is essential to avoid transferring scour and stability problems further downstream;

2. To the extent possible, deep-rooted vegetated side slopes, and inverts with velocities sufficiently limited shall be used to prevent scouring for open-channel drainageways. This guide addresses the plan requirement to control **sediment** and **erosion** from drainageways; and

3. Have reasonable side slopes given the engineering properties of the materials. A 3:1 side slope typically provides adequate stability in an earth channel and is a mowable slope. A 4:1 or shallower side slope is desirable. Deviations from the minimum value should be justified by appropriate calculations (e.g., slope stability calculations) and **maintenance** plans that do not require mowing.

Some areas within the **District** jurisdiction/boundaries have **combined sewer** systems. **Developments** shall provide separate **sanitary sewer** and **storm sewer** systems within the property boundaries of the **development**.

Runoff from rooftops, parking lots, and other impervious surfaces that do not discharge directly into a **site** detention system facility should discharge onto pervious surfaces. This will allow for infiltration, **runoff** reduction, and the improvement of water quality.

Stormwater System Easements

The WMO (§502.8) requires that **major and minor stormwater systems** be located within easements or public **right-of-way** explicitly providing public access for **maintenance** of such facilities. This includes **storm sewer** pipes, overland flow routes, swales, and portions of the curb and gutter system.

Easements should be sized to allow for the **maintenance** of the systems. A minimum of 10 feet should be dedicated over any **storm sewer** line or other conveyance facility. However, consideration should be given to the depth of **storm sewer** in order to allow for access to and **maintenance** of the lines. For example, a **storm sewer** line that is 30 feet deep will likely require more than a 10-foot easement, should **maintenance** be needed. Areas up to 10 feet beyond the established high water level of a **stormwater detention facility** should be placed within an easement.

Easement language should include **maintenance** access provisions for all **stormwater facilities**. Easements should be dedicated and recorded on all legal documents, including plats or titles of all **parcels** containing the easements. The dedication should indicate that the easement serves the purpose of allowing for the **maintenance** and access to the **stormwater facilities**.

Existing Sub-Surface Drainage (Drain Tile)

Per §502.7 of the WMO, the applicant shall locate all existing field tile systems on the project **site** plan. Drain tile can either be safely routed through or around the **development site**. The drain tile can be reconnected to the existing drain tile at the downstream side of the **development** or incorporated into the proposed **minor and major stormwater facility** of the **site** (only in **separate sewer areas**).

Any modifications to drain tile shall not cause damage to upstream, downstream, or adjacent **structures**, land uses, or **stormwater facilities**. Calculations will likely be required to demonstrate this. As the slope of the drain tile is often unknown, an estimate of the capacity of the pipe will have to be made based upon the known existing drain tile size and an estimated pipe slope.

Particular attention should be paid to those drain tile systems that are used to convey **upstream tributary flows**. Drain tiles must maintain drainage service to these upstream **tributary areas** during construction until the new **storm sewer** system is constructed. Drain tiles used as outlets must be within public **rights-of-way** or in an easement.

Any drain tile replaced onsite should be properly reconnected to the downstream system. Onsite drain tile should be located within a public **right-of-way** or a dedicated easement. This information should be shown on the **record drawings** with all appropriate rim, invert, pipe size, pipe material, etc. information shown.

Depressional Storage

Depressional storage is an above-ground storage area that does not have a gravity surface outlet and only drains by evaporation or infiltration. Up to the point of overtopping, a **depressional storage** area effectively reduces the flowrate of **stormwater runoff** leaving the **site** as it stores water onsite.

A hydrologic analysis for the pre-developed **site runoff** rate, which factors in the storage volume of the **depressional storage** area, shall be performed for the 2-year, 10-year and 100-year **storm events** of a 24-hour duration in accordance with §502.4 of the WMO. All areas tributary to the depressional area (both on- and offsite) should be considered in the hydrologic model. If the depressional area is contained onsite, a complete topographic survey with 1-foot contour intervals should be used to determine the relationship between the stage and storage values of the depression, which are to be included in the hydrologic model. If the depressional area extends offsite, **Cook County** one-foot topography may be used. For offsite areas that extend into other counties, the USGS topography maps can be used to complete a stage-storage relationship table. Any outlet from the depression, such as drain tile or an overflow weir, should be identified, and calculations are to be provided to determine the discharge rates from these **structures** as a component of the total existing release rate from the **site**.

The proposed **site's runoff** rate shall not exceed the existing **runoff** rate. A table of existing versus proposed flows and **flood** elevations should be provided for comparison purposes showing no increase in flows and **flood** elevations.

For those **sites** which require **site detention facilities**, the **allowable release rate** from the **site** shall be either equal to or less than the existing **runoff** rate. If the existing **runoff** rate is less than the calculated **allowable release rate**, the existing **runoff** rate becomes the **site's** new **allowable release rate**. The **site detention facility** shall be sized, accounting for the smaller of the two flow values.

If the depressional area is mapped as **floodplain** on the **FIRM** maps, the **floodplain** provisions of the WMO (Article 6) will apply.

Flow Depths on Roadways and Parking Lots

Maximum **flood** depths on any roadway for both storage and conveyance purposes shall not exceed 12 inches during the **base flood** condition. If located in the **regulatory floodway**, maximum **flood** depths on new parking lots shall not exceed 12 inches during the **base flood** condition. Parking areas that are located in the **floodplain** shall post signs indicating the **flood** hazard area. Figure 5.8 illustrates the acceptable **flood** depths on a roadway.

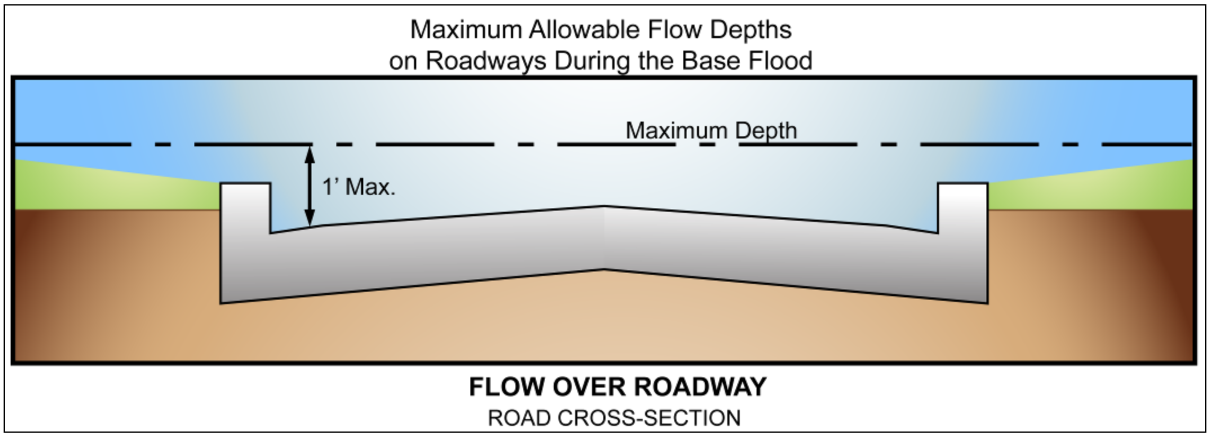


Figure 5.8. Allowable Flood Depths for Roadways

Building Protection Standards

For proposed **buildings** in areas adjacent to or within the limits of the **floodplain**, **building** protection standards are provided in Article 6 of the **TGM**. Proposed **buildings** shall be protected from **stormwater runoff** conveyed through the **site**. The proposed **buildings** shall be designed so that the lowest entry point for usable spaces shall be one foot above the high water elevation as determined from the design of the **major stormwater system** or overland flow paths. All usable space in new **buildings**, or added to existing **buildings**, adjacent to a **site detention facility** shall be elevated, **floodproofed**, or otherwise protected with a minimum of one foot of freeboard for the **base flood** condition to prevent the entry of surface **stormwater**.

SITE VOLUME CONTROL REQUIREMENTS (§503)

Introduction

The WMO (§503.2) requires that one (1) inch of **stormwater runoff** from all impervious surfaces of the **development** be treated using **volume control practices**. Impervious surfaces include: pavement and gravel paved areas, buildings, permanent pool areas, and wet bottomed detention ponds greater than 12-inches in depth. Synthetic turf fields and porous pavement areas are not considered impervious surfaces for purposes of volume control requirements.

The one (1) inch of **stormwater runoff** from these newly developed impervious areas is termed the **volume control storage**. The purpose of these practices is to provide pollutant and volume reduction mechanisms for **stormwater runoff** discharged from the **site** to receiving waters. The WMO presents a hierarchy of **volume control practices** in §503.3 to treat the **volume control storage** in §503.2:

1. **Retention-based practices** with quantifiable storage capacity are the primary form of water quality treatment. The use of **retention-based practices** must be maximized for treating the **volume control storage**; and

2. **Flow-through practices** are required for treatment of any portion of the **volume control storage** that has not been treated using **retention-based practices**."

3. **Redevelopments** with **site** constraints that prevent use of **retention-based practices** to retain the **volume control storage** in full, the **volume control storage** may be reduced by twenty-five percent (25%) for every five-percent (5%) of reduced **impervious area**, if the **development** can meet the conditions provide in §503.3.C of the WMO.

As presented in Table 5-1 of the **TGM** and in §503.1 of the WMO, the volume control requirements apply to:

1. **Residential subdivision development** on **parcels** totaling one (1) acre or more; and/or

2. **Non-residential** or **multi-family residential development** on **parcels** totaling 0.5 acre or more; and/or

3. Roadway **development** that is more than one (1) acre of **new impervious area**, where practicable.

The selection of **volume control practices** should be based on the **site** feasibility, which includes soil suitability. The design of **volume control practices** must be incorporated into the overall **site** design while meeting the **site runoff** requirements of §502 and the **site detention facility** requirements of §504.

For **developments** that require a permit and the ownership area is greater than or equal to the thresholds specified in Table 5-1, **volume control practices** are generally required for the proposed **impervious area**. However, if the area of **development** is less than or equal to 0.1 acre, the volume control requirements do not apply. The following example demonstrates this point:

Commercial Development
Total **Development** Area = 0.49 acre
Total Ownership Area = 1.0 acre
Proposed **Impervious area** = 0.34 acre
Includes Qualified Sewer Construction
Is Volume Control Storage Required? Yes

Although the disturbance is less than 0.5 acre, a permit is required for the **development** since it includes **qualified sewer construction**. Because the ownership area is greater than 0.5 acre (from Table 5-1) and the **development** area is greater than 0.1 acre, the **volume control storage** will have to be provided for the 0.34 acre of proposed **impervious area**.

Impervious Area Reduction

As stated in §503.3.C of the WMO, **redevelopments** with site constraints that prevent the use of **retention based** practices may reduce the **impervious area** to meet the volume control requirements. The **volume control storage** may be reduced by twenty-five percent (25%) for every five-percent (5%) of reduced **impervious area**. Therefore, credit for the entire required **volume control storage** (100%) can be provided by reducing the **impervious area** by 20%. This concept is illustrated in Figure 5.9 below.

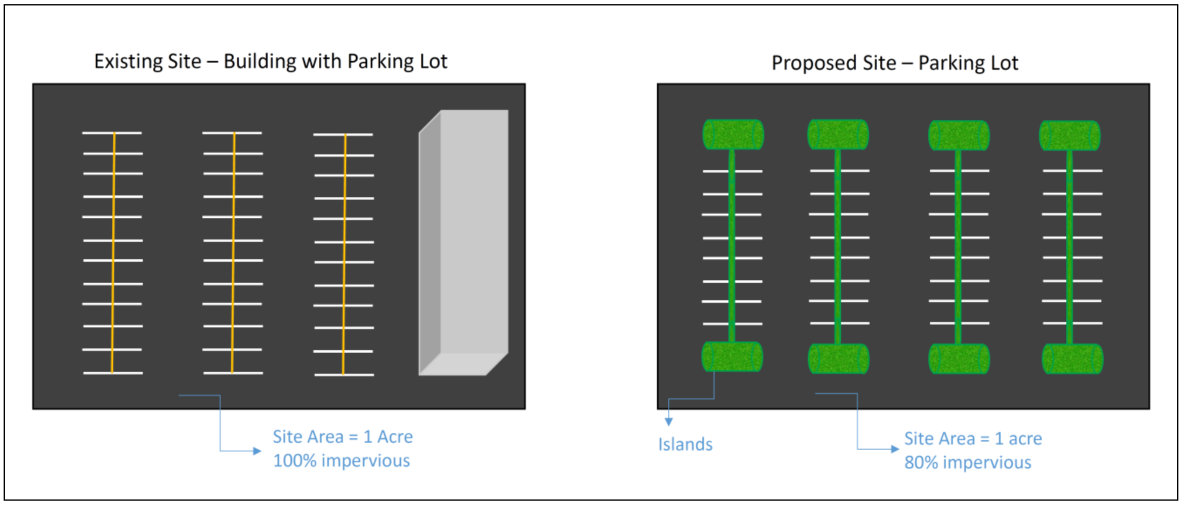


Figure 5.9. Reduction in Impervious Area to Meet Volume Control Requirements

Volume Control Practices Overview

Volume control practices utilize designated infiltration areas or **structures** to capture a portion of **stormwater runoff** (i.e., the **volume control storage**) and retain it onsite such that the **runoff** is able to 1) percolate through (or into) the underlying soils, 2) evaporate, 3) dissipate through evapotranspiration by plants, or 4) drain back slowly into the minor system via underdrains. The process of percolating **runoff** through the soil is an effective mechanism for both **site runoff** volume reduction and pollutant removal. Pollutants such as fine **sediment**, nutrients, bacteria, and organic materials can be filtered, absorbed by soil particles, or utilized by plants, thus providing a water quality benefit.

Since **volume control practices** reduce the quantity of **stormwater runoff** discharged from the **site**, "credit" for these practices provided in §503 may be applied to the **site** detention requirements in §504, when applicable. In other words, the total required **site** detention volume may be reduced by the volume stored within **volume control practices**. Additionally, **site detention facilities** may be modified by storing the **volume control storage** below the outlet restrictor. Credits and approaches for **volume control storage** are discussed in detail later in this section.

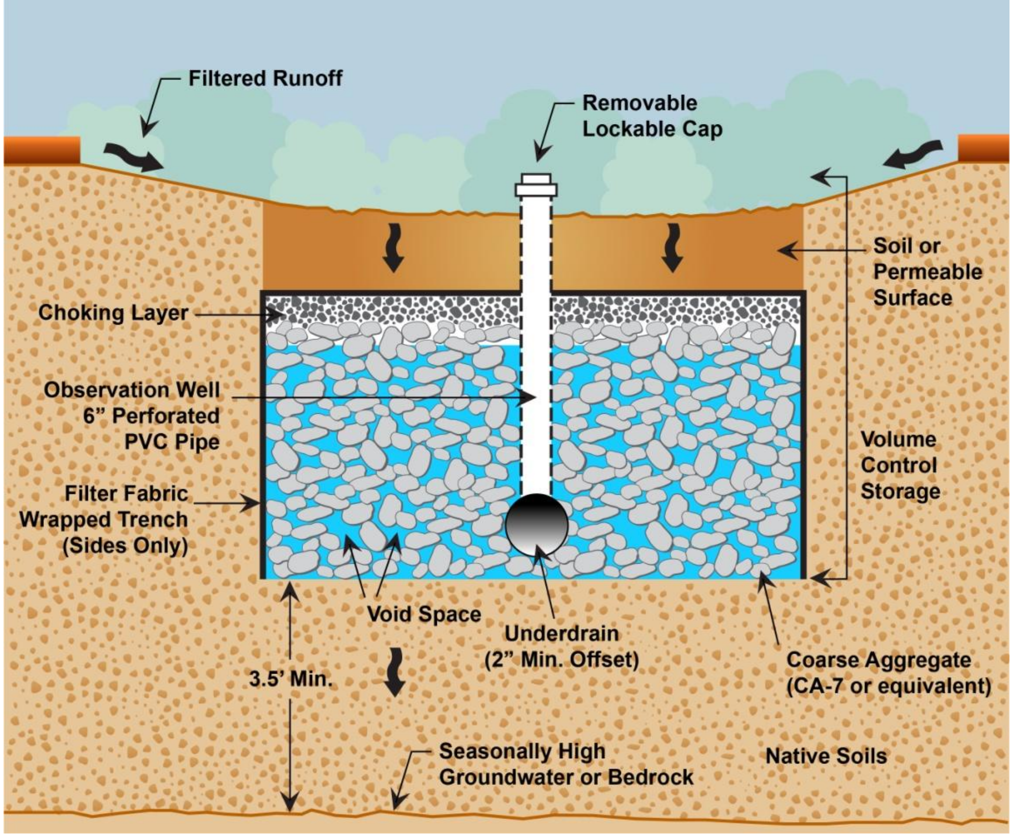


Figure 5.10. Example of Volume Control Practice

Flow-Through Practices

Flow-through practices are designed to provide water quality treatment by filtering out pollutants from the **runoff** before it is discharged from the **site**. Many **flow-through practices** provide some infiltration, however the volume reduction is not quantifiable. Many **flow-through practices** are conveyance systems that provide **stormwater** treatment along the flow path. For most practices, this is in the form of a series of vegetated swales, filter strips, or mechanical **structures** such as oil and grit separators. These practices should be sized to allow sufficient contact time with the treatment practice, such as shallow water depths and low velocities, in order for adequate pollutant removal to occur. **Flow-through practices** utilize deep-rooted plants that can trap suspended **sediment** and incorporate nutrients into their biomass as water flows through the practice.

Figure 5.11. Flow-Through Practice

For the purposes of the WMO, **flow-through practices** also serve as pretreatment practices to protect the functionality of **volume control practices**. **Flow-through practices** are not required for **stormwater runoff** that has originated from roofs.

Site Feasibility Assessment

A **site** feasibility assessment that examines **site** limitations is necessary to determine the appropriate approach for **volume control practice** design. **Volume control practices** should be located on soils that are significantly permeable to ensure that the captured volume of **runoff** can infiltrate and dewater the **structure** at a minimum rate of 0.5 inches per hour. Other considerations, such as the **groundwater** table and discharge of **volume control practice** overflows, should be examined and used in the design. Installing retention based **volume control practices** within a **floodway** is prohibited due to the risk of washout from deep and swift flood waters in these flood prone areas.

Soil Suitability

Retention-based practices require soils with appropriate infiltration capacity. The infiltration rate is strongly influenced by the proportion of sand, silt, and clay (texture). Predominately clay soils have infiltration rates that are too low (in inches per hour) to accommodate the volume of **volume control practices** and predominately sandy soils can infiltrate **runoff** too rapidly and adversely impact **groundwater**. In addition to infiltration capacity, the soils must be free of contaminants, which can also adversely impact **groundwater**. Therefore, **sites** with contaminated soils are not suitable for **volume control practices**.

Onsite soils must be tested in order to determine if they are appropriate for **volume control practices**. Testing must include a determination of the soil type(s) and the infiltration capacity, including the capacity of the soils at the base of the **structure**.

Soil borings or pits should be taken in the location of the proposed **volume control practice** to verify soil particle size distribution (textural class) and to determine the depth to **groundwater** and bedrock. The number of soil borings should be selected as needed to determine soil conditions. The minimum depth of the soil borings or pits must be five feet below the bottom elevation of the proposed **volume control practice**. This serves the purpose of determining the location of the seasonally-high **groundwater** table. Infiltration tests should be conducted at the proposed bottom elevation of the **volume control practice**. The infiltration rate must be measured with a double-ring infiltrometer and meet the requirements of ASTM D3385. For sites where the double-ring infiltrometer test is impractical, the single-ring infiltrometer test may be used, provided that the testing follows the procedure contained on Page 28 of the City of Chicago Stormwater Ordinance Manual (March 2014).

Soils must have sufficient infiltration capacity to accept the **volume control storage**. The infiltration range of onsite soils for **volume control practices** should be between 0.5 and 2.41 inches per hour. These restrictions limit the use of **volume control practices** to soils with textures of sandy loam, loam, silt loam, silt, most sandy clay loams, and only some clay loams and silty clay loams.

In the event that a natural depression is proposed to be used as a **volume control practice**, the applicant must demonstrate the following information:

1. Infiltration capacity of the soils under existing conditions (inches/hour);
2. Existing drawdown time for the high water level (HWL) and a natural overflow elevation; and
3. Operation of the natural depression under post-**development** conditions mimics the **hydrology** of the system under pre-**development** conditions.

Poor infiltration rates (< 0.5 inches/hour) are common in **Cook County** and do not prevent the use of **retention-based practices**. If onsite soils do not provide a suitable infiltration rate, the design of the **volume control practice** should incorporate the use of an underdrain system. As described in the next section, only certain site constraints (contaminated soils or high groundwater levels) are acceptable reasons for not providing **retention-based practices.**

Retention-Based Practices in Sandy Soils

Soils with large percentages of sand generally infiltrate water more quickly than finer textured soils, and therefore, are effective with retention-based practices, provided that precautions are taken to protect the groundwater. The level of treatment in sandy soils, however, is quite variable. Sands can be ideal for filtration of particulate material, whereas soluble pollutants generally move through the soil quite rapidly and unattenuated. Soil cleansing via filtration, adsorption, and microbial uptake can be very effective removal processes for some of the more difficult-to-treat runoff pollutants. However, soils that infiltrate too rapidly may not provide enough time for sufficient treatment, creating the potential for groundwater contamination.

Contaminated Sites

There are sites, such as those previously used as gas stations or sites with known contaminants (based on a Phase I Environmental Site Assessment), where it would be impractical to use retention-based practices. For these sites, the WMO volume control requirements can be met by providing **flow-through practices** or a reduction in **impervious area**.

Groundwater Analysis

An investigation into the location of the seasonally-high **groundwater** table must be carried out in order to avoid **groundwater** contamination. In **combined sewer areas**, the seasonally-high **groundwater** table must be at a minimum of 3.5 feet below the bottom of the proposed **volume control practice** to allow for treatment of collected **runoff** prior to it entering the **groundwater** system (2 feet in **separate sewer areas**). If soil borings or pits do not show the seasonally-high **groundwater** table to be within 3.5 (or 2) feet of the bottom of the proposed **volume control practice**, then further investigation is not required.

For instances where the seasonally-high **groundwater** table is within 3.5 (or 2) feet of the bottom of the proposed **volume control practice**, then the proposed **volume control practice** must be relocated or redesigned such that a minimum of 3.5 (or 2) feet is maintained.

Volume Control Practices for Site Development

The WMO (§503.3.A) requires that **volume control practices** must be sized to retain and/or infiltrate the **volume control storage**. The **volume control storage** is equal to one (1) inch of **runoff** from the impervious surfaces of the **development**. The **volume control practices** can include:

- Infiltration Trenches*
- Infiltration Basins*
- Porous Pavement (storage in the voids below the pavement)

- Bio-Retention Systems*
- Dry Wells
- Open Channel Practices Fitted With Check **Dams***
- Storage Below the Outlet of a **Site Detention facility***
- Constructed **Wetlands** that have Forebays, Deepwater Zones, and Micropools

As discussed above, pretreatment measures to protect the functionality of **volume control practices** are required where necessary. The **volume control practices** marked with an asterisk (*) will usually require pretreatment. A summary of the pretreatment measures that may be used for various volume control practices is included in Table 5-11.

Depending on the volume control practice, the storage volume may consist of surface storage, storage in the voids of growing media, and/or storage in the void space of aggregate. The aggregate layer may be incorporated into a number of practices ranging from infiltration trenches to porous pavements. Many **volume control practices** are in the form of reservoirs filled with coarse aggregate with no outlet beneath a vegetated depressional area (such as with bio-retention systems). When coarse aggregate is used, the capacity of the reservoir is determined by the void space of the coarse aggregate used in the system. If the infiltration rate of the underlying soil is less than 0.5 inches/hour, an underdrain must be used to drain the accumulated volume of **runoff**.

There can be a great deal of flexibility in the types of practices selected as well as the location and configuration of these practices onsite. For example, the dimensions (length, width, and depth) of these practices can be manipulated such that they can take on irregular shapes, thereby allowing for easier integration into the **site** design, such as along property lines, in parking lot islands, or unusable portions of the **site**. Additionally, underground storage can be provided using the stone voids under permeable pavement, and other systems can be designed to function below **impervious areas**.

Volume Control Practice Sizing and Drainage Criteria

Calculate Required Volume Control Storage

Determine the portion of the **volume control storage** that will need to be treated with **volume control practices**. The **volume control storage** is equal to one inch of **runoff** from the impervious surfaces of the **development**. This volume is best represented in cubic feet.

$$V_C = Std_C \text{ x Unit Conversion x } A_{IMPV}$$

Where: V_C = **Volume control storage** (cubic feet)

Std_C = Control Standard = 1.0 in.

Unit Conversion = 1 ft/12 in.

A_{IMPV} = Proposed **Impervious area** (ft^2)

The dimensions of the **volume control practices** will be a combination of the depth and surface area available to retain the **volume control storage**. In order to minimize the footprint of the practice, the allowable depth is often the limiting factor. The maximum allowable depth is determined by the depth necessary to maintain a 3.5 foot separation from the seasonally high **groundwater** level, bedrock, or other limiting layer (3.5 feet in **combined sewer areas**, 2 feet in **separate sewer areas**). The surface area of the **volume control practice** is determined using the design volume and final depth values.

All **volume control practices** must have quantifiable storage space to retain the calculated **volume control storage**. Depending on the volume control practice, the storage volume may consist of surface storage, storage in the voids of growing media, and/or storage in the void space of aggregate.

The capacity of an aggregate-filled reservoir to retain the **volume control storage** should be based on the volume of void space (% porosity) of the coarse aggregate used in the system, where the volume of voids is equal to the **volume control storage**. If test data is not available, use 36% porosity for the coarse aggregate. The size of an aggregate-filled reservoir can be computed by converting the **volume control storage** to reservoir volume:

$$V_{RES\ REQ'D} = V_C \text{ x } \frac{100}{\%\ \text{Void Space}}$$

Where: $V_{RES\ REQ'D}$ = Required Reservoir Volume (cubic feet) = volume of voids + volume of aggregate (This is the volume necessary to contain the coarse aggregate and the **volume control storage**.)

V_C = **Volume control storage** (cubic feet)

Perforated underdrain is required for each volume control **retention-based practice** due to the region’s typical clayey soils where infiltration rates are assumed to be low. In most cases, a required underdrain should be no larger than 4-inches in diameter to encourage retention, have an observation well installed at the terminal end, be spaced no more than 30 feet on center across a retention field area, and laid with the perforations on the bottom of the pipe. Void volume credit available below the invert of the perforated underdrain must be limited to no more than 12-inches and will be credited at 100% toward volume control requirements. Void volumes below the underdrain invert must extend across the entire volume control storage system, and not limited to only an underdrain trenched area. Void volume above the invert of the underdrain up to the ground surface will be deducted by a factor of 50% to account for losses out of the underdrain. When calculating the storage volume in aggregate a void volume of 0.36 is used. A void ratio of 0.25 is used for growing media.

Volume provided above the ground surface will be limited in depth to 12-inches of wetland ponding and credited at 100%. For calculating surface **volume control storage**, the average end area method between the ground elevation and the elevation of the overflow grate/outlet pipe/check dam is recommended. Wet ponding depths above one foot of depth are not considered volume control and are considered impervious areas. For more information, see the **volume control practice** details located in Appendix C of the **TGM.**

If high infiltrating soils are suspected, provide a detailed soils report documenting the additional in situ percolation tests to confirm actual infiltration rates and to allow for design of a volume control facility without underdrains and take credit for additional infiltrative release.

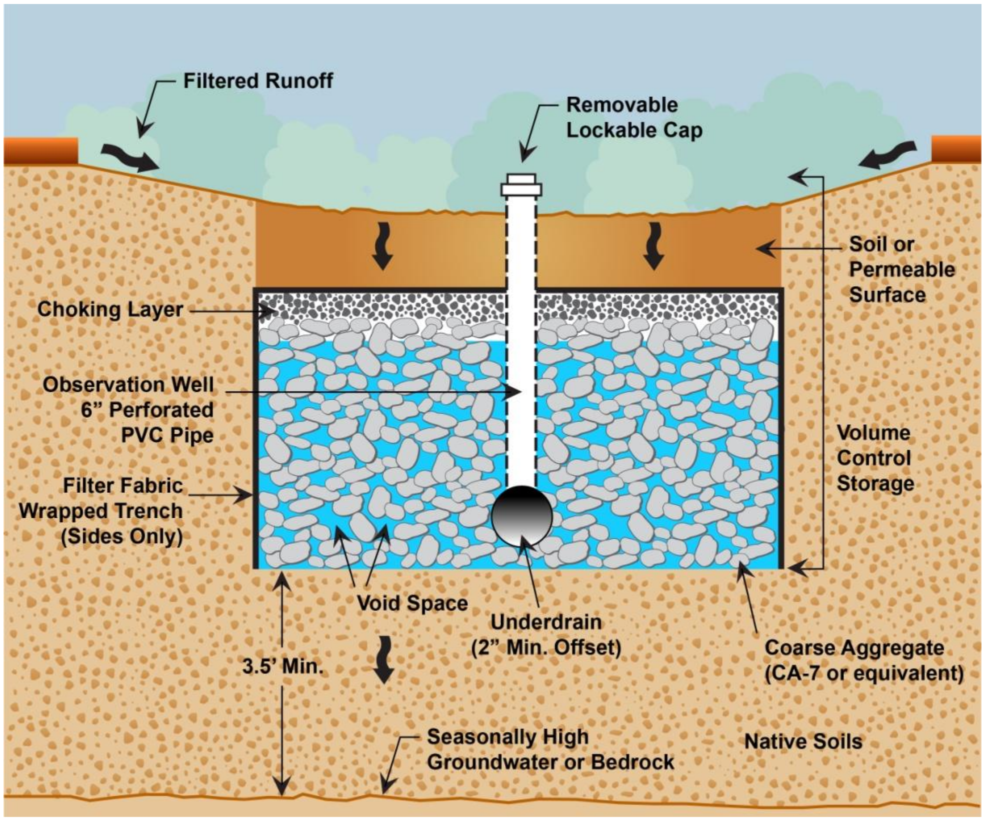


Figure 5.12. Volume Control Practice – Surface Storage, Media Storage and Aggregate Storage

Volume Control Practice Site Location

Determine how **volume control practices** will fit into the **development site** design (on available pervious area of the **development**), and select the appropriate **volume control practice**. Information collected during the **site** feasibility assessment should identify the potential for multiple **volume control practices** versus relying on a single **volume control practice**. Again, **volume control practices** should be located on soils that are significantly permeable to ensure the captured **volume control storage** can infiltrate and dewater the **structure** at a minimum rate of 0.5 inches per hour.

The following should be considered when determining the location of **volume control practices**:

1. Conveyance path of the **runoff** to the practice;

2. Overland flow path from the practice to the main drainage system;

3. Practices should be a minimum of 10 feet from a **building** foundation (unless waterproofed), 20 feet from a **sanitary sewer**, and 100 feet from potable water wells and septic tanks;

4. Practices should not be installed on slopes greater than 15;

5. Practices should not be installed above soils that are considered fill; and

6. A minimum setback of 20 feet from a road's gravel shoulder is required to ensure that the practices do not cause frost heaving.

Volume Control Storage Design

The **volume control practice** should be filled with coarse aggregate that meets **IDOT** Section CA-7 quality and gradation. Other types of coarse aggregate will be permitted, provided that it is crushed angular stone that is clean and washed free of fines. A void ratio of 0.36 is used when calculating the storage volume in aggregate. For growing media, a void ratio of 0.25 applies. A layer of choking stone or filter fabric must separate the growing media from thecoarse aggregate, and/or sand layers. No filter fabric shall be placed along the bottom of the trench between the volume control media and the native subsoil. Filter fabric must wrap along the trenching sides to prevent soils migration from clogging the system. The filter fabric should meet the requirements of the ***Illinois Urban Manual*** *Material Specifications 592* for geotextile fabric.

The storage reservoir should have direct access for **maintenance activities**. An observation well (e.g., a perforated PVC pipe that leads to the bottom of the **structure**) is needed to enable inspectors to visually monitor the drawdown rate of the water. For more information, see the observation well detail located in Appendix C of the **TGM.** One well per 40,000 ft^2 of practice surface area is required. Where infiltration rates of the soil are less than 0.5 inches per hour, v**olume control practices** must incorporate an underdrain pipe that will allow the **structure** to be dewatered within 72 hours or less, if the **structure** becomes clogged. An underdrain can be a perforated pipe system in a gravel bed, installed at the base of the **structure** (minimum of 2" and maximum of 12" from the bottom) to collect and remove filtered **runoff**. The period of inundation is defined as the time from the high water level in the practice to one to two inches above the bottom of the facility (see Figure 5.12). This criterion was established to provide:

1. Wet-dry cycling between rainfall events;

2. Unsuitable mosquito breeding habitat;

3. Suitable habitat for vegetation;

4. Aerobic conditions; and

5. Storage for back-to-back precipitation events.

Additional details and specifications for the design of **volume control practices** are provided in this article and in Appendix C of the **TGM**:

- Underdrain
- Coarse aggregate
- Filter fabric
- Monitoring well
- Turf fields

Overflow Path from Volume Control Practice

In addition to a conveyance design that routes flows to a **volume control practice**, an equally important consideration is a conveyance design that routes flows from the practice back to the main drainage system. The overflow path is a necessary component designed to prevent structural damage to the **volume control practice** from localized **flooding** in the event that the practice does not dewater fast enough to prevent an overflow. Overflows can occur as a result of clogging or during long-duration, high-intensity **storm events** that raise the **groundwater** level to an elevation that impedes infiltration. Therefore, overflow designs should route excess flows through a **stabilized** discharge point that allows these flows to be directed back to the main drainage system in a controlled manner that will not cause scour.

Protection of Volume Control Facilities During Construction

Volume control practices are susceptible to failure during construction and therefore it is important that staging, construction practices, and **erosion and sediment control practices** all be considered during their installation. To protect the long-term functionality of volume control practices, the following measures should be addressed in the construction sequencing, general notes, and/or **soil erosion and sediment control** plan for a **development**:

- **Volume control practices** should be installed toward the end of the construction period.
- The contributing **drainage area** must be stabilized prior to the installation of the **volume control practice**.
- Soil compaction shall be minimized as much as possible during **site** grading. Appropriate measures (such as fencing) should be used to prevent heavy construction equipment traffic from accessing the area.
- **Volume control facilities** must be protected with a double-row of silt fence (or equivalent measure) during construction. The two layers of silt fence should be placed at least 5 feet apart and must follow the ***Illinois Urban Manual*** standards.

- In general, **volume control facilities** should not be used as temporary sediment traps during construction. For **sites** where this is not practicable, special construction notes and/or details are required to protect the functionality of the facility.

Protection of Volume Control Practice Infiltration Capacity: Pretreatment

Pretreatment is critical for **runoff** entering **volume control practices** in order to prevent clogging within the **volume control practice**. This reduces **maintenance** and also provides an added level of protection against **groundwater** contamination. §503.3.A (3) of the WMO requires, where necessary, pretreatment of **runoff** entering a **volume control practice** to protect the functionality of the **structure**. Where practicable, **flow-through practices** such as vegetated swales or filter strips should be used to meet the pretreatment requirement. Additionally, upland drainage should be properly **stabilized** both during and after construction to reduce **erosion**, thus minimizing the **sediment** loads being delivered to the **structure**. The use of trash racks or downspout screens will also satisfy the pretreatment requirements in some cases, as these measures prevent debris from clogging the **volume control practice**. Table 5-11 provides a summary of pretreatment measures that may be used for various volume control practices.

Flow-through Practices for Site Development

The WMO (§503.3.B) requires **flow-through practices** for treatment of any portion of the **volume control storage** that has not been treated using **volume control practices**. **Flow-through practices** must be sized to filter or detain the **volume control storage** as it passes through the **structure**. Maximizing the contact time between the vegetation and the **runoff** is critical to the effectiveness of **flow-through practices** to provide adequate treatment. Vegetation selection varies depending on climate, soil type, topography, land use, available light (shade tolerance), aesthetics, and planned use of the area.

Flow-through practices include, but are not limited to:

- Vegetated Filter Strips;
- Bio Swales;
- Constructed **Wetlands**;
- Catch Basin Inserts; and
- Oil and Grit Separators.

Again, any of these practices are also a suitable form of pretreatment for **volume control practices** and **site detention facilities** (detention ponds). However, these practices are not appropriate for all **sites** due to several limitations, particularly in **redevelopment** areas.

Providing vegetated **flow-through practices** may be difficult in many **redevelopment** areas due to the lack of ideal soils capable of supporting hearty vegetative growth. Many soils have undergone significant compaction and nutrient loss, which can limit root **development** and proper drainage. **Flow-through practices** can also have the potential to interfere with existing infrastructure and practice design should be considered accordingly.

Flow-through Practice Sizing and Criteria

Calculate Volume Control Storage for Flow-Through Treatment

Determine the portion of the **volume control storage** that will need to be treated with **flow-through practices**. The **volume control storage** is equal to one inch of **runoff** from the impervious surfaces created by the **development**. If a portion of the **volume control storage** is to be treated by **volume control practices**, subtract that portion from the **volume control storage** to determine the volume to be treated by **flow-through practices**.

- First, Calculate Volume to be Treated Using a **Volume Control Practice:**

$$V_{C\ RET} = V_C \times \%_{RET}$$

Where: $V_{C\ RET}$ = Portion of **volume control storage** in **Volume Control Practice** (ft^3)

V_C = **Volume control storage** (ft^3)

$\%_{RET}$ = Portion of **volume control storage** in **Volume Control Practice** (%)

- Then, Calculate Volume to be Treated Using a **Flow-Through Practice**

$$V_{C\ FLW} = V_C - V_{C\ RET}$$

Where: $V_{C\ FLW}$ = Portion of **volume control storage** in **Flow-Through Practice** (ft^3)

V_C = **Volume control storage** (ft^3)

$V_{C\ RET}$ = Portion of **volume control storage** in **Volume Control Practice** (ft^3)

Volume Control Examples

This section provides examples of the five most common **volume control practices** that have been utilized in **Cook County**. There are additional acceptable **volume control practices** that have not been addressed in the **TGM**. These practices are itemized in §503.3.B of the WMO.

Porous (Permeable) Pavement

The concept of porous pavement is to allow rainwater to infiltrate into and through the surfaces of parking lots, streets, and other traditional impervious surfaces. When designing a porous surface, the designer must carefully evaluate where the infiltrated rainwater is draining and how the **stormwater** is being conveyed.

The main benefits of porous pavements are increased **stormwater** infiltration, decreased surface **runoff**, improved water quality, and reduction in **runoff** velocity. Porous pavements are particularly important in filtering the first flush pollutants commonly observed at the beginning of a **storm event**. First flush pollutants are present on the land surface before the **storm event** and typically include car oil, gasoline, trash, road salt and suspended solids.

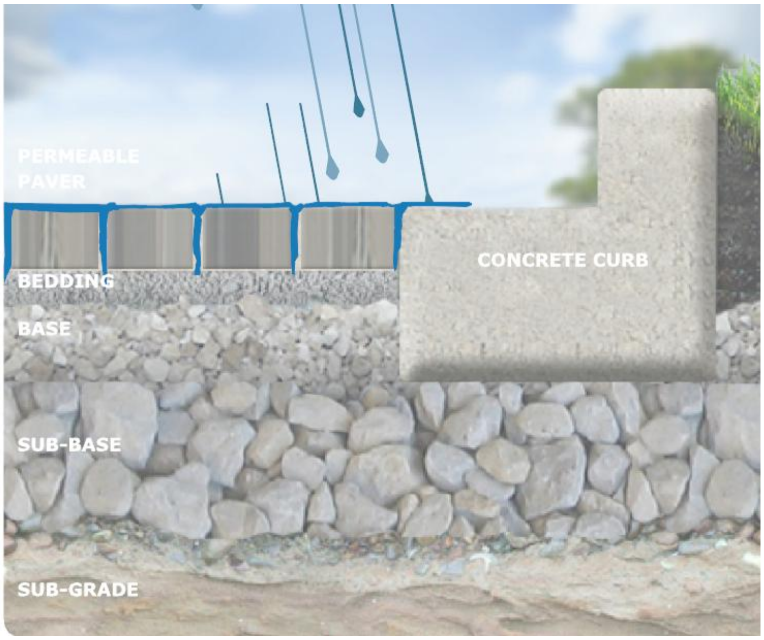


Figure 5.13. Example of a Permeable Paver Parking Lot Cross-Section (APT, 2011)

Design Considerations:

- There are several pavement options: pervious concrete, permeable pavers, and porous asphalt (not preferred).
- Must be sized and designed based on **drainage area**, structural requirements, soils, and the **volume control storage**.
- Underdrains may be used to provide drainage unless infiltration rate is greater than 0.5 inches/hour.
- **Maintenance** is necessary to ensure long-term functionality. **Maintenance** procedures include: sweeping organic materials off of gravel-filled pavers, and conventional street-sweeping with vacuums, brushes, and water to clear out voids (aggregate fill may be needed following each cleaning to refill the voids). Schedule R and Exhibit R must be submitted for the volume control facility, as well as a detail drawing of proposed signage, as required.
- This practice should be used with caution in areas underlain with highly permeable soils (i.e., surface sand or gravel) where infiltrated pollutants could reach **groundwater** without opportunity for attenuation.
- The effects of subgrade compaction, freeze-thaw cycles, de-icing, and snow removal must be considered in determining the applicability of this practice.
- The bottom should be at least 3.5 feet above the seasonal high water table (in **combined sewer areas**, 2 feet in **separate sewer areas**) and as level as possible in order to uniformly distribute infiltration to the surrounding soil.

For additional design considerations for porous pavement, the **Illinois Urban Manual** practice standard is available on-line at: http://aiswcd.org/IUM/standards/urbst890.html.

Dry Wells

A dry well consists of an excavated area which is backfilled with aggregate to temporarily store and infiltrate **stormwater runoff** from rooftops. Their typical application is for single family residences. The purpose of the dry well is to reduce **runoff** volume and peak discharges from a **development**. They also have the ability to filter soluble contaminants out of the **stormwater runoff**.

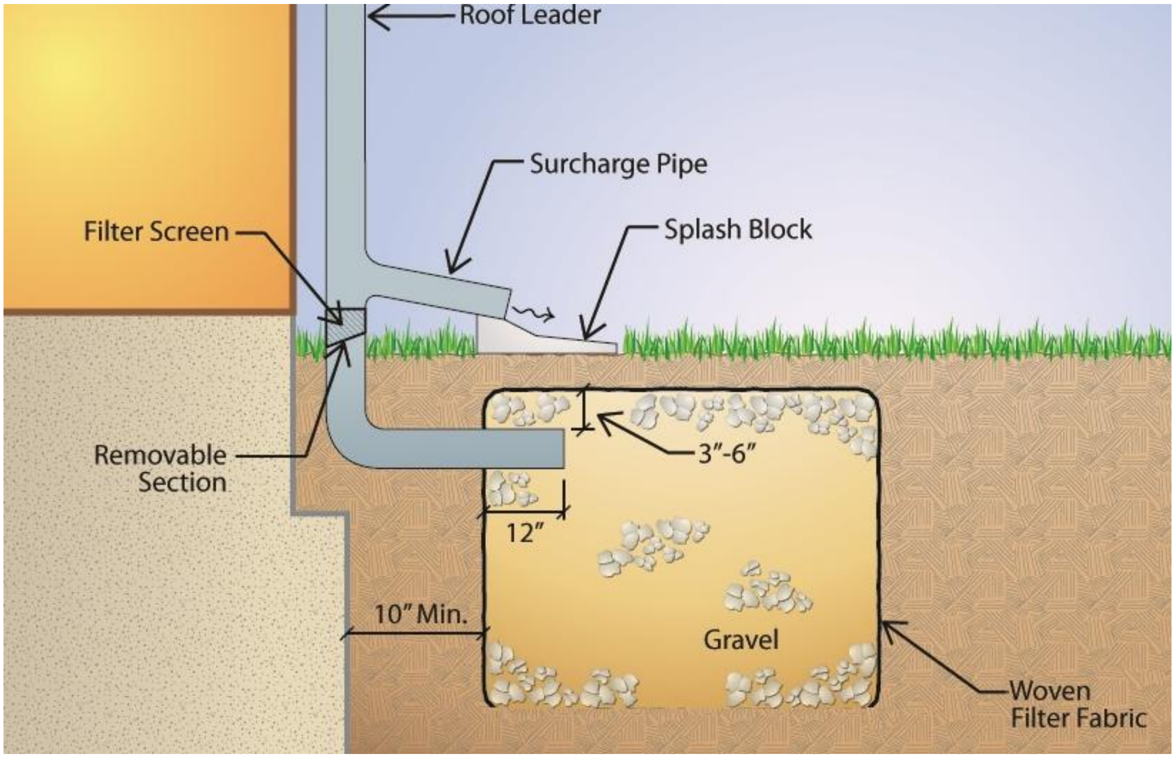


Figure 5.14. Typical Cross-section for Dry Well

Design Considerations:

- Must be sized and designed based on the **drainage area**, soils, and **volume control storage**.
- Dry wells can be constructed in two different forms: either a structural chamber that is assembled or inserted into an excavated pit, or an excavated pit filled with aggregate.
- It is important that the location of the dry well is adequately placed so that it does not cause **basement** seepage, **flooding**, or ponding at the ground surface.
- Dry wells should drain accumulated volume within 72 hours.
- They must be sized with consideration of both **drainage area** (1 acre maximum) and soil type (sandy soils will drain much more quickly than clay dominated soils).

- The bottom of the well should be at least 3.5 feet above the seasonal high water table (in **combined sewer areas**, 2 feet in **separate sewer areas**) and as level as possible in order to uniformly distribute infiltration to the surrounding soil.
- Dry wells should be protected from construction **site runoff** to prevent clogging.
- Dry well use is restricted by concerns of **site** feasibility, soil types, clogging, seasonally high **groundwater**, and bedrock.

For additional design considerations for dry wells, the **Illinois Urban Manual** practice standard is available on-line at: http://aiswcd.org/IUM/standards/urbst847.html.

Bio-retention System

Bio-retention systems consist of landscaped areas that are designed to intercept, infiltrate, and store **stormwater runoff** from the **site**. A permeable soil layer allows **stormwater runoff** to infiltrate to a layer of coarse aggregate, where **stormwater** can be stored in the void space of the stone. Bio-retention systems provide surface storage (between the ground elevation and the elevation of the overflow grate), storage in the void space of the growing media, and storage in the void space of aggregate.

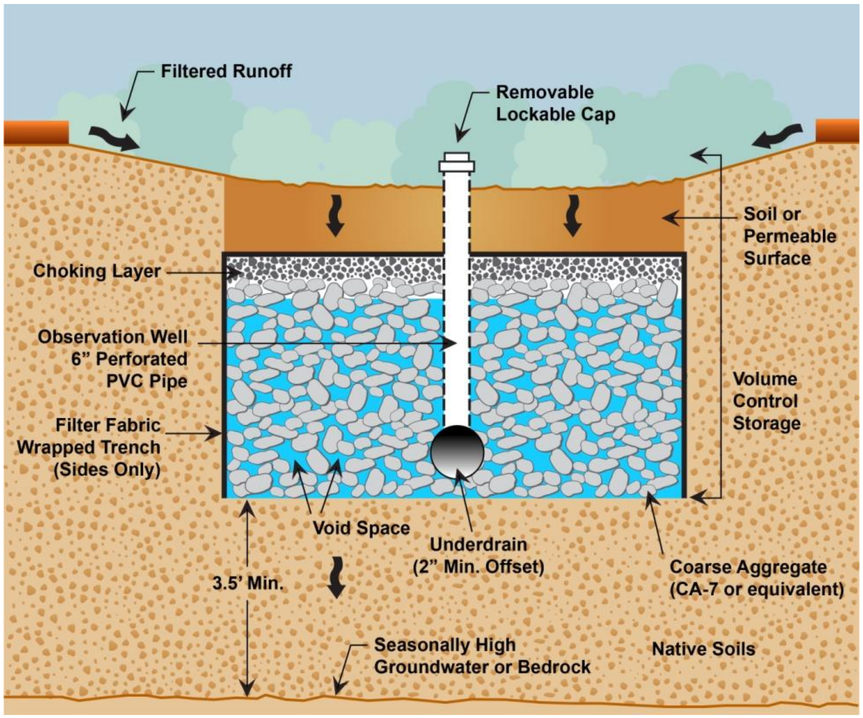


Figure 5.15. Typical Cross-section for Bio-retention System

Design Considerations:

- Bio-retention Systems are commonly located in parking lot islands or as small pockets within residential land uses.
- **Runoff** can be drained to Bio-retention Systems using curb cuts or wheel stops.

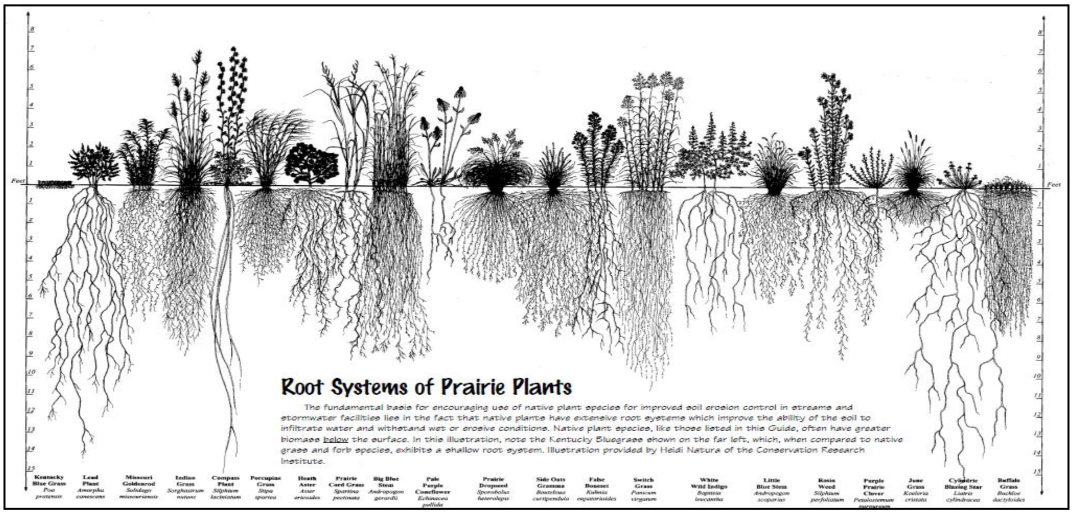
- They must be sized and designed based on **drainage area**, soils, and the **volume control storage**.
- Generally, a one- to three-foot gravel layer wrapped in a woven geotextile provides temporary **stormwater** storage. However, this may vary depending on the amount of storage needed to meet the volume control requirement.
- Mix should be 50% sand, 30% organic (e.g. aged composted leaf mulch), 20% high quality topsoil (minimal clay content). Several District mixes, which vary by the underlying soil type, are also acceptable and consist of the following:

    <u>Mix-1, for Area Where Native Soil is Clay</u>
    50% sand, 50 % **District** composted biosolids or any other compost
    (Incorporate in top 4-inches)

    <u>Mix-2, for Areas Where Native Soil is Sandy</u>
    40% top soil, 60% **District** composted biosolids or any other compost
    (Incorporate in top 4-inches)

    <u>Mix-3, for Areas Where Native Soil is Loamy</u>
    25 % Sand, 75 % **District** composted biosolids or any other compost
    (Incorporate in top 4-inches)

- The mulch layer should consist of shredded hardwood mulch (commercial), **District's** unscreened composted biosolids, or another non-floating mulch.
- During smaller **storm events**, **runoff** filters through the mulch and prepared soil mix is collected in a perforated underdrain and returned to the storm drain system. **Runoff** from larger storms is generally diverted past the bio-retention system to the **storm sewer** system.
- Additional design details are available on-line at: http://www.stormwatercenter.net/.



The fundamental basis for encouraging use of native plant species for improved soil erosion control in streams and stormwater facilities lies in the fact that native plants have extensive root systems which improve the ability of the soil to infiltrate water and withstand wet or erosive conditions. Native plant species, like those listed in this Guide, often have greater biomass <u>below</u> the surface. In this illustration, note the Kentucky Bluegrass shown on the far left, which, when compared to native grass and forb species, exhibite a shallow root system. Illustration provided by Heidi Natura of the Conservation Research Institute.

Figure 5.16. Root Systems of Grass and Prairie Plants (Source: Heidi Natura, CRI, 1995).

Water Reuse Systems

Water reuse systems consist of structures that are designed to intercept and temporarily store stormwater runoff. These systems are beneficial because they capture stormwater runoff and allow it to be used for irrigation, which promotes infiltration of that stored water following a storm event. If a storage system does not contain a water reuse application, it does not qualify as a volume control practice since there is no infiltration.

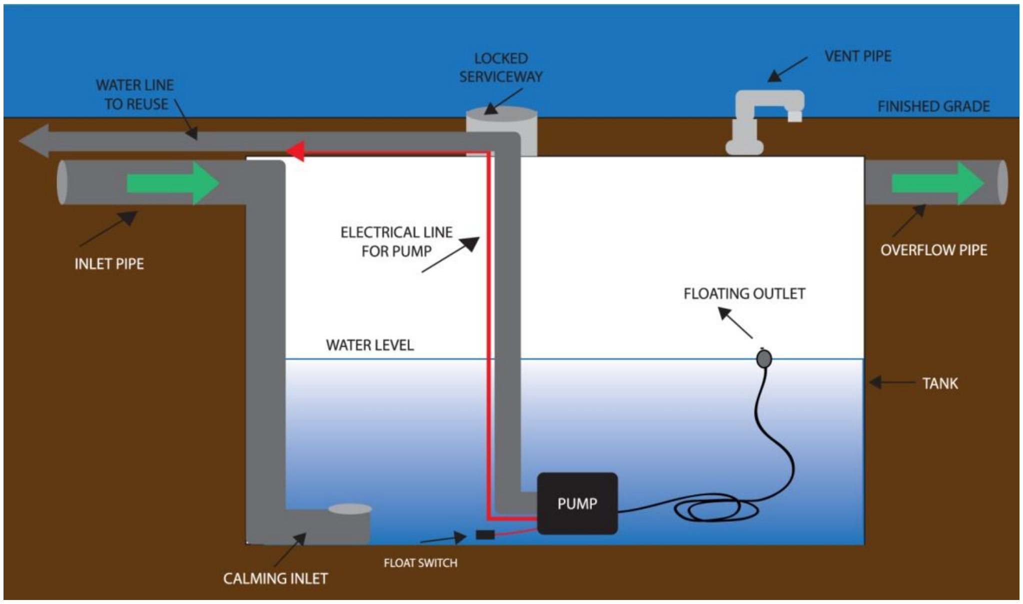


Figure 5.17. Typical Water Reuse System (Stormwatersmart.org, 2015)

Water reuse systems can either be above-ground or underground, and may be gravity-drained or pump-evacuated. A typical underground water reuse system is shown as Figure 5.17. There are several common variations of these systems available that include:

- Rain barrels
- Rain cisterns (above-ground and underground)
- Underground storage (tanks, vaults, or other manufactured products)

Design Considerations:

- Water reuse systems are commonly used to intercept and store stormwater runoff from rooftops, but can be designed with other areas such as parking lots.
- Placement of water reuse system in the up-gradient portions of a site may eliminate or reduce the need for pumping.
- An overflow pipe must be provided to bypass large storm events through the system.

- To ensure that the storage is available for the next storm event, the system should be designed so that it completely drains within 72 hours. If a low-flow pump is used to dewater the facility, an operation plan should be provided that follows this dewatering schedule.
- Overflow conditions should be considered when prescribing an offset from building foundations. The minimum setback for water reuse systems should be 10 feet from the nearest building foundation (unless waterproofed).
- Pretreatment measures for water reuse systems consist of screens and/or trash racks to filter debris from incoming stormwater runoff.

Green Roofs

A green roof is a conventional rooftop that includes a covering of vegetation which allows it to act like a pervious surface instead of an impervious one. There are two types of green roofs: intensive and extensive. Extensive green roofs involve a shallow growing medium layer (typically four inches or less) and therefore support plants with shallow root systems, such as herbs, grasses, moss, and sedum. Intensive green roofs include a deeper growing medium layer (typically between 4 and 12 inches) that can support plant species with deeper root zones, including trees and shrubs. Intensive green roof systems are generally limited to flat roofs and require significantly more maintenance than extensive green roof systems.

The overall thickness of a green roof may range anywhere from two inches to 12 inches, and consists of multiple layers that include: planting layer (native vegetation), growing medium layer, geotextile fabric, drainage layer, insulation, membrane protection and root barrier, and structural supports. An example cross-section of a green roof is provided as Figure 5.18 below.

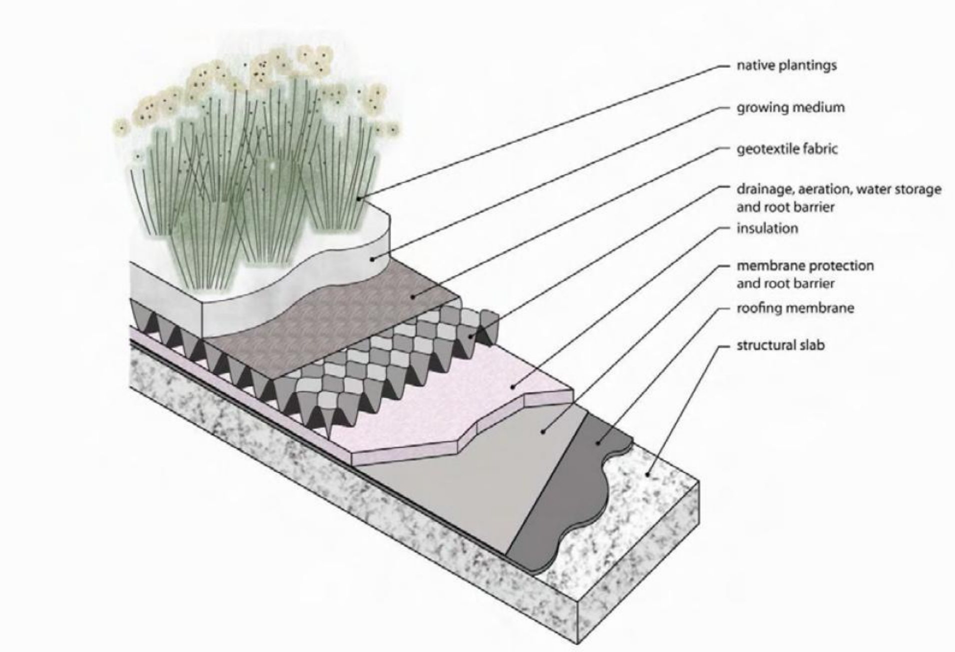


Figure 5.18. Typical Green Roof Cross-section (Source: City of Chicago, 2014).

Green roofs provide runoff storage volume in the void space of both the growing medium layer and the drainage layer. Therefore, the runoff volume reduction potential of a green roof is a function of the thickness of its growing medium and drainage layers. Table 5-9 provides a summary of the curve number and available volume control storage for a variety of media thicknesses. For calculating volume control storage, a void ratio of 0.25 should be used for the growth medium layer and also the drainage layer (typically pea gravel).

Table 5-9. Summary of Curve Numbers and Volume Control Storage for Green Roofs

Media Depth* (inches)	Void Ratio	Reduced CN	Reduced Runoff Coefficient, C	Volume Control Storage (ft^3/ft^2 of Green Roof)
0	---	98	0.90	---
2	0.25	94	0.83	0.042
4	0.25	90	0.74	0.083
6	0.25	85	0.66	0.125
9	0.25	79	0.54	0.188
12	0.25	72	0.40	0.25
>12	0.25	63	0.10	>0.25

*Media Depth includes growing medium layer and drainage layer

Design Considerations:

- Roofs with slopes greater than 45° are typically not suitable for a green roof system.
- Careful attention and additional maintenance are necessary during the first two growing seasons to ensure establishment and proper function as a volume control system.

- Access should be considered for ease of inspection and maintenance.
- The load-bearing capacities of green roofs must be verified by a licensed structural engineer and architect (design plans must be sealed by either). Roof structure must be able to support snow loads in addition to green roof loading.
- A minimum setback of two feet is required from the roof perimeter and all roof penetrations (e.g. water connections, building parts for the usage of roof area, etc.)
- Growth media should consist of 80% lightweight inorganic materials and 20% organic matter.
- Native plants should be selected according to ASTM E2400-06, *Guide for Selection, Installation and Maintenance of Plant for Green (Vegetated) Roof Systems*.
- If vegetation consists of drought-resistant plants, irrigation is usually only necessary during the plant establishment period. Otherwise, an irrigation system is a typical component of a green roof system (water reuse system may be used for irrigation).
- Pretreatment measures are not required for green roof systems.

Filter Strip

Filter strips are vegetated sections of land that treat sheet flow from adjacent **impervious areas**. Filter strips are beneficial because they remove pollutants from **stormwater** before they reach the receiving **storm sewer** system. Filter strips may provide some reduction in **stormwater runoff** volume, but their primary function is to filter out contaminants in **stormwater runoff**. Since they do not provide any quantifiable storage, the use of filter strips is appropriate as a **flow-through practice** only.

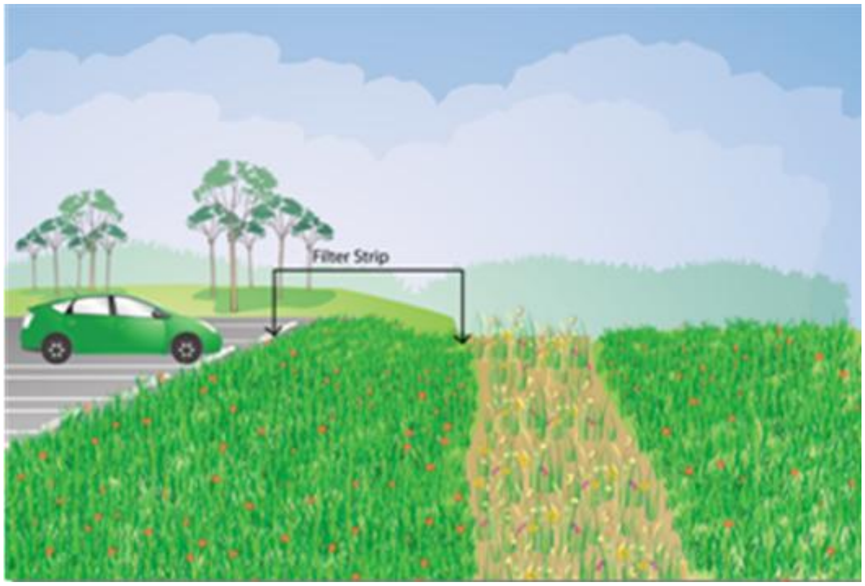


Figure 5.19. Illustration of a Filter Strip

Design Considerations:

- Filter strips are suitable for draining areas that are five acres or less.
- The minimum length of the filter strip may be determined by the type of vegetative cover, permeability of the soil present, and slope of the filter strip. In general, filter strips should be no less than 30 feet in length and should not exceed 100 to 150 feet in length, as sheet flow will concentrate and cause **erosion**.
- Longitudinal slopes for filter strips should be between 2 and 5%, but can be up to 10%. The slope should be uniform throughout the strip to maintain sheet flow.
- Since concentrated flows entering a filter strip can cause **erosion**, a level spreader may be required at the top of the slope.

For additional design considerations, the **Illinois Urban Manual** practice standard is available on-line at: http://aiswcd.org/IUM/standards/urbst835.html.

Vegetated Swale

Vegetated swales are shallow earthen channels that are designed to slow **stormwater runoff** and promote infiltration. Similar to filter strips, vegetated swales intercept **stormwater runoff** from nearby **impervious areas**. Their primary function is to filter pollutants and **sediment** from **stormwater runoff**. Since they do not provide any quantifiable storage, the use of vegetated swales is appropriate as a **flow-through practice** only.

Vegetated swales may be combined with non-infiltration related storage volume to provide the required **volume control storage**. For example, an underground concrete vault (concrete bottom) may be used to provide the one inch of volume over the proposed **impervious area**. The vault may be pump-evacuated to a **flow-through practice** (such as a vegetated swale) to treat the volume. For configurations such as these, the storage volume component of volume control is provided in the vault, while the infiltration and pollutant removal component is provided in the **flow-through practice**. These facilities must be operated and maintained in a manner that maximizes the availability of the provided storage volume.

Figure 5.20. Typical Cross-section for Vegetated Swale

Design Considerations:

- Vegetated swales can be applied in most **development** situations with few restrictions. They are well-suited to treat highway or residential road **stormwater runoff** due to their linear nature.
- They must be sized and designed based on **drainage area**, soils, and the **volume control storage**.
- To maximize the wetted perimeter, side slopes of 4:1 or flatter are recommended. Side slopes should not exceed a 3:1 ratio.
- Longitudinal channel slopes should range from as close to zero as drainage permits to 4%. Slopes greater than 4% can be used if check **dams** are used to reduce flow velocity.
- Additional design details are available on-line at: http://www.stormwatercenter.net/.

Example 5.5 – Calculation of Required Volume Control

A five acre commercial area is proposed in a **combined sewer area** with the land use described below. The required volume control is to be provided by a permeable parking lot. The average surface elevation is at 636.0 feet and the **groundwater** elevation has been determined to be at 631.0 feet.

- Area of **building** = 2.3 acres
- Area of permeable parking lot = 1.5 acres
- Area of dry-bottom detention pond = 0.3 acres
- Landscaping = 0.9 acres

- Volume of voids in the stone below the permeable pavement is 36%
- Depth of permeable paver and settling base = 9 inches
- Infiltration rate of underlying soil = 0.3 inches/hour

The required **volume control storage**, V_c, is calculated as follows:

$$V_c = \quad 2.3 \text{ acres} \times \frac{43{,}560 \text{ ft}^2}{\text{acre}} \times 1'' \textbf{ runoff} \times \frac{\text{ft}}{12''} = 8{,}349 \text{ ft}^3$$

Note that the permeable pavement is not included in the **impervious area** calculation.

The first step is to calculate the volume that is provided in the 2-inch (minimum) offset between the bottom of the facility and the invert of the underdrain, over the 1.5-acre parking lot. The void space storage in this location is credited at 100%, and is calculated by the following:

$$2 \text{ inches} \times \frac{1 \text{ ft}}{12 \text{ inches}} \times 1.5 \text{ acre} \times \frac{43{,}560 \text{ ft}^2}{1 \text{ acre}} \times 0.36 = 3{,}920 \text{ ft}^3$$

Therefore, a storage volume of 4,429 ft^3 must be provided above the underdrain invert (8,349 ft^3 – 3,920 ft^3). Since the storage volume in this location is only credited at 50%, the required volume and depth of stone above the underdrain invert can be calculated by:

$$\frac{4{,}429 \text{ ft}^3}{0.36 \times 0.50} = 24{,}606 \text{ ft}^3 \times \frac{1}{1.5 \text{ acre}} \times \frac{1 \text{ acre}}{43{,}560 \text{ ft}^2} = 0.38 \text{ ft}$$

Since the required depth of stone is now known, the design can be checked against the site constraints and WMO requirements:

Depth to the bottom of stone:

636.0 ft – (9 in/12) – 0.38 ft – (2 in/12) = 634.70 ft

Depth from the bottom to **groundwater**:

634.70 ft – 631.0 ft = 3.70 ft → OK > 3.5 ft

See Figure 5.21 below for the configuration of the proposed permeable pavement parking lot, as determined in Example 5.5.

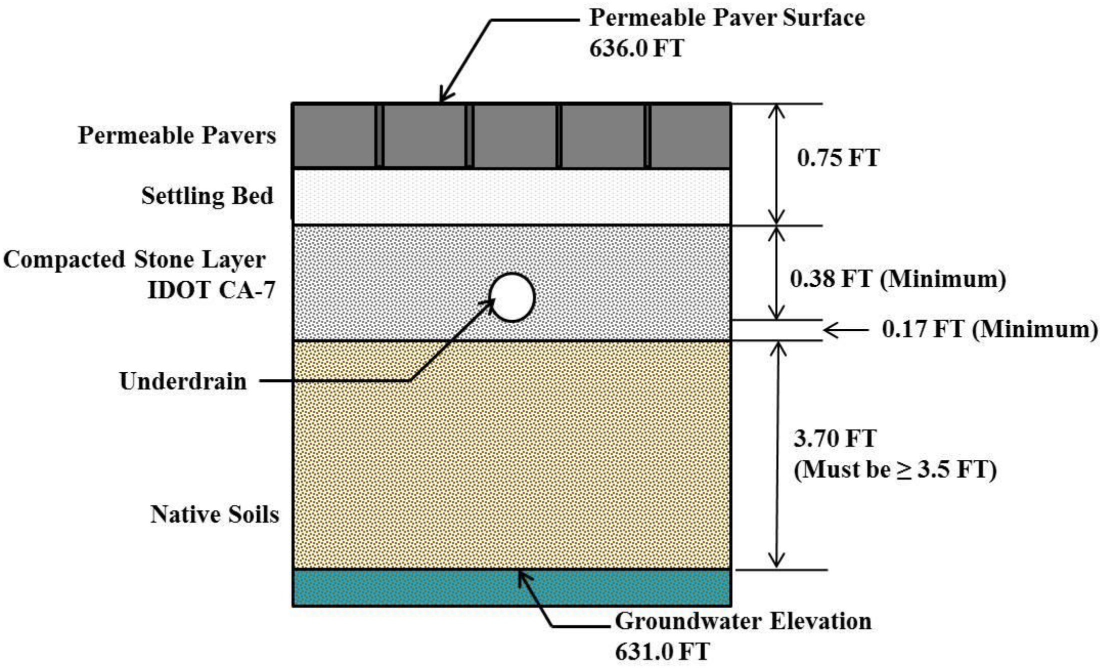


Figure 5.21. Configuration of Volume Control Storage for Example 5.5

While this design provides the required **volume control storage** and satisfies the WMO offset requirements for **groundwater**, there are several alternative designs that could also work. Since the void storage below the underdrain invert is credited at 100%, the offset from the bottom could be increased (instead of the minimum 2 inches) to provide more **volume control storage**. This would reduce the depth of stone needed above the underdrain invert. Alternatively, the depth of stone could also be reduced by expanding the area of the permeable pavement parking lot.

Other Design Requirements:

- Because the infiltration rate of the existing soils is less than 0.5 inches/hour, an underdrain must be provided for the storage. The underdrain must be offset a minimum height of 2 inches (0.17 feet), with a maximum allowable offset of 12 inches (1 foot) from the bottom.
- Because this example is located in a **combined sewer area**, the bottom of the volume control storage must be at least 3.5 feet above the seasonal **groundwater** elevation. In **separate sewer areas**, the requirement is 2 feet.
- The permeable pavement storage will need two monitoring wells (1 per 40,000 ft^2).
- In this example, the aggregate was assumed to be **IDOT** CA-7. Other aggregate sizes may be used for **volume control practices**, provided that it is crushed, angular stone that is cleaned and washed free of fines. Since the available void space will vary with aggregate size, extra care must be taken in the volume control storage calculations.

- The permeable pavement should be designed to slope towards a drainage **structure** such as an inlet so overflows can be captured. This **structure** can also be the **structure** that the underdrain connects to. Since the only **impervious area** not draining to the **volume control practice** is the roof area, no **flow-through practice** is required.

Table 5-10. Summary of Storage Calculations for Volume Control Practices

Volume Control Practice	Void Space of Aggregate[1]	Surface Storage[2]	Growing Media[3]
Bioretention Facility	**X**	**X**	**X**
Bioswale[4]	**X**	**X**	**X**
Constructed Wetlands	**X**	**X**	**X**
Drywell	**X**		
Green Roof	**X**		**X**
Infiltration Trench	**X**		
Permeable Pavement	**X**		
Storage Below Detention Basin Outlet		**X**	
Vegetated Filter Strip (Flow-Through)			
Water Reuse System		**X**	

[1] A void ratio of 0.36 shall be used to calculate volume in CA-1 or CA-7 gradations, 0.25 for pea gravel or CA-16 (volume above underdrain credited at 50%)

[2] Storage calculated using average-end method between surface elevation and elevation of overflow grate/check dam/outlet pipe

[3] Porosity of 0.25 shall be used to calculate volume in growing media (volume above underdrain credited at 50%)

[4] Surface storage only if check dams are installed

Table 5-11. Summary of Pretreatment Measures for Volume Control Practices

Volume Control Practice	Pretreatment Measures
Bioretention Facility	• Level spreader must be installed where runoff enters the facility as shallow concentrated flow to distribute the runoff as sheet flow over the entire facility. • Vegetated filter strip, grass-lined channel, or sump must be installed upstream of the facility to filter out settleable particle and floatable materials. • Where inflow velocities are greater than 3 ft/s, a vegetated filter strip or rock outlet protection must be installed to prevent erosion and distribute flows across the facility. • Vegetated portions of the contributing drainage area must be stabilized.
Bioswale	• Level spreader must be installed where runoff enters the facility as shallow concentrated flow to distribute the runoff as sheet flow over the entire facility. • Vegetated portions of the contributing drainage area must be stabilized.
Constructed Wetlands	• Where inflow velocities are greater than 3 ft/s, rock outlet protection should be provided to prevent erosion and distribute the flows into the facility. • Vegetated portions of the contributing drainage area must be stabilized. • Sediment forebay shall be installed upstream of the facility.
Drywell	• Filter screens must be installed on all roof drains directed toward the facility. • For facilities that include inflow pipes, sump and/or trash rack shall be installed at manhole immediately upstream of facility.
Green Roof	• No pretreatment measures required.
Infiltration Trench	• Level spreader must be installed where runoff enters the facility as shallow concentrated flow to distribute the runoff as sheet flow over the entire facility. • Vegetated filter strip, grass-lined channel, or sump must be installed upstream of the trench to filter out settleable particle and floatable materials. • Where inflow velocities are greater than 3 ft/s, a vegetated filter strip or rock outlet protection should be provided to prevent erosion and distribute flows across the facility. • Vegetated portions of the contributing drainage area must be stabilized.
Permeable Pavement	• Vegetated filter strip, grass-lined channel, or sump must be installed upstream of the facility to filter out settleable particle and floatable materials. • Vegetated portions of the contributing drainage area must be stabilized.
Storage Below Detention Basin Outlet	• Where inflow velocities are greater than 3 ft/s, rock outlet protection should be provided to prevent erosion and distribute the flows into the facility. • Vegetated portions of the contributing drainage area must be stabilized. • Sediment forebay shall be installed upstream of the facility.
Vegetated Filter Strip (Flow-Through)	• Level spreader must be installed where runoff enters the facility as shallow concentrated flow to distribute the runoff as sheet flow over the entire facility. • Vegetated portions of the contributing drainage area must be stabilized.
Water Reuse System	• Filter screens must be installed on all roof drains directed toward the facility. • For facilities that include inflow pipes, sump and/or trash rack shall be installed at manhole immediately upstream of facility.

Impervious Area Reduction for Redevelopment Sites

For **redevelopment sites**, where **volume control practices** are not feasible due to **site** limitations (contaminated soils, high **groundwater** table, close proximity to wells, etc.) a reduction in existing **impervious area** may be permitted. This strategy relies on several techniques to reduce the total area of rooftops, parking lots, streets, sidewalks and other types of impervious cover created at a **development site**. The basic approach is to reduce each type of impervious cover by downsizing the required minimum geometry specified in current local codes, keeping in mind that there are minimum requirements that must be met for fire, snowplow, and school bus operation. In many communities, local codes may need to be changed to allow the use of this group of better **site** design techniques.

The WMO (§503.3.C) states that for **redevelopments**, a proportionate reduction in existing **impervious area** is required for retention of any portion of the **volume control storage** that cannot be addressed using **volume control practices**. To utilize this provision, the applicant must:

1. Demonstrate that **site** limitations prevent the use of **volume control practices** to retain the **volume control storage** in full; and

2. Provide the **volume control storage** onsite to the maximum extent practicable.

For **developments** that satisfy the above requirements, for every 5% of **impervious area** that is reduced onsite, the **volume control storage** may be reduced by 25%. To satisfy the full **volume control storage** requirements, the **site's impervious area** would have to be reduced 20% from existing conditions.

Demonstration of Redevelopment Site Limitations

If the **redevelopment site** can retain a portion of the **volume control storage**, then **volume control practices** must be provided. For any portion of the **volume control storage** that cannot be retained on **site**, then soil data, **groundwater** data, and other **site** design limitations, such as zoning requirements, must be included in the **Watershed Management Permit** submittal. The information provided must demonstrate why a portion of the **volume control storage** cannot be retained and infiltrated or treated with a **flow-through practice**.

Volume Control Storage Credit for Volume Control Practices

As stated in the WMO (§503.3.C), the required **detention facility** volume can be reduced by the **volume control storage** provided onsite. To ensure that the **detention facility** and outlet control **structures** are appropriately sized for the 100-year, 24-hour **runoff** volume, the **volume control storage** credit is provided as a reduction in the overall curve number (CN) of the developed **site**. The following steps outline the CN reduction methodology:

For a given **watershed** (**site**) with area, A_W, **stormwater** storage is required using the **NRCS** procedure and is also required to provide a volume of infiltration storage equal to or greater than 1 inch over the **impervious area**, A_I, of the **development**. This procedure was developed to reflect the volumetric reduction in the **runoff** hydrograph for the **site** area (A_W).

The **NRCS runoff** equation is:

$$R_W = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

Where,

R_W = **runoff** depth (in) from Area, A_W
P = rainfall depth used to calculate **runoff** (in),
S = potential maximum retention after **runoff** begins (in), and is calculated by:

$$S = \frac{1000}{CN_W} - 10$$

Where,

CN_W = **runoff** curve number for the **watershed**

The volume of **runoff** (acre-feet), V_W, from **watershed** A_W can then be calculated by:

$$V_W = \frac{A_W}{12} \times R_W$$

The total volume of **runoff** from the **site** can be reduced by the volume control required and the extra **green infrastructure** volume that may be provided:

$$V_{ADJ} = V_W - V_R - V_{GI}$$

where,

V_{ADJ} = adjusted **runoff** volume from **site** (acre-feet)
V_R= volume of **volume control storage** (one-inch over **impervious area** of **development**)
V_{GI} = volume of **green infrastructure** provided in addition to the required one-inch

This reduced volume of **runoff** can be reflected in an overall reduction to the CN used in detention basin sizing by using:

$$\frac{V_{ADJ}}{A_W} = R_{ADJ} = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

Since R_{ADJ} is known, and P = 7.58 inches for the 100-year, 24-hour **storm event**, we can solve for S, which then translates to the adjusted CN. The adjusted curve number (CN_{ADJ}) is then used to calculate the required detention volume for the **site**.

Example 5.6 – Calculation of Volume Control Storage Credit (CN Reduction)

For a 10-acre proposed residential area with a developed CN of 78, and 3 acres of **impervious area**, find the revised CN resulting from the one-inch volume control provisions of the WMO.

The future 100-year **runoff** volume for the proposed **development** without volume control can be calculated using the **NRCS runoff** equation.

$$R_W = \frac{(7.58\text{ in}-0.2S)^2}{(7.58+0.8S)}$$

$$S = \frac{1000}{78} - 10 = 2.82\text{ in}$$

$$R_W = \frac{(7.58\text{ in} - (0.2)(2.82\text{ in}))^2}{7.58\text{ in} + 0.8(2.82\text{ in})}$$

$$Rw = 5.00\text{ inches}$$

The total volume is therefore:

$$V_W = \frac{R_W}{12} \text{ x } A_W = \frac{5}{12} \text{ x 10 acres} = 4.17\text{ acre-feet}$$

The volume associated with the total **impervious area** that must be stored is:

$$V_R = 3\text{ acres} \text{ x } \frac{1\text{ in}}{12} = 0.25\text{ acre feet}$$

For this example, V_{GI} =0, so the adjusted **runoff** volume is:

$$V_{ADJ} = 4.17\text{ acre-feet} – 0.25\text{ acre feet} = 3.92\text{ acre-feet}$$

And therefore:

$$12 \text{ x } \frac{V_{ADJ}}{A_W} = \frac{(P – 0.2S)^2}{(P + 0.8S)} = 4.70\text{ in}$$

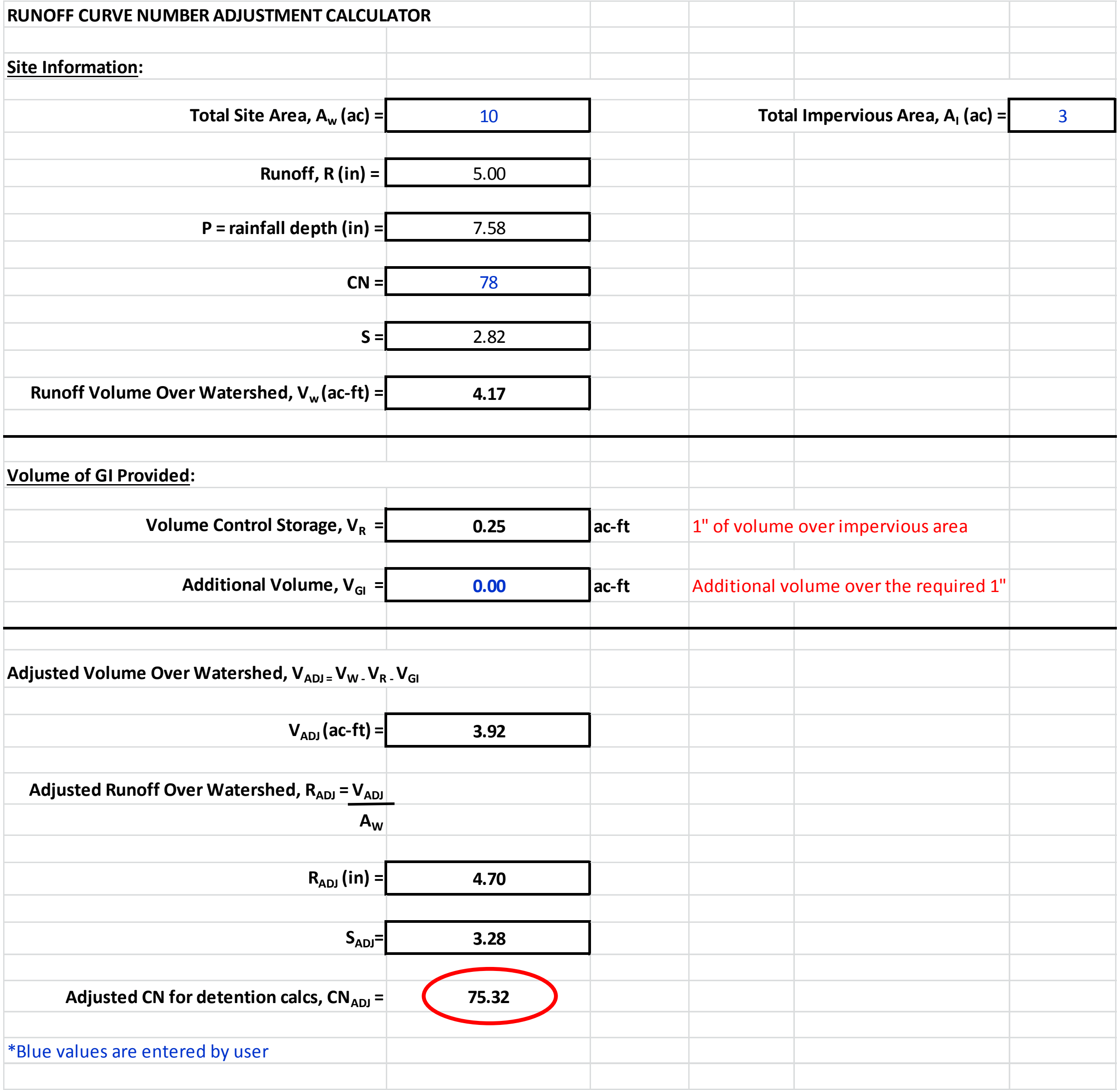
Since P=7.58 inches:

$$4.70 \text{ inches} = \frac{(7.58 \text{ in} - 0.2S)^2}{(7.58 \text{ in} + 0.8S)}$$

Solving this equation iteratively:

$S = 3.28$, and the adjusted CN, $CN_{ADJ} = 75.32$

The curve number in this example is reduced from 78 to 75.32. This procedure reflects the **stormwater** volume reduction and allows for hydrologic routing through proposed **stormwater** management facilities. To simplify this procedure, an Excel spreadsheet has been provided which reflects the analysis described above. The applicant would only have to provide areas, developed CN and the depth of storage being provided and the reduced CN would be solved for the user. The spreadsheet version of Example 5.6 is shown below.



RUNOFF CURVE NUMBER ADJUSTMENT CALCULATOR

Site Information:

Total Site Area, A_w (ac) = 10

Total Impervious Area, A_I (ac) = 3

Runoff, R (in) = 5.00

P = rainfall depth (in) = 7.58

CN = 78

S = 2.82

Runoff Volume Over Watershed, V_w (ac-ft) = 4.17

Volume of GI Provided:

Volume Control Storage, V_R = 0.25 ac-ft 1" of volume over impervious area

Additional Volume, V_{GI} = 0.00 ac-ft Additional volume over the required 1"

Adjusted Volume Over Watershed, $V_{ADJ} = V_W - V_R - V_{GI}$

V_{ADJ} (ac-ft) = 3.92

Adjusted Runoff Over Watershed, $R_{ADJ} = \frac{V_{ADJ}}{A_W}$

R_{ADJ} (in) = 4.70

S_{ADJ} = 3.28

Adjusted CN for detention calcs, CN_{ADJ} = 75.32

*Blue values are entered by user

Volume control facilities that make use of void volume above the invert of an underdrain (and which is reduced by 50%), may take full credit for this void volume toward required detention (at 100%), provided the void volume is tributary to the restrictor.

SITE DETENTION REQUIREMENTS (§504)

As presented in §504.1 of the WMO, **site detention facilities** are required for:

1. "**Residential subdivision development** on **parcels** totaling five acres or more" in size;

2. **Non-residential** or **multi-family residential development** on **parcels** totaling three acres or more in size "with new **development** on the **parcel**(s) that totals either individually or in the aggregate to more than 0.5 acres after the effective date" of the WMO; and

3. Roadway **development** that creates more than one acre of **new impervious area**, where practicable.

Non-residential developments (including those **developments** that have been made exempt from the WMO) will be required to provide **site** detention volume when new **development** to the **site** (any activity or disturbance that affects **runoff**, including the grading of pervious to pervious, or the replacement of impervious on impervious) reaches 0.5 acres in aggregate since the effective date of the WMO. The provision allows existing **non-residential developments** to expand or construct additions before having to provide detention storage.

Allowable Release Rate

The **allowable release rate** for a **development site** that discharges to a **waterway** is (§504.3):

- 0.30 cfs per acre of **development** for the 100-year event.

The calculation of the **allowable release rate** is based on the area of **development** (§504.2). The **allowable release rate** is the maximum allowable **stormwater** discharge release rate for the entire **development site**. Though the **site detention facility** release rate may be impacted by the presence of existing **depressional storage** or **unrestricted flow** within the **development**, the **site** maximum **allowable release rate** is 0.30 cfs per acre. §504.4 states that the release rate from the **site detention facility** plus the flowrate from any **unrestricted flow** areas must not exceed the **allowable release rate** for the **development**.

***Allowable release rate** for the 100-year event:*

$$Q_{Allowable\_(100\text{-}year)}\ (\text{cfs}) = 0.30\ \text{cfs/acre} \times A_{Development}\ (\text{acres})$$

Where: $Q_{Allowable\_(100\text{-}year)}$ = the **allowable release rate** for the 100-year event in cubic feet per second (cfs)

$A_{Development}$ = the area of the **site development** (acres)

Existing Depressional Storage

§504.5 requires that the **allowable release rate** be reduced to reflect the retention provided in existing **depressional storage** areas. For **sites** where **depressional storage** exists and where the existing **runoff** rate for the **development** is less than the **allowable release rate** provided in §504.3, the **allowable release rate** must be reduced to the existing 100-year **runoff** rate for the **development**. The required detention volume for the **site** must be calculated using the existing **runoff** rate for the **development site**. The existing **runoff** rate must in turn be calculated using event hydrograph methods that account for the existing onsite **depressional storage** volume.

Unrestricted Flows

Unrestricted flow is **stormwater runoff** from a **development** which is not directed to the **detention facility**. This is commonly due to the location of the **site stormwater facility** or sometimes due to topography. The areas generating **unrestricted flow** are referred to as unrestricted or uncontrolled release rate areas.

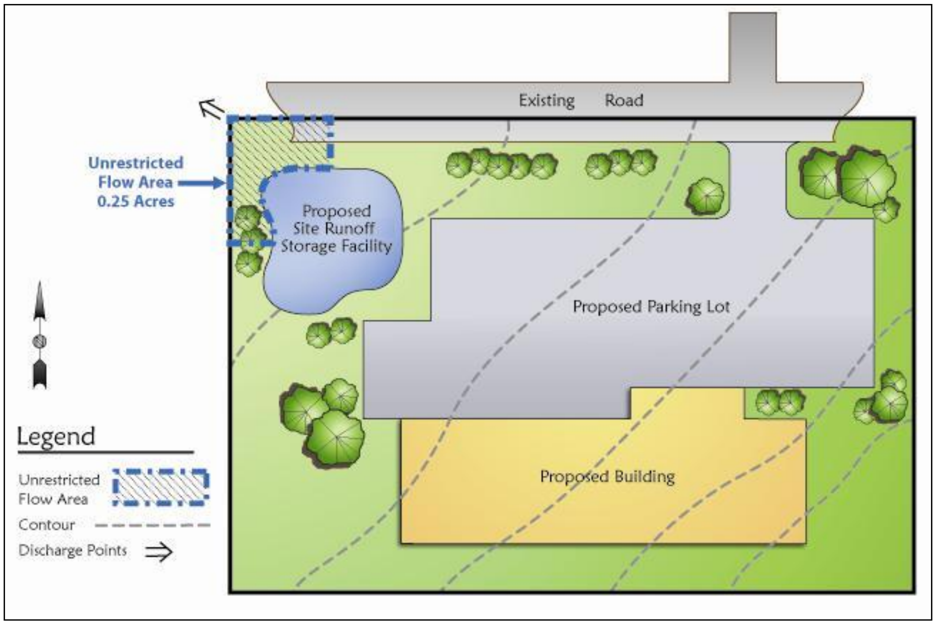


Figure 5.22. Unrestricted Flow Area on the Downstream Side of a Proposed Site Detention Facility

Every attempt should be made to direct all **runoff** from the **development site** to the **site detention facility**. Per §504.6, when all **runoff** from a **development** cannot be captured and conveyed to the **site detention facility**, the **unrestricted flow** must be addressed by demonstrating that it does not cause offsite damage. In addition, one of the following methods must be used:

Option 1 – Equivalent Area

Diverting an equivalent upstream **tributary area** where detention is not provided to the **site detention facility** is one option of addressing the **unrestricted flow**. If the **site** has **upstream tributary flows**, this flow can be diverted into the storage basin. Essentially, if **upstream**

tributary flow were to be bypassed through the **site**, a portion of this flow could be directed to the **site detention facility**. The **site allowable release rate** would remain the same as calculated in §504.3. CN values and time of concentration calculations would include the equivalent upstream **tributary area**. It should be noted that the diversion of equivalent area is only allowed if the property is owned by the applicant. Also, this is not a viable option if there is no offsite area tributary to the project **site**.

Option 2 – Reduction in Release Rate

Reducing the **allowable release rate** for the area that is tributary to the **site detention facility** to compensate for **unrestricted flow** leaving the **site** is another option of addressing this **unrestricted flow**. The **site allowable release rate** would remain as calculated in §504.3, but the **site detention facility** release rate is reduced.

An event hydrograph model run is needed to determine the release rate of the unrestricted area(s). Both the CN and the t_c must be calculated. The CN and the t_c must be specific to the unrestricted area.

Option 3 – Native Planting Conservation Area

Planting the **unrestricted flow** area with native deep-rooted vegetation approved by the **District** is the third option of addressing the **unrestricted flow**. **Unrestricted flow** areas must be placed in an easement and maintained as a **native planting conservation area** in perpetuity. The **allowable release rate** for the **development** must be based on the **development** area tributary to the **site detention facility**.

The **native planting conservation area** within a **development** must be planted and maintained with deep rooted native plants. The proposed plant mix and a landscape plan must be submitted with the **Watershed Management Permit** application. With Option 3, the **site allowable release rate** is re-calculated based on the area of the **development** minus the **unrestricted flow** area. The **site detention facility** release rate would become the **site allowable release rate**.

Site Detention Volume Determination

The **site** detention volume must be sufficient such that the 100-year **allowable release rate** specified in §504.3, or the reduced **allowable release rate** as required in §504.5 (**depressional storage**) and/or §504.6 (**unrestricted flow**), will not be exceeded.

The **site detention facility** volume must be calculated using either an event hydrograph routing method or the nomograph relating percent impervious to required detention volume.

Event Hydrograph Method

The event hydrograph method must be used when:

1. The **allowable release rate** calculated in §504.3 is adjusted due to **site depressional storage**;

2. The **allowable release rate** calculated in §504.3 is adjusted through Option 2 (§504.6.B(2)) or Option 3 (§504.6.B(3)) due to **unrestricted flow**;
3. There is **upstream tributary flow** to the **site** (§504.10); and/or
4. There are tailwater conditions on the **site detention facility** outlet **structure**.

The **site detention facility** must be sized such that the 100-year, 24-hour **storm event** release rate will not exceed the **allowable release rate** and the design high water level will be contained within the **site detention facility**. The event hydrograph methodology contained in the *Site Runoff Requirements* section of the **TGM** should be followed.

Detention Volume Nomograph

For simple cases, a detention volume nomograph can be used to determine the required detention volume for the **development**. The nomograph relates detention volume with the reduced CN of a **site** (the overall CN adjusted for the **volume control** storage provided), and is used instead of an event hydrograph method. The detention nomograph is provided as Figure 5.23.

The use of the nomograph is only allowable for the following scenarios:

- The **allowable release rate** is not affected by existing onsite **depressional storage**.
- The **allowable release rate** is not affected by any **unrestricted flow**.
- There is no **upstream tributary flow** to the proposed **detention facility**.
- There are no tailwater conditions on the proposed **detention facility's** outlet control **structure**.

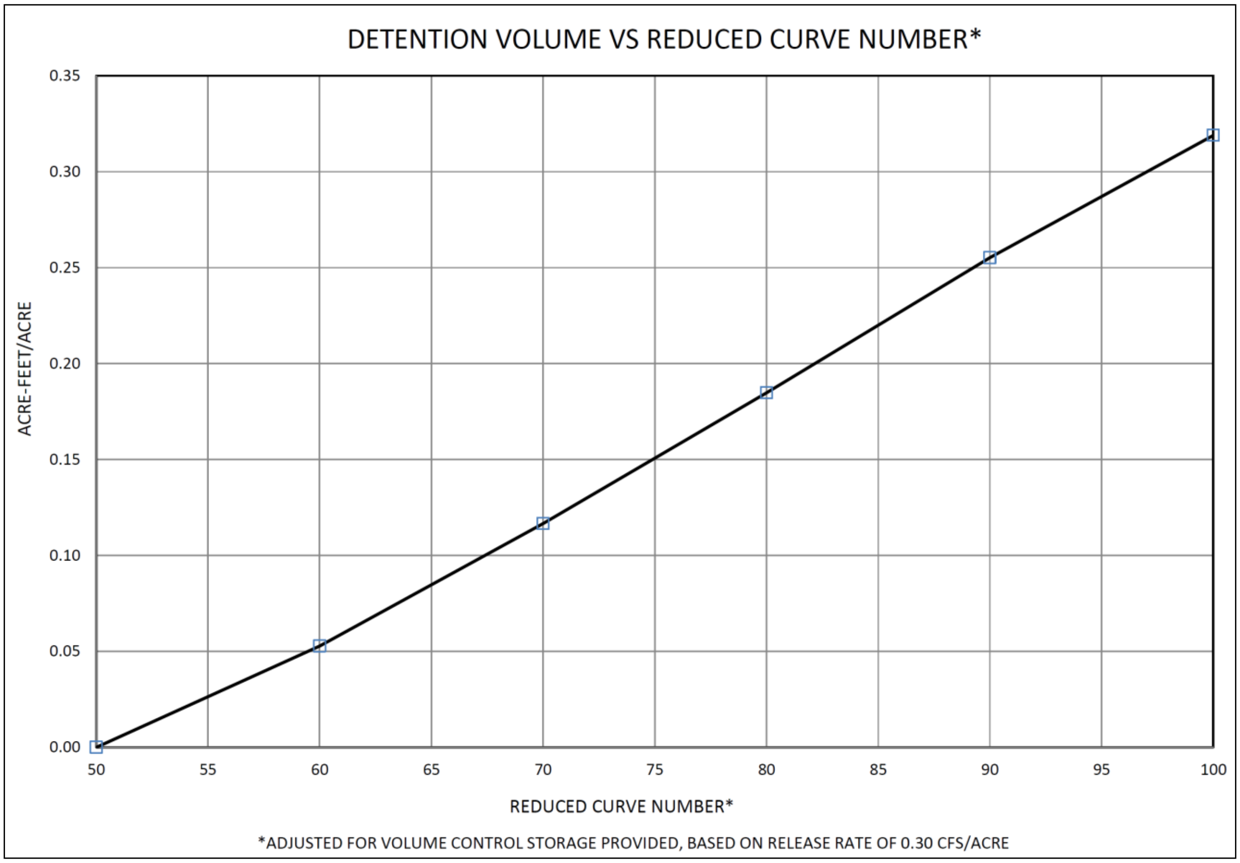


Figure 5.23. Detention Nomograph

Upstream Tributary Flow

Upstream tributary flow includes **stormwater runoff** or **groundwater** flows from **tributary areas** upstream of a **development site**. The WMO (§504.10) requires that the proposed **site detention facility** be designed to safely convey any upstream flow through the project **site**. Flows from the upstream **tributary areas** to the **site** should be computed under fully developed conditions to ensure the proposed **stormwater facilities** are not undersized. Upstream offsite flows cannot be blocked.

When an **upstream tributary flow** to a **development site** exists, it must be addressed in conjunction with the design of the **site detention facility** by one of the following options:

Option 1 – Bypass Flow

Provide **site detention facility** volume for the **development** at the **allowable release rate** while bypassing **upstream tributary flows** either (1) around the **site** to the existing discharge location or (2) through the proposed **detention facility**. The preferred method is to bypass offsite flows around the **site**. The bypass flow can be routed around the **development** via **storm sewer** or swales and must be designed to convey the 100-year critical duration **storm event**. If it is not feasible to route offsite flows around the **site**, the bypass flow may be routed through the proposed **detention facility**, with the overflow weir/restrictor sized to convey the onsite and offsite 100-year critical duration peak flowrate. In either case, the bypass flow must not result in an increase in velocities or flows when leaving the proposed **development site** downstream or on adjacent properties. The applicant is responsible for demonstrating that any downstream or adjacent drainage conditions are not exacerbated when compared with pre-development conditions.

The method for bypassing upstream flows through the **site** should be based on the following hierarchy:

1. Where possible, the preferred method is to bypass flows around the **site**. Offsite flows may be routed through the **detention facility** if the **drainage areas** meet the requirements of (2) below. If the ratio of offsite to onsite **drainage area** is greater than 5:1, the upstream flow must be bypassed around the **detention facility**.

2. If the ratio of offsite to onsite **drainage area** is 5:1 or less, the 100-year critical duration peak flowrate from upstream areas should be bypassed through the overflow weir of the **detention facility**.

Whenever offsite flows are routed through the **detention facility**, an analysis of the basin drawdown time must be performed. In cases where the drawdown time is excessive (greater than 72 hours), it is recommended that the upstream **tributary area** is bypassed around the **detention facility**.

Option 2 – Routing Through Site Detention facility

Provide **site detention facility** volume to accommodate both the **runoff** for the **development** and the **upstream tributary flow** area on the **site** at the **site's allowable release rate**. If it is infeasible to route the **upstream tributary flows** around the **site**, this alternative would be to route the offsite flows into the **site detention facility** while providing additional storage volume for the offsite flow. The **site detention facility** volume will be determined based on the **allowable release rate** (calculated from the **site** area excluding the upstream **tributary area**) and the **stormwater runoff** from the **site** area and the upstream **tributary area** for the 100-year, 24-hour storm.

Option 3 – Store the Upstream Tributary Flow

Sufficient **site detention facility** volume can be provided to accommodate **runoff** from the **development** and **the upstream tributary flow** area with a release rate that ensures that no adverse offsite impacts will occur and that a **water resource benefit** is provided. The applicant must consider **runoff** from all **tributary areas** and demonstrate the impacts for 2-year, 10-year, and 100-year **storm events**, at a minimum, using **critical duration analysis** and the methodology provided in §504.9. The minimum **site detention facility** volume required must be based on the **site allowable release rate** as determined in §504.3 and §504.4. The **District** or **authorized municipality** will determine whether the proposed release rate is sufficiently providing a **water resource benefit**.

Design Considerations for the Site Detention Facility

Site detention facilities may be dry-bottom or wet-bottom basin designs. The treatment of the **volume control storage** may be addressed within the basin design. The design of the outlet **structure** and emergency overflow, and the consideration of tailwater conditions are of particular concern, and are discussed below. Numerous other design considerations must also be made. §504.11 requires that the **site detention facility** must:

1. Be designed to provide one foot of freeboard (an elevation higher than the **base flood** water level) for the 100-yr, 24-hr storm;
2. Be designed to function with a gravity outlet wherever possible;
3. Function without human intervention and under tailwater conditions with minimal **maintenance**;
4. Provide an overflow **structure** and overflow path that can safely pass a **design runoff rate** of at least 1.0 cfs/acre of **tributary area** to the **site detention facility**;
5. Provide side slope **stabilization**;
6. Provide earth **stabilization** and armoring with riprap, concrete or other durable material when high erosive forces could lead to soil **erosion** or washout. Examples of where armoring may be required include:
    a. **Storm sewer** flared end sections;
    b. Emergency Overflows; and
    c. Berms adjacent to streams.
7. Be accessible and maintainable; and
8. Provide a **maintenance** agreement.

Other design considerations include:

1. Above-ground **site detention facilities** should provide access for **maintenance** like riding lawn mowers or a small truck;
2. **Site detention facilities** with retaining walls, a ramp will need to be provided for the above mentioned equipment access;
3. Above-ground **site detention facilities** should be designed with side slopes of 4-feet horizontal to 1-foot vertical (4:1) for ease of maintenance. The design minimum for side slopes is 3-feet horizontal to 1-foot vertical (3:1);

4. Underground facilities should also provide access for **maintenance** through manholes large enough for the necessary **maintenance** to be performed, including access ladders. Access is recommended near all storm sewer outlets and inlets to the underground detention system to allow for ease of inspection, maintenance, and debris removal. A minimum of two access points is recommended at opposite ends of the vault for safety, with a minimum of at least one ladder provided egress (e.g. located along a wall with integrated steps).

5. Appurtenance rim elevations for **stormwater facilities** upstream and downstream of the **site detention facility** must be higher than the **base flood** level of the **site detention facility**; and

6. When feasible the **site detention facilities** should be constructed and functioning prior to issuance of **building** permits or the operation of **sanitary sewers** for service. Volume control facilities that make use of void volume above the invert of an underdrain (and which have been reduced by 50%), may take full credit for this void volume toward required detention (at 100%), provided the void volume is tributary to the restrictor.

7. Volume control facilities that make use of void volume above the invert of an underdrain (and which have been reduced by 50%), may take full credit for this void volume toward required detention (at 100%), provided the void volume is tributary to the restrictor.

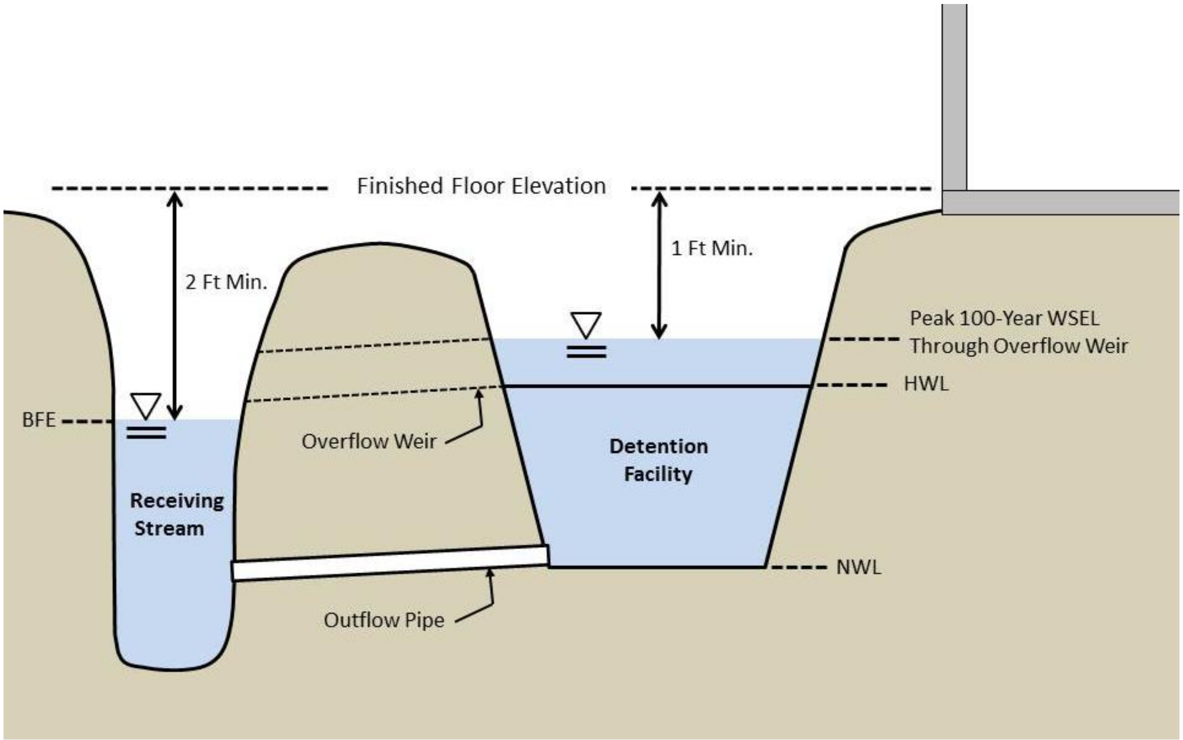


Figure 5.24. Critical Elevations for Detention Facilities

As shown in Figure 5.24, the finished floor elevation (or low-entry elevation) for proposed **structures** must be at least one foot above the peak 100-year water surface elevation (WSEL) through the overflow weir. The overflow weir will typically be set at the HWL of the detention basin and sized to convey the peak 100-year onsite and offsite flowrate. The depth of flow in the overflow weir is the peak 100-year WSEL for the **detention facility**, and one foot of freeboard must be provided for adjacent **structures** above this elevation.

Tailwater/Zero Release Rate Conditions

Tailwater conditions occur when a **stormwater facility** discharges to a **floodplain**, depressional area, a poor draining swale, **storm sewers**, or a condition where the HGL for the downstream condition is higher than the outlet of the **site stormwater** management facility.

When tailwater conditions are present, the **allowable release rate** for the **site detention facility** is 0 cfs (zero release rate) until the water level in the facility (the head) exceeds the tailwater elevation. This prevents negative impacts to the downstream condition, as there will be no discharge until the facility outlet can drain. Tailwater conditions may require an increase in the size of the **site detention facility**. The outlet control **structure** should be sized assuming no tailwater conditions, but the volume in the **detention facility** should be determined by the appropriate tailwater conditions. The **site detention facility** may not exceed the **allowable release rate** when the **detention facility** is modeled without tailwater conditions.

Determining the tailwater condition depends upon the type of downstream facility. If the storage facility is discharging into a river or **floodplain**, the **100-year flood elevation** is used as the tailwater condition. If the storage facility is discharging into a depressional area, drainage swale, or **storm sewer** system the applicant should use the highest elevation (**base flood**) as the tailwater elevation. The local governing body may determine that the tailwater elevation may need to be raised or lowered based on local conditions.

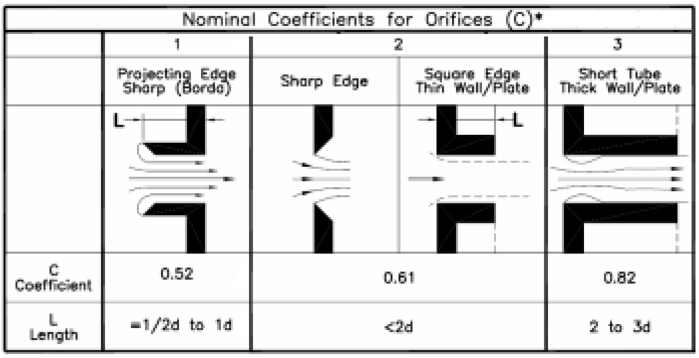
Site Detention facility Outlet Structure

The outlet **structure** of the **site detention facility** (restrictor) should be sized using the orifice equation such that the **allowable release rate** is met at the design high water level (HWL) of the **detention facility**. The orifice equation consists of the following:

$$Q = C \times A \times (2gh)^{0.5}$$

Where,
Q = discharge rate, in cfs
C = orifice discharge coefficient (taken from Figure 5.23)
A = orifice area (ft^2)
g = acceleration due to gravity, 32.2 ft/s^2
h = head (ft); vertical distance between water surface and center of orifice

The appropriate orifice discharge coefficient, C, should be used based on the restrictor type shown in Figure 5.25. In addition, the head on the orifice should account for any tailwater effects from receiving streams, if applicable.



Nominal Coefficients for Orifices (C)*

	1	2		3
	Projecting Edge Sharp (Borda)	Sharp Edge	Square Edge Thin Wall/Plate	Short Tube Thick Wall/Plate
C Coefficient	0.52	0.61		0.82
L Length	=1/2d to 1d	<2d		2 to 3d

Figure 5.25. Orifice Discharge Coefficients (Source: MWRD, 1978)

The outlet control device for **site detention facilities** must meet the following design requirements:

1. The outflow control restrictor (including the emergency overflow device) must be located within the property boundary;

2. The location of the outflow control restrictor must be clearly shown on a utility **site** plan drawing and a section detail must be provided; installation must be permanent and durable; there must be no plastic or removable or adjustable gates or systems, or bypass shunts/secondary outlets;

3. The outflow control restrictor must be visible and readily accessible for **maintenance** (cleaning/rodding) and inspection;

4. Outflow control restrictors must be located on the downstream side of the **structure** for ease of debris clearing;

5. The outflow control restrictor must be designed to be self-cleaning;

6. Restrictor plates must be of minimum ¼ inch thickness, and shaped to conform to the concrete **structure** wall, with a minimum of four steel anchor bolts screwed with epoxy at least three inches into the wall, with all bolt heads tack welded to the plate;

7. Restrictor pipes must be inserted in larger diameter pipes; the restrictor pipe must be a minimum 2-foot long with the annular space over the length filled with non-shrink grout suitable for submergence;

8. The outflow control restrictor must have a minimum 4-foot diameter manhole or **structure** of equivalent clearance; and

9. Though there is no minimum restrictor size (diameter), **maintenance** increases with smaller diameters. It is recommended that whenever possible, head be reduced; or if there are multiple onsite restrictors, the **site detention facilities** should be combined to

facilitate increasing the restrictor size (where appropriate). If a project entails any restrictor with less than a 4-inch diameter, a letter from the applicant is required acknowledging a more stringent outlet **maintenance** plan, and a letter from the **permittee** is required acknowledging enforcement of the **maintenance** plan. Restrictors less than 4 inches in diameter should be installed with hoods and in a wall with a pre-sump. Another acceptable alternative for restrictors less than four inches in diameter is the use of the City of Chicago's vortex restrictor.

The preferred outlet control structure consists of a center-wall style restrictor configured with a removable hood that prevents debris from clogging the restrictor. Example details for these devices are provided in Appendix C.

Emergency Overflow Structure

The WMO (§504.11.C) requires an overflow **structure** and overflow path for the **site runoff** facility that can safely pass a **design runoff rate** of at least 1.0 cfs per acre of **tributary area**. The **design runoff rate** must be determined in accordance with the methodology for **major stormwater system**s. A properly sized overflow **structure** and overflow path will help protect the **detention facility** from damage should the water level in the facility exceed the one foot of freeboard and direct the overflow downstream to an appropriate location. The overflow **structure** may be set at the high water elevation of the facility but all other elevations surrounding the facility will need to be constructed at least one foot above the high water level in the emergency overflow **structure**. The capacity of the emergency overflow **structure** and overflow path for the **site detention facility** shall be designed by calculating the 100-year critical duration peak onsite and offsite flowrate to the **detention facility**. The design flowrate should be based on the 100-year critical duration flowrate, or 1.0 cfs/acre of **tributary area**, whichever is higher.

$$Q_{Overflow}\text{ (cfs)} = 1.0\text{ cfs/acre} \times A_{Tributary}\text{ (acres)}$$

Where: $Q_{Overflow}$ is the minimum overflow **structure** capacity
$A_{Tributary}$ is all of the area tributary to the **site detention facility**, including the **development site** and **upstream tributary flow** directed to the **site detention facility**

The overflow weir for the **site detention facility** should be sized using the weir equation:

$$Q = C \times L \times H^{3/2}$$

Where,
Q = flowrate (cfs)
C = weir coefficient (assume = 3.0)
L = length of weir (ft)
H = head on weir (ft)

The following examples provide sample calculations for the sizing and design of a **detention facility** for several scenarios that range from simple to complex.

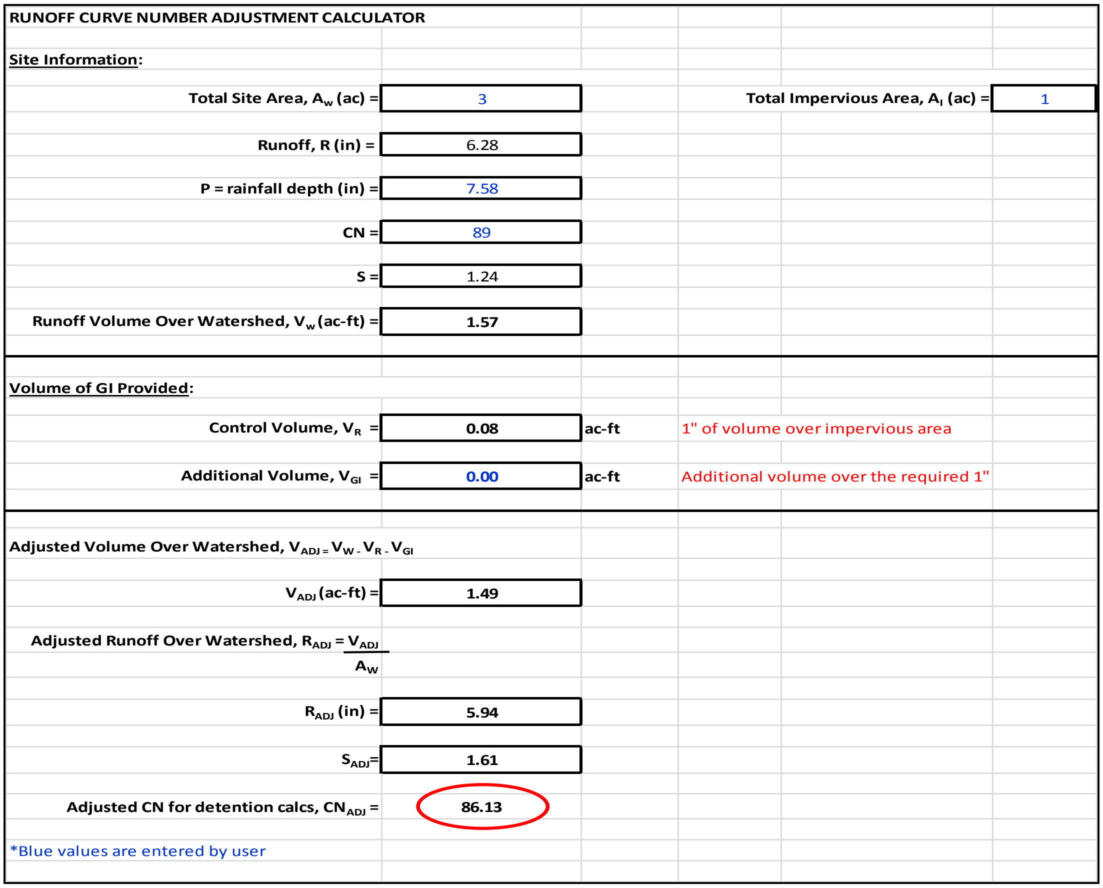
Example 5.7 – Simple Detention (Nomograph Method)

Using the **site** from *Example 5.2*, determine the required detention volume using the detention nomograph. It is assumed that the proposed **site** will provide the required volume control storage in the aggregate voids under the permeable pavement parking lot. It is also assumed there is no offsite **tributary area** or **unrestricted flow** area for the **site**.

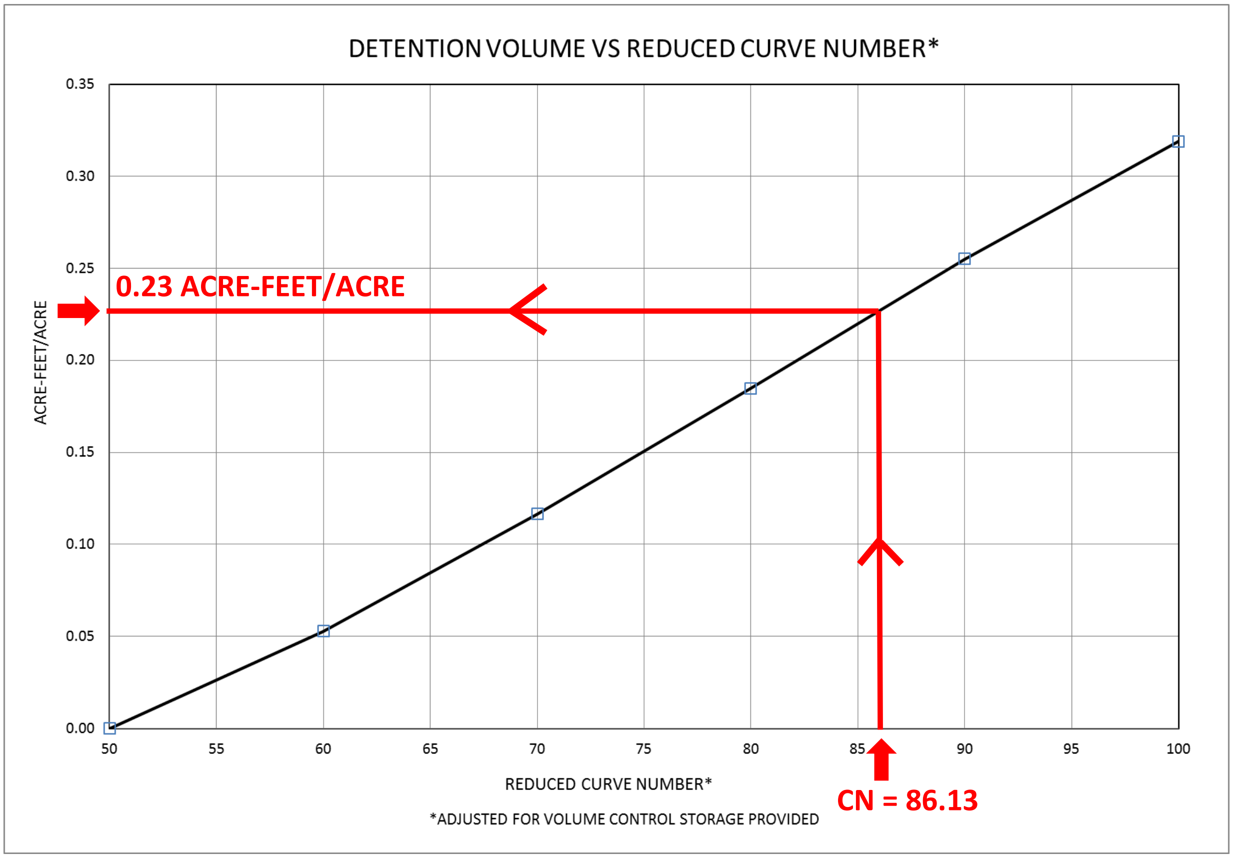
As shown in Example 5.2, the curve number for the **site** is 89, with a total **impervious area** (open water and **building**) of one acre. The required volume control storage, V_c, for the **site** is calculated as:

$$V_c = \quad 1 \text{ in x} \quad \frac{1 \text{ foot}}{12 \text{ inches}} \quad \text{x } 1 \text{ acre} = \quad 0.083 \text{ acre-feet}$$

Using the CN Adjustment Calculator for the **site** (shown below), the adjusted CN is calculated to be 86.13.

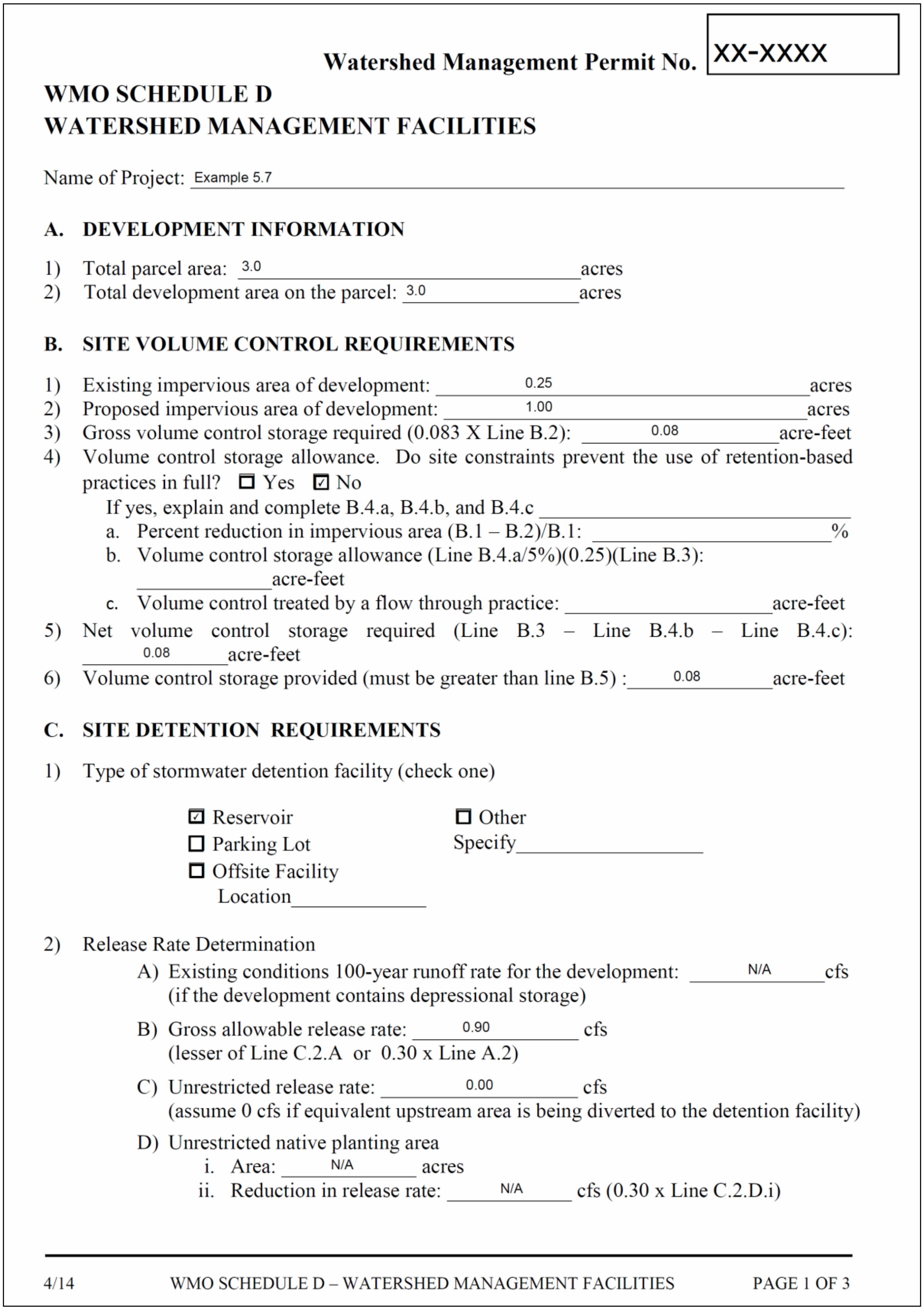


RUNOFF CURVE NUMBER ADJUSTMENT CALCULATOR				
Site Information:				
Total Site Area, A_w (ac) =	3		**Total Impervious Area, A_I (ac) =**	1
Runoff, R (in) =	6.28			
P = rainfall depth (in) =	7.58			
CN =	89			
S =	1.24			
Runoff Volume Over Watershed, V_w (ac-ft) =	**1.57**			
Volume of GI Provided:				
Control Volume, V_R =	**0.08**	**ac-ft**	1" of volume over impervious area	
Additional Volume, V_{GI} =	**0.00**	**ac-ft**	Additional volume over the required 1"	
Adjusted Volume Over Watershed, $V_{ADJ} = V_W - V_R - V_{GI}$				
V_{ADJ} (ac-ft) =	**1.49**			
Adjusted Runoff Over Watershed, $R_{ADJ} = \frac{V_{ADJ}}{A_W}$				
R_{ADJ} (in) =	**5.94**			
S_{ADJ} =	**1.61**			
Adjusted CN for detention calcs, CN_{ADJ} =	**86.13**			
*Blue values are entered by user				



Using the detention nomograph, 0.23 acre-feet of detention volume is required for every acre of **development**, based on the adjusted CN of 86.13. By multiplying this value times the **development** area of 3 acres, the required detention volume is calculated to be 0.69 acre-feet.

It should be noted that although the detention nomograph is allowed to determine the required detention volume, an event hydrograph method must be used to size the overland flow routes in and out of the **detention facility**.



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D
WATERSHED MANAGEMENT FACILITIES

Name of Project: Example 5.7

A. DEVELOPMENT INFORMATION

1) Total parcel area: 3.0 acres
2) Total development area on the parcel: 3.0 acres

B. SITE VOLUME CONTROL REQUIREMENTS

1) Existing impervious area of development: 0.25 acres
2) Proposed impervious area of development: 1.00 acres
3) Gross volume control storage required (0.083 X Line B.2): 0.08 acre-feet
4) Volume control storage allowance. Do site constraints prevent the use of retention-based practices in full? ☐ Yes ☑ No
   If yes, explain and complete B.4.a, B.4.b, and B.4.c ____________
   a. Percent reduction in impervious area (B.1 – B.2)/B.1: ____________%
   b. Volume control storage allowance (Line B.4.a/5%)(0.25)(Line B.3): ____________acre-feet
   c. Volume control treated by a flow through practice: ____________acre-feet
5) Net volume control storage required (Line B.3 – Line B.4.b – Line B.4.c): 0.08 acre-feet
6) Volume control storage provided (must be greater than line B.5) : 0.08 acre-feet

C. SITE DETENTION REQUIREMENTS

1) Type of stormwater detention facility (check one)

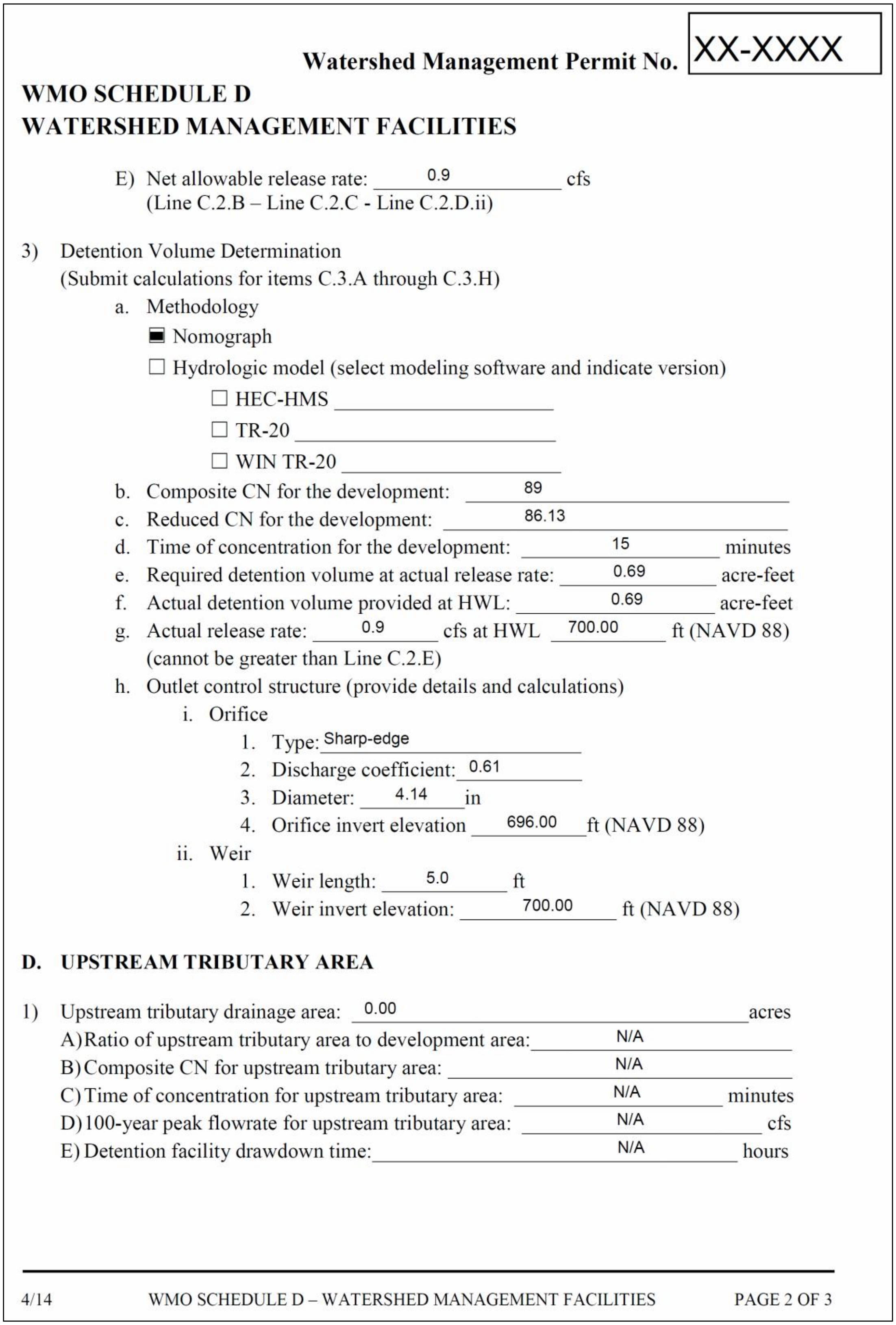
☑ Reservoir
☐ Parking Lot
☐ Offsite Facility
Location____________

☐ Other
Specify____________

2) Release Rate Determination
   A) Existing conditions 100-year runoff rate for the development: N/A cfs
   (if the development contains depressional storage)
   B) Gross allowable release rate: 0.90 cfs
   (lesser of Line C.2.A or 0.30 x Line A.2)
   C) Unrestricted release rate: 0.00 cfs
   (assume 0 cfs if equivalent upstream area is being diverted to the detention facility)
   D) Unrestricted native planting area
      i. Area: N/A acres
      ii. Reduction in release rate: N/A cfs (0.30 x Line C.2.D.i)

4/14 WMO SCHEDULE D – WATERSHED MANAGEMENT FACILITIES PAGE 1 OF 3

Schedule D Form for Example 5.7 (Page 1 of 3)



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D
WATERSHED MANAGEMENT FACILITIES

E) Net allowable release rate: 0.9 cfs
(Line C.2.B – Line C.2.C - Line C.2.D.ii)

3) Detention Volume Determination
(Submit calculations for items C.3.A through C.3.H)

a. Methodology

■ Nomograph

☐ Hydrologic model (select modeling software and indicate version)

- ☐ HEC-HMS ______
- ☐ TR-20 ______
- ☐ WIN TR-20 ______

b. Composite CN for the development: 89

c. Reduced CN for the development: 86.13

d. Time of concentration for the development: 15 minutes

e. Required detention volume at actual release rate: 0.69 acre-feet

f. Actual detention volume provided at HWL: 0.69 acre-feet

g. Actual release rate: 0.9 cfs at HWL 700.00 ft (NAVD 88)
(cannot be greater than Line C.2.E)

h. Outlet control structure (provide details and calculations)

i. Orifice

1. Type: Sharp-edge
2. Discharge coefficient: 0.61
3. Diameter: 4.14 in
4. Orifice invert elevation 696.00 ft (NAVD 88)

ii. Weir

1. Weir length: 5.0 ft
2. Weir invert elevation: 700.00 ft (NAVD 88)

D. UPSTREAM TRIBUTARY AREA

1) Upstream tributary drainage area: 0.00 acres

A) Ratio of upstream tributary area to development area: N/A

B) Composite CN for upstream tributary area: N/A

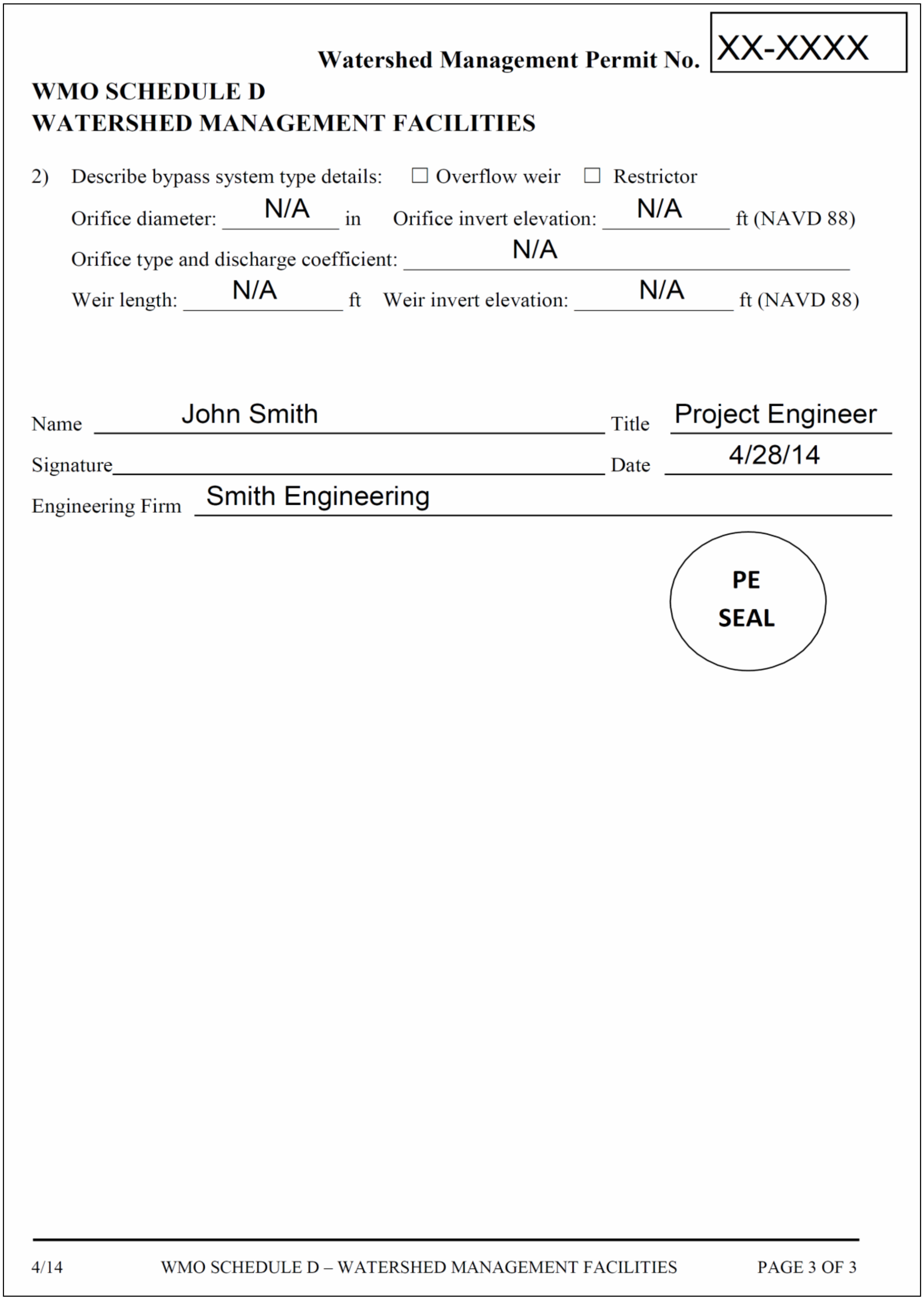
C) Time of concentration for upstream tributary area: N/A minutes

D) 100-year peak flowrate for upstream tributary area: N/A cfs

E) Detention facility drawdown time: N/A hours

4/14 WMO SCHEDULE D – WATERSHED MANAGEMENT FACILITIES PAGE 2 OF 3

Schedule D Form for Example 5.7 (Page 2 of 3)



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D
WATERSHED MANAGEMENT FACILITIES

2) Describe bypass system type details: ☐ Overflow weir ☐ Restrictor

Orifice diameter: N/A in Orifice invert elevation: N/A ft (NAVD 88)

Orifice type and discharge coefficient: N/A

Weir length: N/A ft Weir invert elevation: N/A ft (NAVD 88)

Name John Smith Title Project Engineer

Signature Date 4/28/14

Engineering Firm Smith Engineering

PE
SEAL

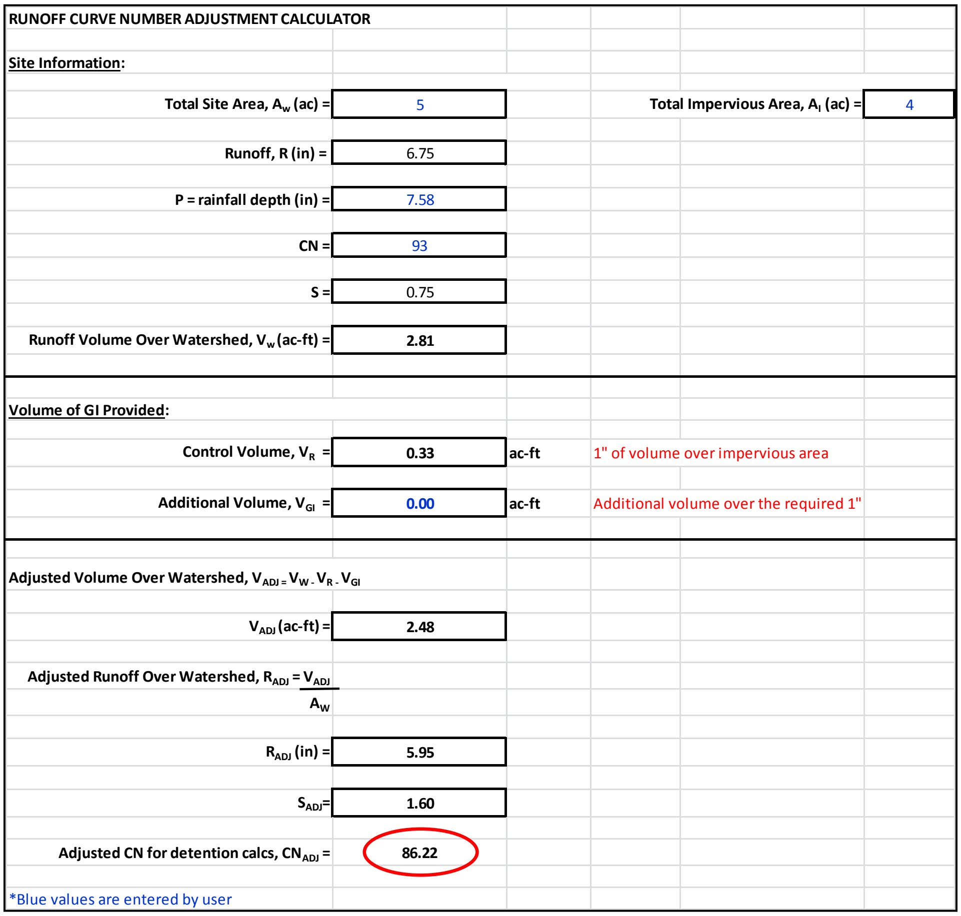
4/14 WMO SCHEDULE D – WATERSHED MANAGEMENT FACILITIES PAGE 3 OF 3

Schedule D Form for Example 5.7 (Page 3 of 3)

Example 5.8 –Detention with Unrestricted Releases and Upstream Tributary Area

A proposed 5-acre commercial **development** has a curve number (CN) of 93 (80% impervious) and a time of concentration (t_c) of 15 minutes. Determine the **stormwater** detention volume required for the **site**. Based on **Cook County** one-foot topography, it was determined there are 3 acres of offsite **tributary area** to the project **site**. The offsite **tributary area** has a CN of 89 and a t_c of 12 minutes. The proposed **development** will provide the 1 inch of **volume control storage** in the void space of aggregate under a permeable parking lot. There is a 0.2-acre area with a CN of 74 and a t_c of 10 minutes that will release undetained from the **site**.

Step 1: Using the ***Runoff** Curve Number Adjustment Calculator* spreadsheet, determine the reduced curve number resulting from the 1 inch of **volume control storage** provided for the **site**.



RUNOFF CURVE NUMBER ADJUSTMENT CALCULATOR

Site Information:

Total Site Area, A_w (ac) = 5
Total Impervious Area, A_I (ac) = 4
Runoff, R (in) = 6.75
P = rainfall depth (in) = 7.58
CN = 93
S = 0.75
Runoff Volume Over Watershed, V_w (ac-ft) = **2.81**

Volume of GI Provided:

Control Volume, V_R = **0.33** ac-ft 1" of volume over impervious area
Additional Volume, V_{GI} = **0.00** ac-ft Additional volume over the required 1"

Adjusted Volume Over Watershed, $V_{ADJ} = V_W - V_R - V_{GI}$

V_{ADJ} (ac-ft) = **2.48**

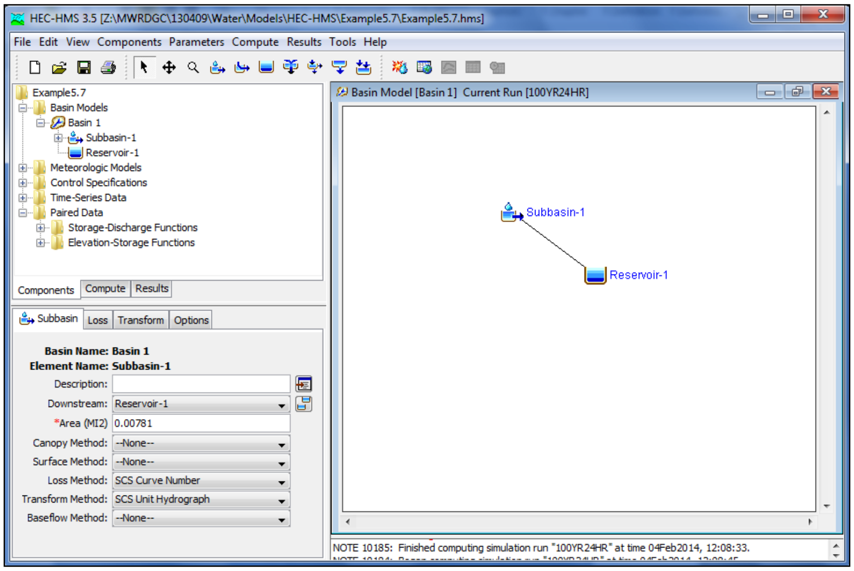
Adjusted Runoff Over Watershed, $R_{ADJ} = \frac{V_{ADJ}}{A_W}$

R_{ADJ} (in) = **5.95**
S_{ADJ} = **1.60**
Adjusted CN for detention calcs, CN_{ADJ} = **86.22**

*Blue values are entered by user

As shown in the above figure, a CN of 86.22 should be used in the **stormwater** detention calculations.

Step 2: Using the adjusted CN for the proposed **development** determined in Step 1, set up a HEC-HMS hydrologic model to determine the required **stormwater** detention volume.



HEC-HMS 3.5 [Z:\MWRDGC\130409\Water\Models\HEC-HMS\Example5.7\Example5.7.hms]

Basin Model [Basin 1] Current Run [100YR24HR]

Basin Name: Basin 1
Element Name: Subbasin-1
Description:
Downstream: Reservoir-1
*Area (MI2) 0.00781
Canopy Method: --None--
Surface Method: --None--
Loss Method: SCS Curve Number
Transform Method: SCS Unit Hydrograph
Baseflow Method: --None--

```
NOTE 10185: Finished computing simulation run "100YR24HR" at time 04Feb2014, 12:08:33.
```

In this case, there is one subbasin that represents the project **site** (Subbasin-1) and one storage area that represents the proposed detention pond (Reservoir-1). The subbasin and pond are the components of the *Basin Model*.

The *Meteorological Model* contains the rainfall depth information, which is the 100-year, 24-hour from Table 5-3 (7.58 inches). The *Time-Series Data* contains the time distribution of rainfall, which is the Huff 3rd quartile distribution for the 24-hour storm duration.

For *Subbasin-1*, enter the information for the project **site**:

- Area = 0.00781 square miles (5 acres)
- CN = 86.22 (from CN adjustment calculator spreadsheet) (do not enter % impervious)
- Lag time = 9 minutes (0.6 * t_c)
- SCS CN and Unit Hydrograph Methodology

The stage-storage-discharge relationship for the detention basin is specified under Paired Data. For *Reservoir-1*, a spreadsheet is used to determine the stage-storage-discharge relationship for the proposed **detention facility**.

Step 3: Determine the **allowable release rate** for the **site**, accounting for any unrestricted areas. The **allowable release rate** from the **site** is initially 1.5 cfs (0.3 cfs/acre x 5 acres) but should be adjusted to account for the 0.2-acre undetained area (which was delineated based on the proposed grading plan). The 100-year, 24-hour unrestricted release rate from this area must be calculated using the HEC-HMS hydrologic model. Information for the unrestricted area is provided below:

- $\text{Area}_{\text{unrest}}$ = 0.00031 square miles (0.2 acres)
- CN = 74
- Lag time = 6 minutes (0.6 * t_c = 10 minutes)

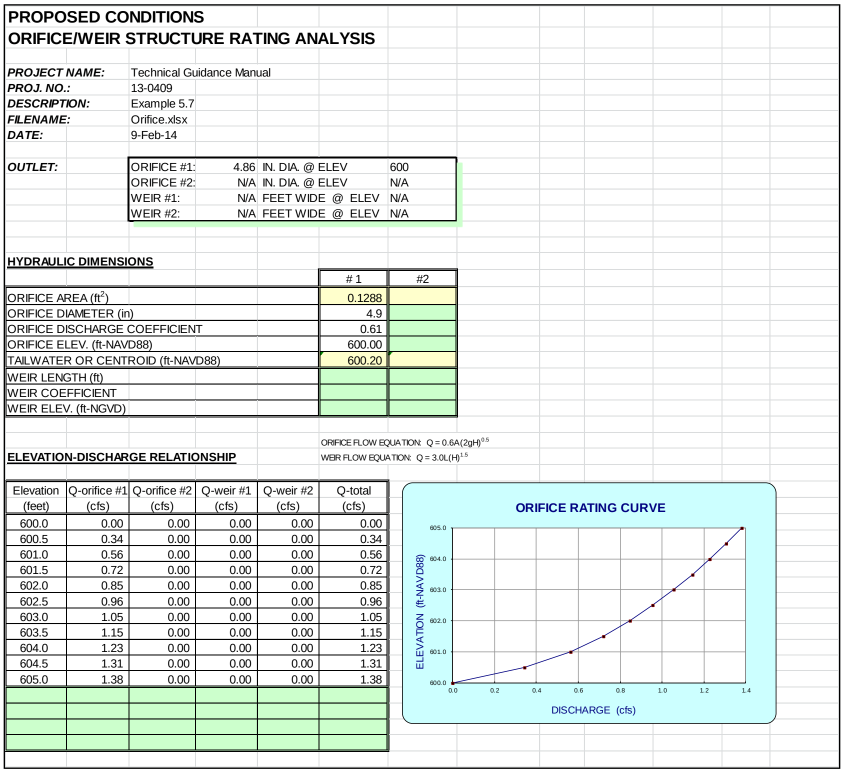
A new subbasin (*Undetained*) must be added to the HEC-HMS model. As shown in the next HEC-HMS screenshot, the undetained area is not routed to the proposed **detention facility**. From the HEC-HMS model, the 100-year, 24-hour unrestricted release rate from the **site** is **0.12 cfs**. The **allowable release rate** for the **site** must be adjusted for the unrestricted release rate by the following:

Maximum **allowable release rate** – unrestricted release rate = net **allowable release rate**,

The net **allowable release rate** = 1.50 cfs – 0.12 cfs = 1.38 cfs

It is assumed that there is no tailwater condition for the **site**. Based on the **site** conditions, it was determined that five feet of depth is possible in the proposed detention basin.

Step 4: Use the orifice equation spreadsheet to size the restrictor. Using the elevation-discharge spreadsheet, a 4.9-inch diameter restrictor is needed to pass the net **allowable release rate** of 1.38 cfs at the HWL of 605 ft.



PROPOSED CONDITIONS
ORIFICE/WEIR STRUCTURE RATING ANALYSIS

PROJECT NAME:	Technical Guidance Manual
PROJ. NO.:	13-0409
DESCRIPTION:	Example 5.7
FILENAME:	Orifice.xlsx
DATE:	9-Feb-14

OUTLET:	ORIFICE #1:	4.86	IN. DIA. @ ELEV	600
	ORIFICE #2:	N/A	IN. DIA. @ ELEV	N/A
	WEIR #1:	N/A	FEET WIDE @ ELEV	N/A
	WEIR #2:	N/A	FEET WIDE @ ELEV	N/A

HYDRAULIC DIMENSIONS

	# 1	#2
ORIFICE AREA (ft^2)	0.1288	
ORIFICE DIAMETER (in)	4.9	
ORIFICE DISCHARGE COEFFICIENT	0.61	
ORIFICE ELEV. (ft-NAVD88)	600.00	
TAILWATER OR CENTROID (ft-NAVD88)	600.20	
WEIR LENGTH (ft)		
WEIR COEFFICIENT		
WEIR ELEV. (ft-NGVD)		

ORIFICE FLOW EQUATION: $Q = 0.6A(2gH)^{0.5}$
WEIR FLOW EQUATION: $Q = 3.0L(H)^{1.5}$

ELEVATION-DISCHARGE RELATIONSHIP

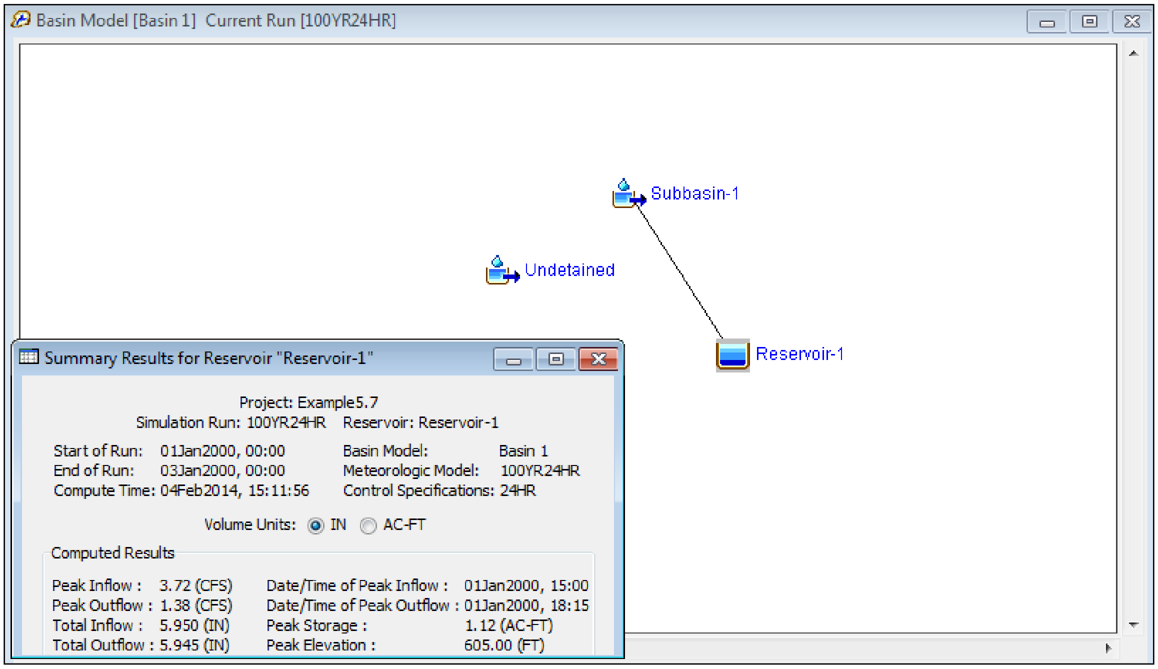
Elevation (feet)	Q-orifice #1 (cfs)	Q-orifice #2 (cfs)	Q-weir #1 (cfs)	Q-weir #2 (cfs)	Q-total (cfs)
600.0	0.00	0.00	0.00	0.00	0.00
600.5	0.34	0.00	0.00	0.00	0.34
601.0	0.56	0.00	0.00	0.00	0.56
601.5	0.72	0.00	0.00	0.00	0.72
602.0	0.85	0.00	0.00	0.00	0.85
602.5	0.96	0.00	0.00	0.00	0.96
603.0	1.05	0.00	0.00	0.00	1.05
603.5	1.15	0.00	0.00	0.00	1.15
604.0	1.23	0.00	0.00	0.00	1.23
604.5	1.31	0.00	0.00	0.00	1.31
605.0	1.38	0.00	0.00	0.00	1.38

Step 5: Use the elevation-storage spreadsheet to obtain the appropriate relationship to enter into HEC-HMS. For each iteration, the HEC-HMS stage-storage relationship must be revised under *Paired Data* to match the iterated spreadsheet below.

POND:	Proposed Detention Facility		Centerline Elevation	
JOB NO.	130409	Side Slopes	Orifice Radius:	
PROJECT:	Example 5.7	1	Orifice Coeff:	
FILE:	Storage.xls	4	Weir Elevation:	
DATE:	2/4/2014		Length of Weir	
DA			Weir Coeff	

Elevation (ft)	INC	Area (ft2)	Area (ac)	Average Area (ac)	Incremental Storage (ac-ft)	Cummulative Storage (ac-ft)
600.00		6,080	0.140			
				0.155	0.15	
601.00		7,392	0.170			0.155
				0.186	0.19	
602.00		8,831	0.203			0.341
				0.221	0.22	
603.00		10,399	0.239			0.562
				0.258	0.26	
604.00		12,094	0.278			0.820
				0.299	0.30	
605.00		13,918	0.320			1.118

Using the elevation-storage spreadsheet, iteratively enter the elevation-storage relationship until the proposed basin fills up for the 100-year, 24-hour **storm event**. As shown in the figure below, a volume of 1.12 acre-feet is required for this **site**.



Project: Example5.7
Simulation Run: 100YR24HR Reservoir: Reservoir-1

Start of Run: 01Jan2000, 00:00 Basin Model: Basin 1
End of Run: 03Jan2000, 00:00 Meteorologic Model: 100YR24HR
Compute Time: 04Feb2014, 15:11:56 Control Specifications: 24HR

Volume Units: ● IN ○ AC-FT

Computed Results

Peak Inflow : 3.72 (CFS) Date/Time of Peak Inflow : 01Jan2000, 15:00
Peak Outflow : 1.38 (CFS) Date/Time of Peak Outflow : 01Jan2000, 18:15
Total Inflow : 5.950 (IN) Peak Storage : 1.12 (AC-FT)
Total Outflow : 5.945 (IN) Peak Elevation : 605.00 (FT)

From the results of the HEC-HMS analysis for the 100-year, 24-hour **storm event**, the HWL of the proposed **detention facility** is 605.00 ft.

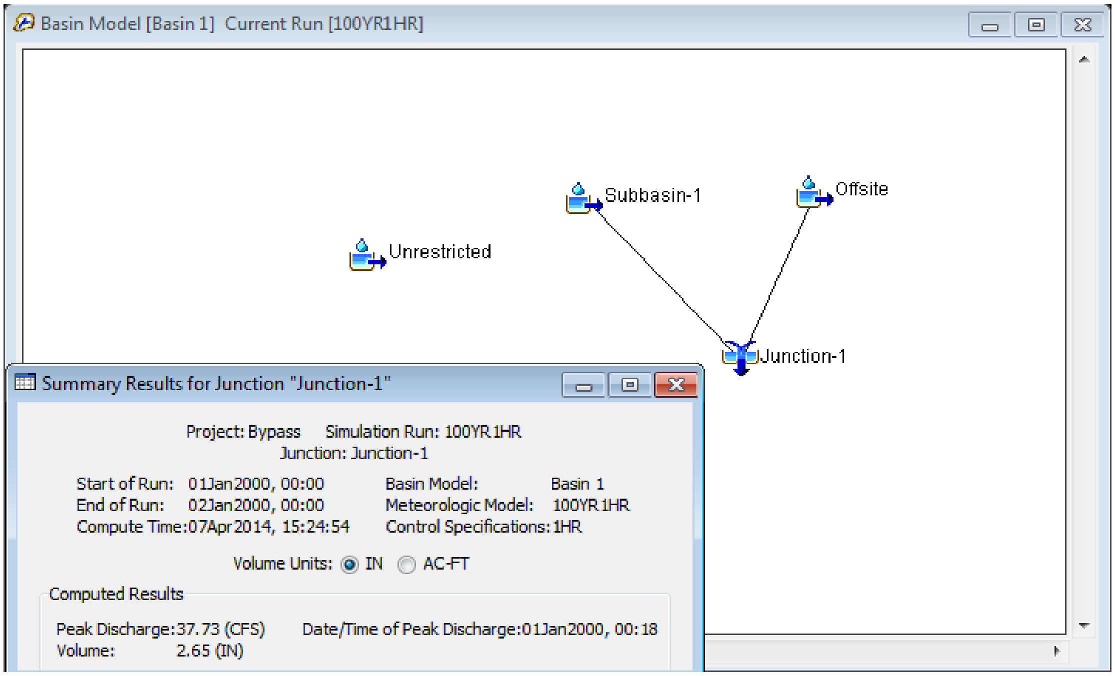
Step 6: Determine the 100-year peak flowrate from offsite and onsite **tributary areas** to the **detention facility**. An emergency overflow weir is needed to safely pass offsite flows, and onsite flows in the event the restrictor becomes blocked. Information for the offsite area is provided below:

- Offsite area = 0.00469 square miles (3.0 acres)
- CN = 89
- Lag time = 7.2 minutes (0.6 * t_c = 12 minutes)

A new subbasin (*Offsite*) is required to determine the peak flowrate of the offsite **tributary area** to the **detention facility** and a *junction (Junction-1)* is required to add the **runoff** hydrographs from the onsite and offsite **tributary areas**. In determining the design flowrate, the unadjusted CN for the **site** should be used. The reduced CN is to be used only for the determination of required detention volume; all other **stormwater** design should be based on the composite CN for the **site**.

A **critical duration analysis** must be run for the offsite plus the onsite **tributary area** to determine the 100-year peak flowrate. The overflow weir must, at a minimum, be sized for 1.0 cfs/acre of **tributary area**, so a check must be done following the HEC-HMS **critical duration analysis**. The results of the HEC-HMS analysis show that the 1-hour is the critical duration, with a peak flowrate of 37.7 cfs.

37.7 cfs = 4.7 cfs/acre > 1 cfs/acre → OK



Project: Bypass   Simulation Run: 100YR1HR
Junction: Junction-1

Start of Run: 01Jan2000, 00:00   Basin Model: Basin 1
End of Run: 02Jan2000, 00:00   Meteorologic Model: 100YR1HR
Compute Time:07Apr2014, 15:24:54   Control Specifications:1HR

Volume Units: ● IN ○ AC-FT

Computed Results

Peak Discharge:37.73 (CFS)   Date/Time of Peak Discharge:01Jan2000, 00:18
Volume: 2.65 (IN)

Step 7: Size the overflow weir so that it can safely pass the 100-year critical duration flowrate from all onsite and offsite **tributary area**.

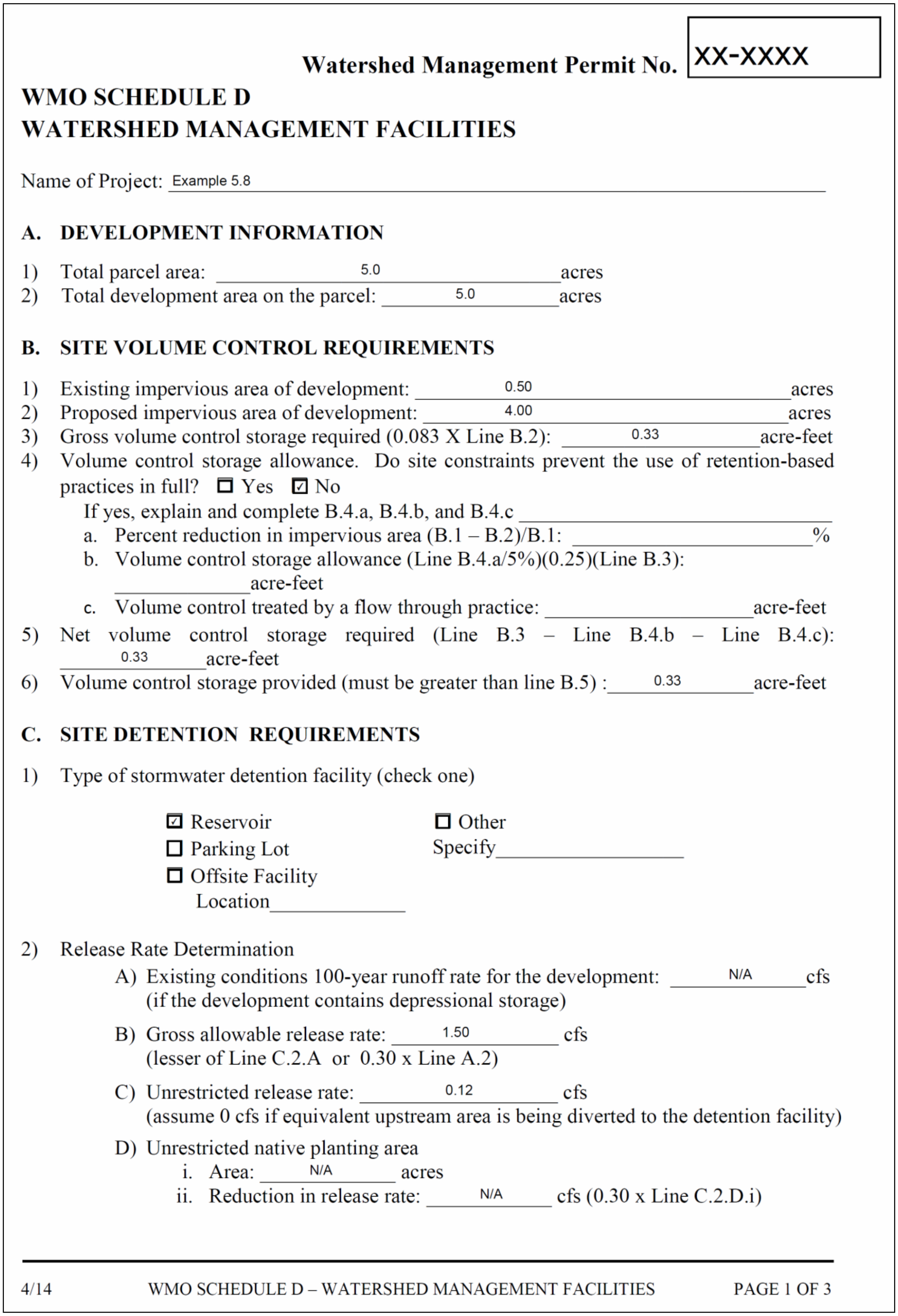
Using the weir equation,

$$Q = C \times L \times H^{3/2}$$

Where,
Q = flowrate (31.0 cfs)
C = weir coefficient (assume 3.0)
L = length of weir (ft)
H = head on weir (ft, assume 1 ft)

The weir equation becomes: $Q = 37.7 \text{ cfs} = 3.0 \times L \times (1)^{3/2}$, and solving for L,

L = 12.6 ft, which is the minimum length of weir required to pass the 100-year peak flowrate (with one foot of head) for the onsite and offsite area. In traditional detention basins, the emergency overflow weir is a trapezoidal or rectangular channel set at the HWL of the **detention facility**. It is also acceptable to provide the emergency overflow weir as a wall at the outlet control **structure** (within the manhole). The top of the wall is set to the HWL of the **detention facility**. Details for both types of overflow **structures** are provided in Appendix C. In either case, it must be demonstrated that the emergency overflow weir can safely pass the peak 100-year flowrate for the total **tributary area** (offsite and onsite) coming to the **detention facility**.



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D
WATERSHED MANAGEMENT FACILITIES

Name of Project: Example 5.8

A. DEVELOPMENT INFORMATION

1) Total parcel area: 5.0 acres
2) Total development area on the parcel: 5.0 acres

B. SITE VOLUME CONTROL REQUIREMENTS

1) Existing impervious area of development: 0.50 acres
2) Proposed impervious area of development: 4.00 acres
3) Gross volume control storage required (0.083 X Line B.2): 0.33 acre-feet
4) Volume control storage allowance. Do site constraints prevent the use of retention-based practices in full? ☐ Yes ☑ No
   If yes, explain and complete B.4.a, B.4.b, and B.4.c ____________
   a. Percent reduction in impervious area (B.1 – B.2)/B.1: ____________%
   b. Volume control storage allowance (Line B.4.a/5%)(0.25)(Line B.3): ____________acre-feet
   c. Volume control treated by a flow through practice: ____________acre-feet
5) Net volume control storage required (Line B.3 – Line B.4.b – Line B.4.c): 0.33 acre-feet
6) Volume control storage provided (must be greater than line B.5) : 0.33 acre-feet

C. SITE DETENTION REQUIREMENTS

1) Type of stormwater detention facility (check one)

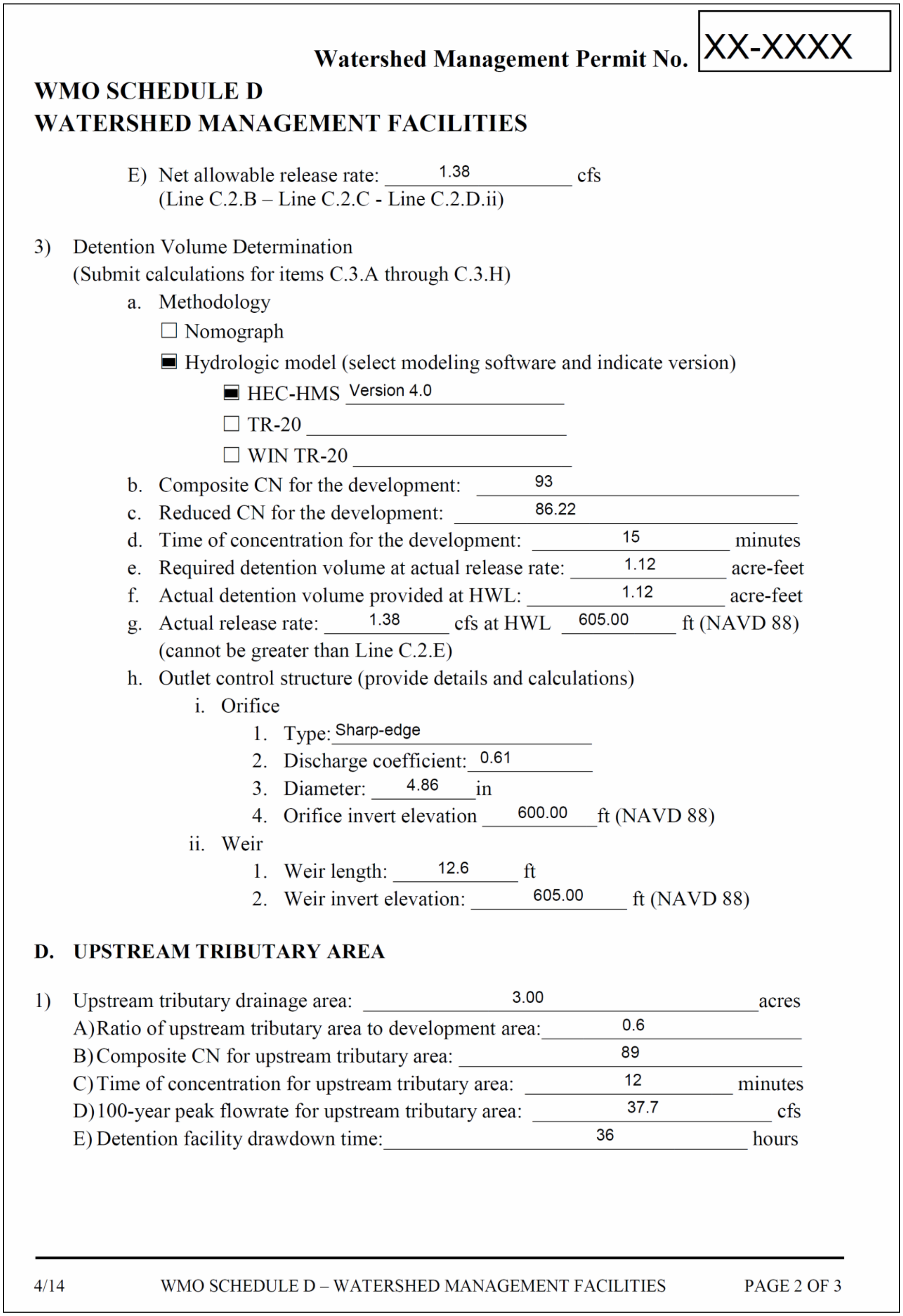
☑ Reservoir
☐ Parking Lot
☐ Offsite Facility
Location____________

☐ Other
Specify____________

2) Release Rate Determination
   A) Existing conditions 100-year runoff rate for the development: N/A cfs (if the development contains depressional storage)
   B) Gross allowable release rate: 1.50 cfs (lesser of Line C.2.A or 0.30 x Line A.2)
   C) Unrestricted release rate: 0.12 cfs (assume 0 cfs if equivalent upstream area is being diverted to the detention facility)
   D) Unrestricted native planting area
      i. Area: N/A acres
      ii. Reduction in release rate: N/A cfs (0.30 x Line C.2.D.i)

4/14 WMO SCHEDULE D – WATERSHED MANAGEMENT FACILITIES PAGE 1 OF 3

Schedule D Form for Example 5.8 (Page 1 of 3)



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D
WATERSHED MANAGEMENT FACILITIES

E) Net allowable release rate: 1.38 cfs
(Line C.2.B – Line C.2.C - Line C.2.D.ii)

3) Detention Volume Determination
(Submit calculations for items C.3.A through C.3.H)

a. Methodology
   ☐ Nomograph
   ☒ Hydrologic model (select modeling software and indicate version)
      ☒ HEC-HMS Version 4.0
      ☐ TR-20
      ☐ WIN TR-20

b. Composite CN for the development: 93

c. Reduced CN for the development: 86.22

d. Time of concentration for the development: 15 minutes

e. Required detention volume at actual release rate: 1.12 acre-feet

f. Actual detention volume provided at HWL: 1.12 acre-feet

g. Actual release rate: 1.38 cfs at HWL 605.00 ft (NAVD 88)
(cannot be greater than Line C.2.E)

h. Outlet control structure (provide details and calculations)

i. Orifice
1. Type: Sharp-edge
2. Discharge coefficient: 0.61
3. Diameter: 4.86 in
4. Orifice invert elevation 600.00 ft (NAVD 88)

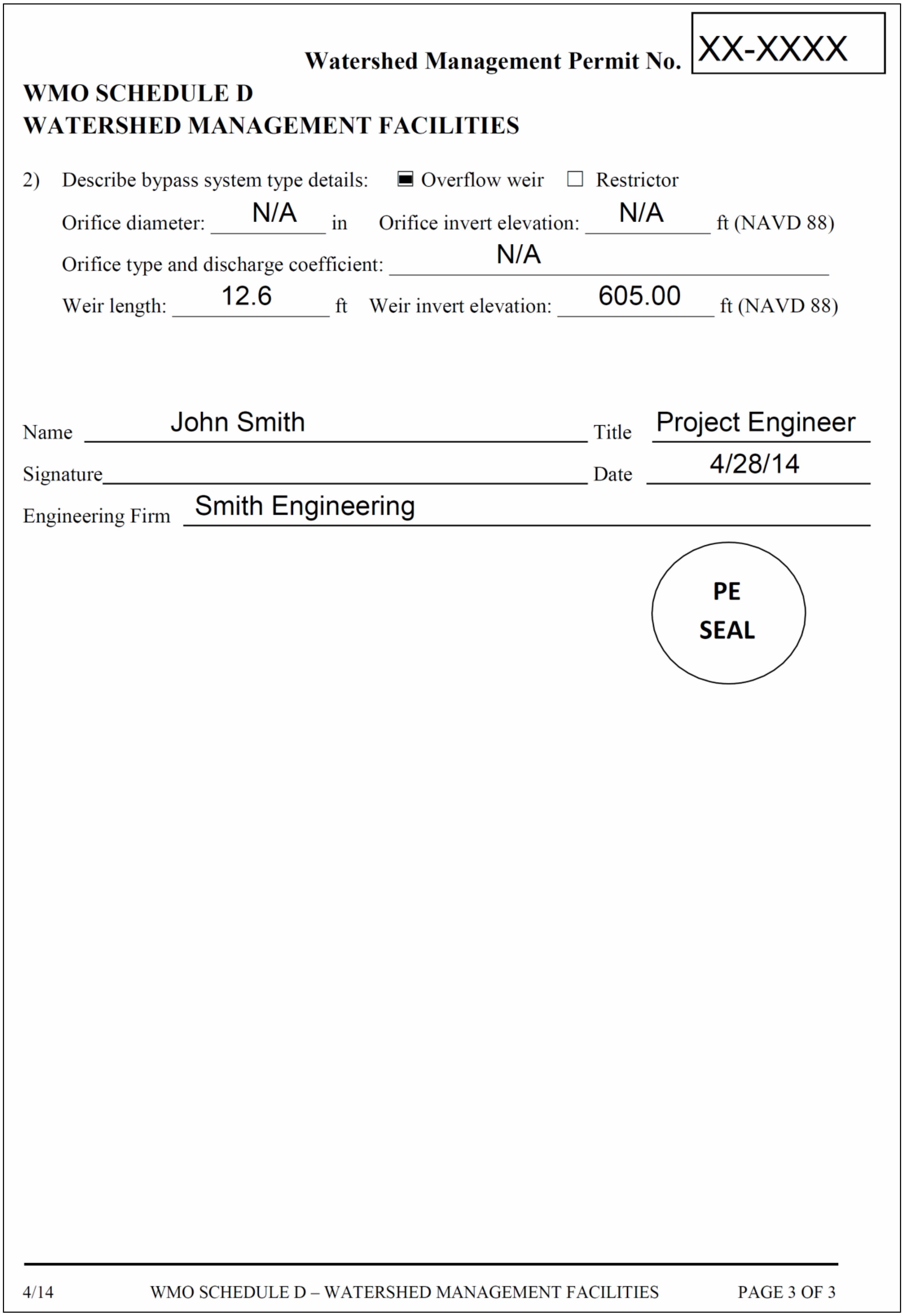
ii. Weir
1. Weir length: 12.6 ft
2. Weir invert elevation: 605.00 ft (NAVD 88)

D. UPSTREAM TRIBUTARY AREA

1) Upstream tributary drainage area: 3.00 acres
A) Ratio of upstream tributary area to development area: 0.6
B) Composite CN for upstream tributary area: 89
C) Time of concentration for upstream tributary area: 12 minutes
D) 100-year peak flowrate for upstream tributary area: 37.7 cfs
E) Detention facility drawdown time: 36 hours

4/14 WMO SCHEDULE D – WATERSHED MANAGEMENT FACILITIES PAGE 2 OF 3

Schedule D Form for Example 5.8 (Page 2 of 3)



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D
WATERSHED MANAGEMENT FACILITIES

2) Describe bypass system type details: ☒ Overflow weir ☐ Restrictor

Orifice diameter: N/A in Orifice invert elevation: N/A ft (NAVD 88)

Orifice type and discharge coefficient: N/A

Weir length: 12.6 ft Weir invert elevation: 605.00 ft (NAVD 88)

Name John Smith Title Project Engineer

Signature Date 4/28/14

Engineering Firm Smith Engineering

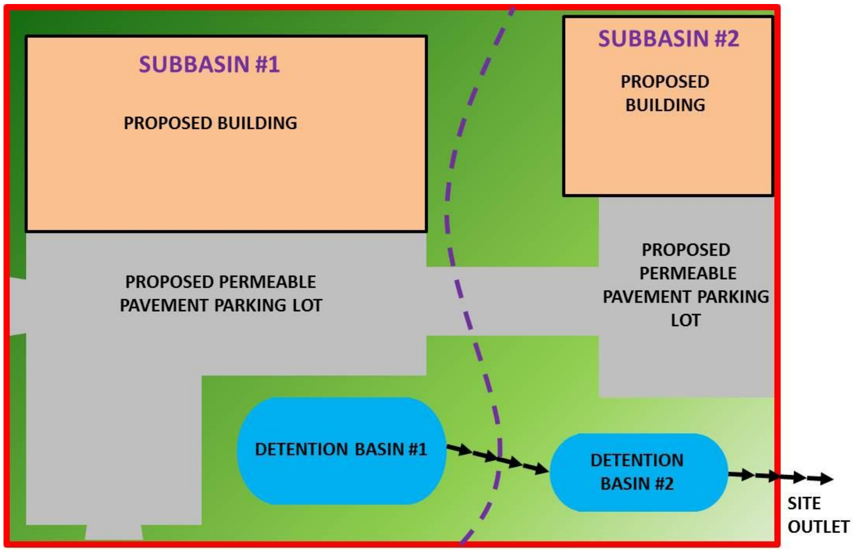
PE
SEAL

4/14 WMO SCHEDULE D – WATERSHED MANAGEMENT FACILITIES PAGE 3 OF 3

Schedule D Form for Example 5.8 (Page 3 of 3)

Example 5.9 – Detention Ponds in Series with Tailwater

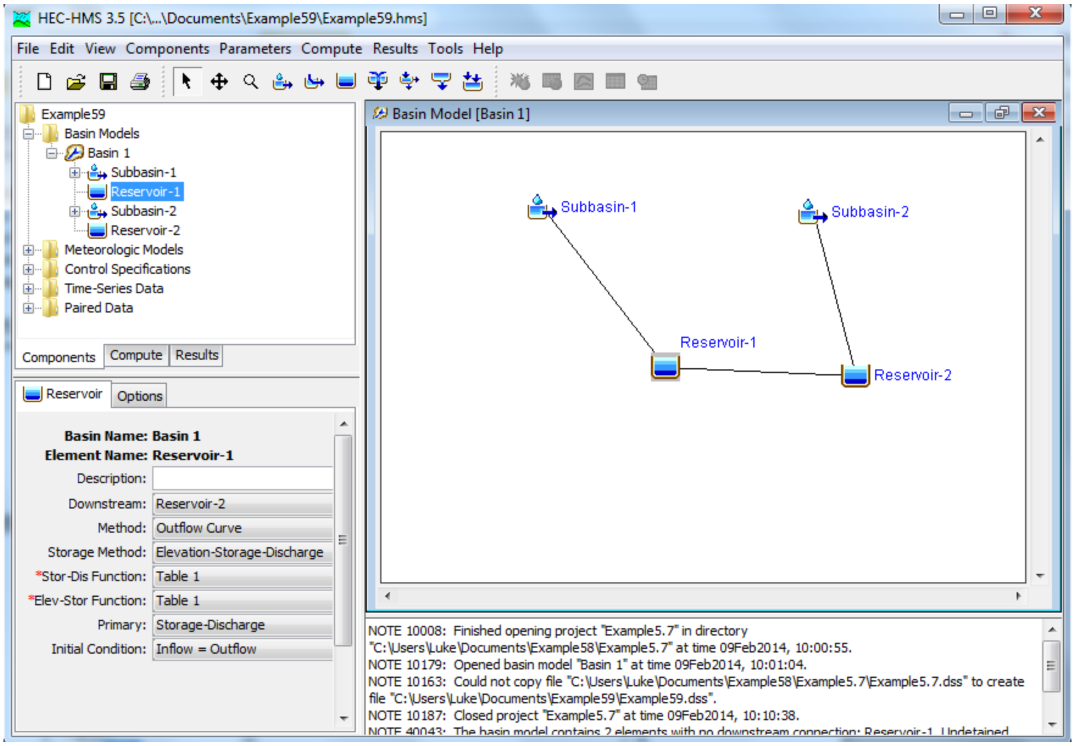
As shown in the figure below, the required detention volume for a proposed 10-acre commercial **development** will be provided in two detention basins in series. Based on **Cook County** one-foot topography, there is no offsite **tributary area** to the project **site**. However, Detention Basin 2 will discharge to a receiving stream with a **100-year flood elevation** of 699 feet. Based on the proposed grading plan, the **site** is separated into two subbasins. Determine the requirements of the two detention basins based on the WMO and **TGM**.



Subbasin 1
Area = 0.009375 square miles (6 acres)
Impervious Area = 2.25 acres
Curve Number = 92
Adjusted Curve Number = 88.79 (assumes 1 inch on volume control storage)
SCS Lag Time = 9 minutes

Subbasin 2
Area = 0.00625 square miles (4 acres)
Impervious Area = 1.25 acres
Curve Number = 88
Adjusted Curve Number = 85.36 (assumes 1 inch on volume control storage)
SCS Lag Time = 6 minutes

Step 1: Determine the required storage volumes for each of the proposed detention basins. A HEC-HMS model must be developed for the onsite area that will be detained. As shown in the next HEC-HMS screenshot, Subbasin-1 drains to Reservoir-1, and the outflow of Reservoir-1 then drains into Reservoir-2, along with Subbasin-2.



Basin Name: Basin 1
Element Name: Reservoir-1
Description:
Downstream: Reservoir-2
Method: Outflow Curve
Storage Method: Elevation-Storage-Discharge
*Stor-Dis Function: Table 1
*Elev-Stor Function: Table 1
Primary: Storage-Discharge
Initial Condition: Inflow = Outflow

```
NOTE 10008:  Finished opening project "Example5.7" in directory
"C:\Users\Luke\Documents\Example58\Example5.7" at time 09Feb2014, 10:00:55.
NOTE 10179:  Opened basin model "Basin 1" at time 09Feb2014, 10:01:04.
NOTE 10163:  Could not copy file "C:\Users\Luke\Documents\Example58\Example5.7\Example5.7.dss" to create
file "C:\Users\Luke\Documents\Example59\Example59.dss".
NOTE 10187:  Closed project "Example5.7" at time 09Feb2014, 10:10:38.
NOTE 40043:  The basin model contains 2 elements with no downstream connection: Reservoir-1, Undetained
```

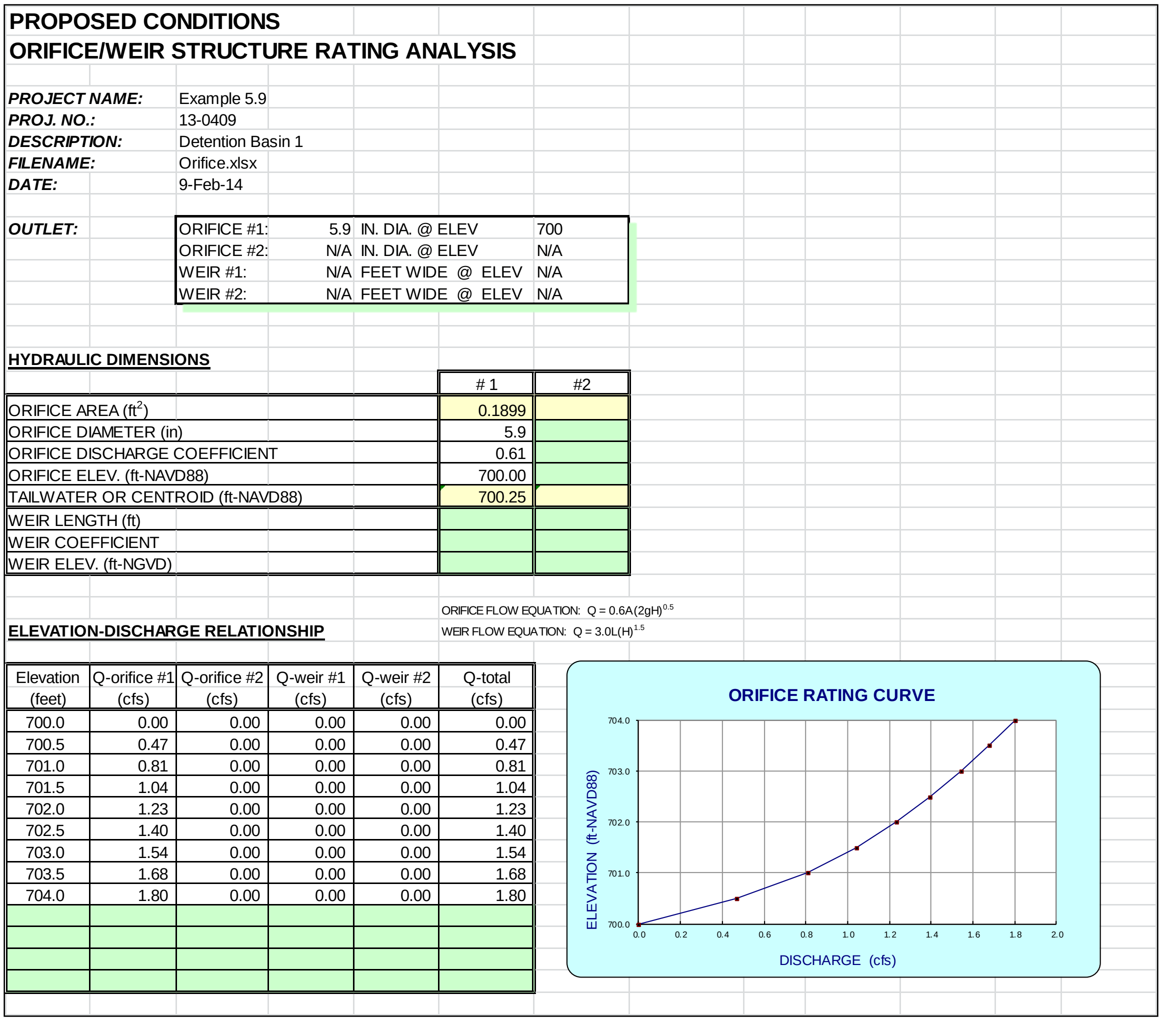
It is assumed there are no unrestricted areas for the proposed **development**. Therefore, the **allowable release rate** for the **site** is simply determined by 0.30 cfs/acre times the acreage. It should be noted that as long as the release rate at the **site's** outlet (Reservoir-2 outflow) is less than or equal to 3 cfs (0.30 cfs/acre x 10 acres), the proposed detention configuration is flexible. For example, if **site** conditions prevent Reservoir-1 from detaining Subbasin-1 at 0.30 cfs/acre, Reservoir-1 can be sized to underdetain its **tributary area**, so long as Reservoir-2 is oversized to meet the **allowable release rate** for the **site**.

From the proposed grading plan, it is determined that four feet of bounce can be accommodated in the proposed detention basins. The elevations of the **site** allow the following configurations of the detention basins:

Detention Basin 1 (Reservoir-1)
Normal Water Level (NWL) = 700 ft
High Water Level (HWL) = 704 ft
Tailwater Condition = none

Detention Basin 2 (Reservoir-2)
Normal Water Level (NWL) = 698 ft
High Water Level (HWL) = 702 ft
Tailwater Condition = 699 ft (**100-year flood elevation** of receiving stream)

Step 2: Using the orifice equation, determine the restrictor size for the proposed detention basins. Since the outflow of the restrictor for Reservoir-2 must be less than or equal to 3.0 cfs, the configuration of the detention basins is flexible as long as the ultimate outflow of the **site** meets this release. The restrictor for Reservoir-1 can release 1.8 cfs at the HWL of 704 ft and the restrictor for Reservoir-2 can release 3.0 cfs at the HWL of 702 ft. From the orifice equation spreadsheet (shown below), a 5.9-inch diameter restrictor is needed to convey 1.8 cfs at the HWL of 704 ft.



PROPOSED CONDITIONS
ORIFICE/WEIR STRUCTURE RATING ANALYSIS

PROJECT NAME:	Example 5.9
PROJ. NO.:	13-0409
DESCRIPTION:	Detention Basin 1
FILENAME:	Orifice.xlsx
DATE:	9-Feb-14

OUTLET:

ORIFICE #1:	5.9	IN. DIA. @ ELEV	700
ORIFICE #2:	N/A	IN. DIA. @ ELEV	N/A
WEIR #1:	N/A	FEET WIDE @ ELEV	N/A
WEIR #2:	N/A	FEET WIDE @ ELEV	N/A

HYDRAULIC DIMENSIONS

	# 1	#2
ORIFICE AREA (ft^2)	0.1899	
ORIFICE DIAMETER (in)	5.9	
ORIFICE DISCHARGE COEFFICIENT	0.61	
ORIFICE ELEV. (ft-NAVD88)	700.00	
TAILWATER OR CENTROID (ft-NAVD88)	700.25	
WEIR LENGTH (ft)		
WEIR COEFFICIENT		
WEIR ELEV. (ft-NGVD)		

ORIFICE FLOW EQUATION: $Q = 0.6A(2gH)^{0.5}$
WEIR FLOW EQUATION: $Q = 3.0L(H)^{1.5}$

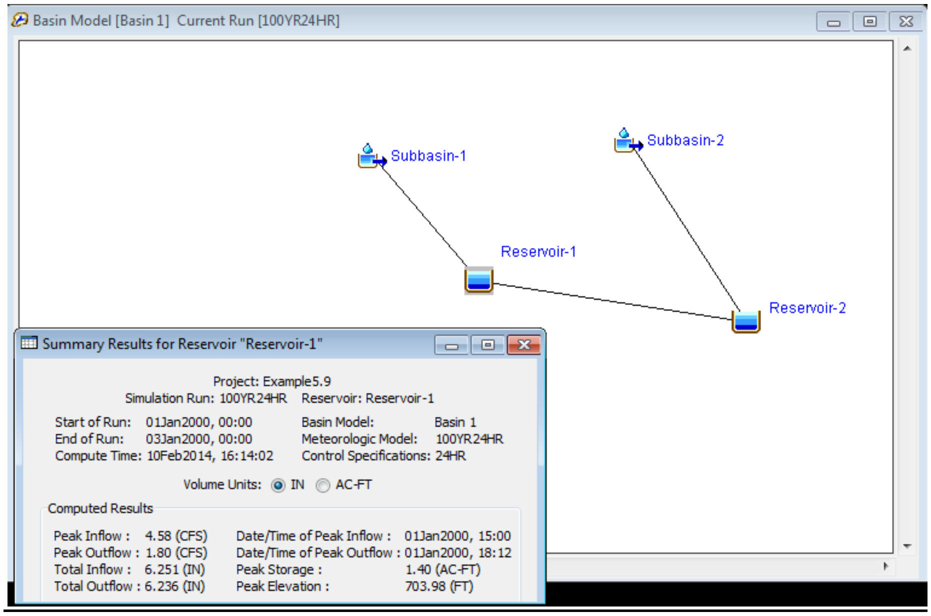
ELEVATION-DISCHARGE RELATIONSHIP

Elevation (feet)	Q-orifice #1 (cfs)	Q-orifice #2 (cfs)	Q-weir #1 (cfs)	Q-weir #2 (cfs)	Q-total (cfs)
700.0	0.00	0.00	0.00	0.00	0.00
700.5	0.47	0.00	0.00	0.00	0.47
701.0	0.81	0.00	0.00	0.00	0.81
701.5	1.04	0.00	0.00	0.00	1.04
702.0	1.23	0.00	0.00	0.00	1.23
702.5	1.40	0.00	0.00	0.00	1.40
703.0	1.54	0.00	0.00	0.00	1.54
703.5	1.68	0.00	0.00	0.00	1.68
704.0	1.80	0.00	0.00	0.00	1.80

Step 3: Using the elevation-storage spreadsheet and solving iteratively, it is determined that 1.40 acre-feet of storage volume is required for Detention Basin 1, as shown in the next HEC-HMS screenshot. For each iteration, the HEC-HMS stage-storage relationship must be revised under *Paired Data* to match the iterated spreadsheet below.

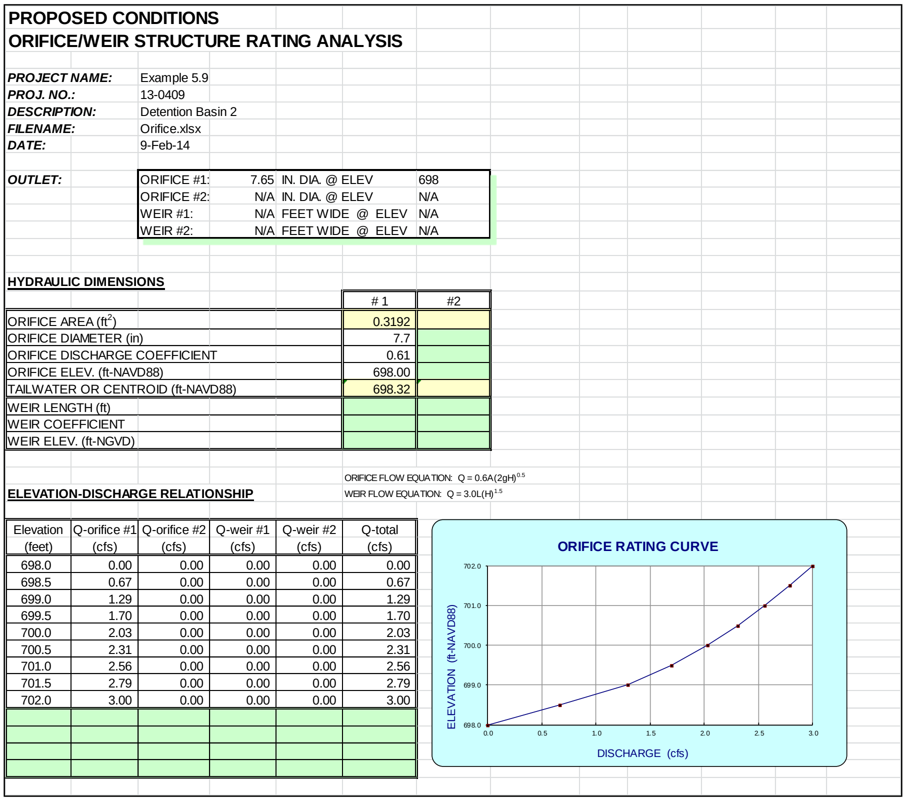
POND:	Proposed Detention Facility 1				Centerline Elevation	
JOB NO.		130409		Side Slopes	Orifice Radius:	
PROJECT:		Example 5.9		1	Orifice Coeff:	
FILE:		Storage.xls		4	Weir Elevation:	
DATE:		2/10/2014			Length of Weir	
DA					Weir Coeff	

		Area		Average	Incremental	Cummulative
Elevation	INC			Area	Storage	Storage
(ft)	0.25	(ft2)	(ac)	(ac)	(ac-ft)	(ac-ft)
700.00		11,543	0.265			
				0.285	0.29	
701.00		13,326	0.306			0.285
				0.328	0.33	
702.00		15,237	0.350			0.613
				0.373	0.37	
703.00		17,276	0.397			0.987
				0.421	0.42	
704.00		19,443	0.446			1.408



```
                    Project: Example5.9
      Simulation Run: 100YR24HR   Reservoir: Reservoir-1

Start of Run:    01Jan2000, 00:00      Basin Model:          Basin 1
End of Run:      03Jan2000, 00:00      Meteorologic Model:   100YR24HR
Compute Time: 10Feb2014, 16:14:02      Control Specifications: 24HR

              Volume Units: ◉ IN  ○ AC-FT

Computed Results

Peak Inflow :    4.58 (CFS)    Date/Time of Peak Inflow :   01Jan2000, 15:00
Peak Outflow :  1.80 (CFS)     Date/Time of Peak Outflow : 01Jan2000, 18:12
Total Inflow :    6.251 (IN)   Peak Storage :               1.40 (AC-FT)
Total Outflow : 6.236 (IN)     Peak Elevation :             703.98 (FT)
```

Step 4: Size the restrictor for Detention Basin 2 so that it releases 3.0 cfs at the HWL assuming full release conditions. As shown below, a 7.7-inch diameter restrictor is needed to convey the **allowable release rate**. However, since there is a tailwater condition on this restrictor, the detention volume must be sized assuming the 100-year tailwater of the receiving stream (699 ft). Therefore, another stage-discharge spreadsheet needs to be developed to determine the outflow assuming the 100-year tailwater of 699 ft.



PROPOSED CONDITIONS
ORIFICE/WEIR STRUCTURE RATING ANALYSIS

PROJECT NAME:	Example 5.9
PROJ. NO.:	13-0409
DESCRIPTION:	Detention Basin 2
FILENAME:	Orifice.xlsx
DATE:	9-Feb-14

OUTLET:

ORIFICE #1:	7.65	IN. DIA. @ ELEV	698
ORIFICE #2:	N/A	IN. DIA. @ ELEV	N/A
WEIR #1:	N/A	FEET WIDE @ ELEV	N/A
WEIR #2:	N/A	FEET WIDE @ ELEV	N/A

HYDRAULIC DIMENSIONS

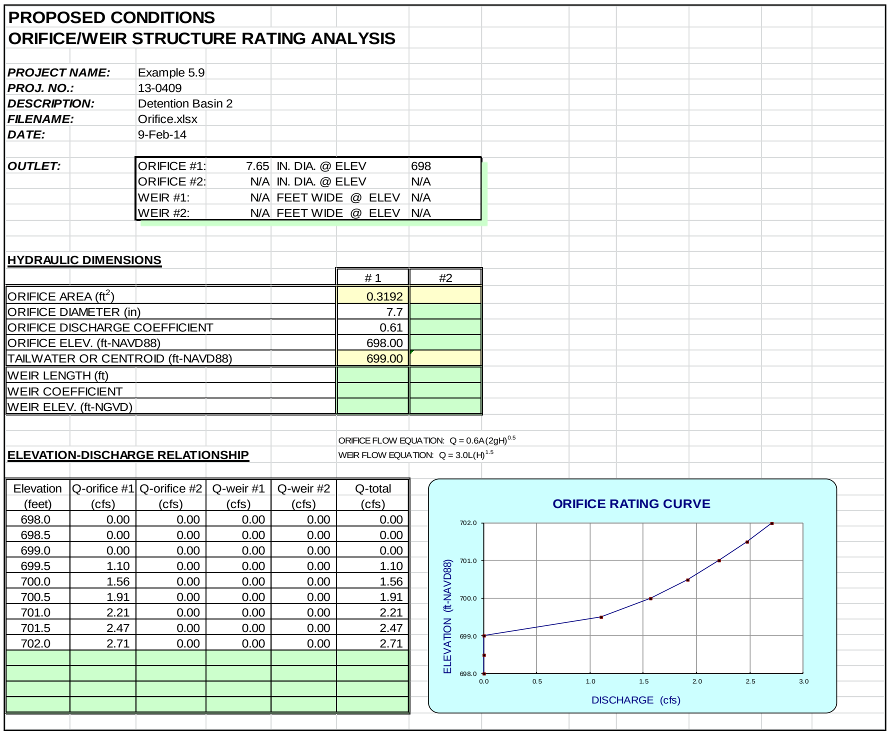
	# 1	#2
ORIFICE AREA (ft^2)	0.3192	
ORIFICE DIAMETER (in)	7.7	
ORIFICE DISCHARGE COEFFICIENT	0.61	
ORIFICE ELEV. (ft-NAVD88)	698.00	
TAILWATER OR CENTROID (ft-NAVD88)	698.32	
WEIR LENGTH (ft)		
WEIR COEFFICIENT		
WEIR ELEV. (ft-NGVD)		

ORIFICE FLOW EQUATION: $Q = 0.6A(2gH)^{0.5}$
WEIR FLOW EQUATION: $Q = 3.0L(H)^{1.5}$

ELEVATION-DISCHARGE RELATIONSHIP

Elevation (feet)	Q-orifice #1 (cfs)	Q-orifice #2 (cfs)	Q-weir #1 (cfs)	Q-weir #2 (cfs)	Q-total (cfs)
698.0	0.00	0.00	0.00	0.00	0.00
698.5	0.67	0.00	0.00	0.00	0.67
699.0	1.29	0.00	0.00	0.00	1.29
699.5	1.70	0.00	0.00	0.00	1.70
700.0	2.03	0.00	0.00	0.00	2.03
700.5	2.31	0.00	0.00	0.00	2.31
701.0	2.56	0.00	0.00	0.00	2.56
701.5	2.79	0.00	0.00	0.00	2.79
702.0	3.00	0.00	0.00	0.00	3.00

Restrictor for Detention Basin 2 assuming free flow (no tailwater)



PROPOSED CONDITIONS
ORIFICE/WEIR STRUCTURE RATING ANALYSIS

PROJECT NAME:	Example 5.9
PROJ. NO.:	13-0409
DESCRIPTION:	Detention Basin 2
FILENAME:	Orifice.xlsx
DATE:	9-Feb-14

OUTLET:

ORIFICE #1:	7.65	IN. DIA. @ ELEV	698
ORIFICE #2:	N/A	IN. DIA. @ ELEV	N/A
WEIR #1:	N/A	FEET WIDE @ ELEV	N/A
WEIR #2:	N/A	FEET WIDE @ ELEV	N/A

HYDRAULIC DIMENSIONS

	# 1	#2
ORIFICE AREA (ft^2)	0.3192	
ORIFICE DIAMETER (in)	7.7	
ORIFICE DISCHARGE COEFFICIENT	0.61	
ORIFICE ELEV. (ft-NAVD88)	698.00	
TAILWATER OR CENTROID (ft-NAVD88)	699.00	
WEIR LENGTH (ft)		
WEIR COEFFICIENT		
WEIR ELEV. (ft-NGVD)		

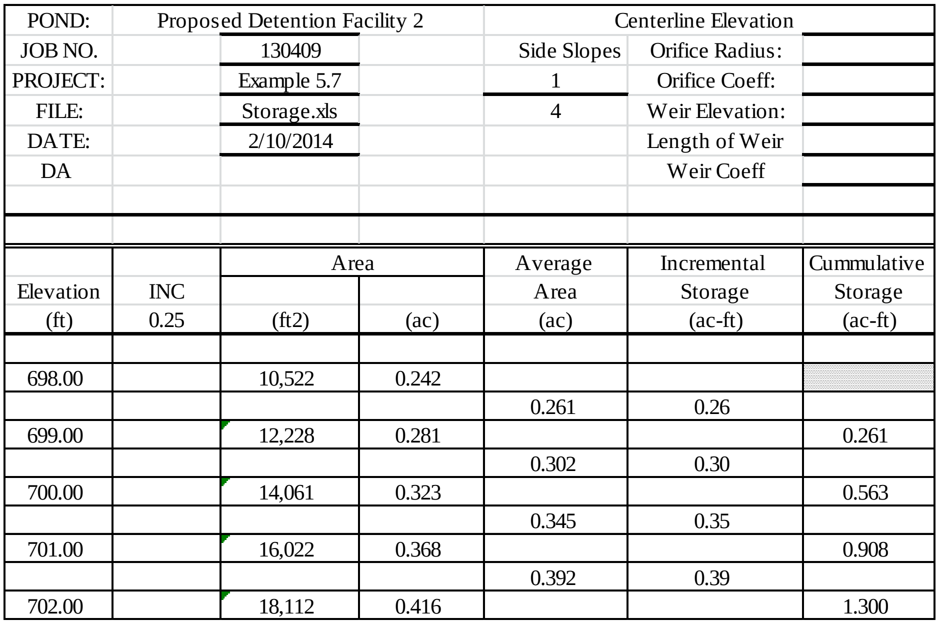
ORIFICE FLOW EQUATION: $Q = 0.6A(2gH)^{0.5}$
WEIR FLOW EQUATION: $Q = 3.0L(H)^{1.5}$

ELEVATION-DISCHARGE RELATIONSHIP

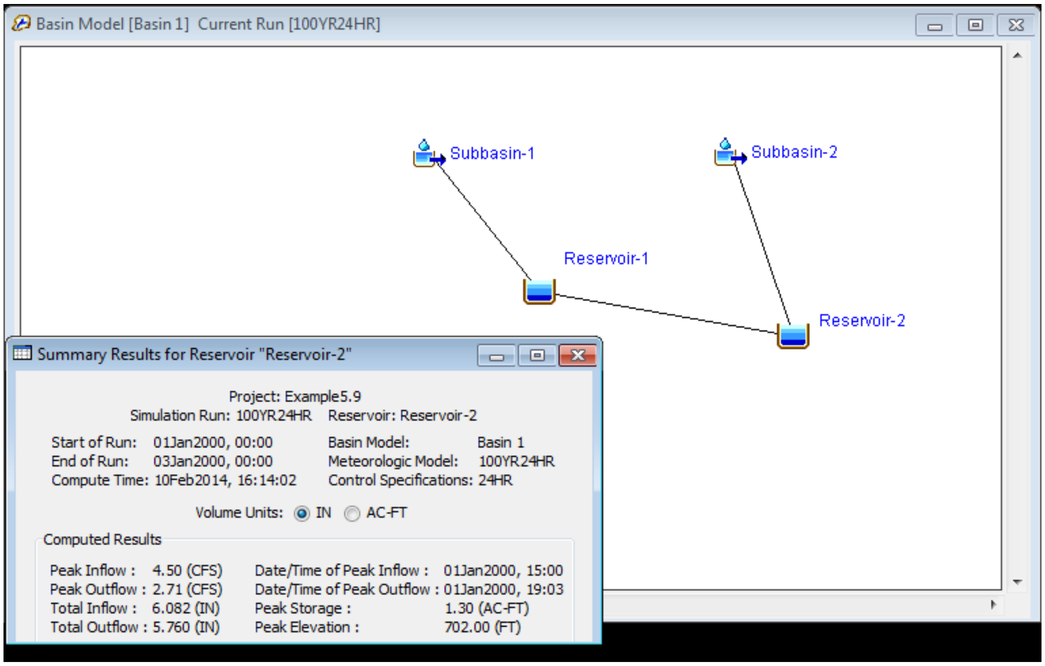
Elevation (feet)	Q-orifice #1 (cfs)	Q-orifice #2 (cfs)	Q-weir #1 (cfs)	Q-weir #2 (cfs)	Q-total (cfs)
698.0	0.00	0.00	0.00	0.00	0.00
698.5	0.00	0.00	0.00	0.00	0.00
699.0	0.00	0.00	0.00	0.00	0.00
699.5	1.10	0.00	0.00	0.00	1.10
700.0	1.56	0.00	0.00	0.00	1.56
700.5	1.91	0.00	0.00	0.00	1.91
701.0	2.21	0.00	0.00	0.00	2.21
701.5	2.47	0.00	0.00	0.00	2.47
702.0	2.71	0.00	0.00	0.00	2.71

Restrictor for Detention Basin 2 assuming tailwater of 699 ft

Step 5: Using the elevation-storage spreadsheet and solving iteratively, it is determined that 1.30 acre-feet of storage volume is required for Detention Basin 2, which allows the basin to fill up to its HWL of 702 ft and release at the **allowable release rate**. Note that when there is a 100-year tailwater on the detention basin, the release rate is only 2.7 cfs. When there is no tailwater condition, the outflow of the detention basin will be no greater than the maximum **allowable release rate** of 3.0 cfs.



POND:	Proposed Detention Facility 2			Centerline Elevation		
JOB NO.		130409		Side Slopes	Orifice Radius:	
PROJECT:		Example 5.7		1	Orifice Coeff:	
FILE:		Storage.xls		4	Weir Elevation:	
DATE:		2/10/2014			Length of Weir	
DA					Weir Coeff	
		Area		Average	Incremental	Cummulative
Elevation	INC			Area	Storage	Storage
(ft)	0.25	(ft2)	(ac)	(ac)	(ac-ft)	(ac-ft)
698.00		10,522	0.242			
				0.261	0.26	
699.00		12,228	0.281			0.261
				0.302	0.30	
700.00		14,061	0.323			0.563
				0.345	0.35	
701.00		16,022	0.368			0.908
				0.392	0.39	
702.00		18,112	0.416			1.300



Summary Results for Reservoir "Reservoir-2"

Project: Example5.9
Simulation Run: 100YR24HR Reservoir: Reservoir-2

Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time: 10Feb2014, 16:14:02

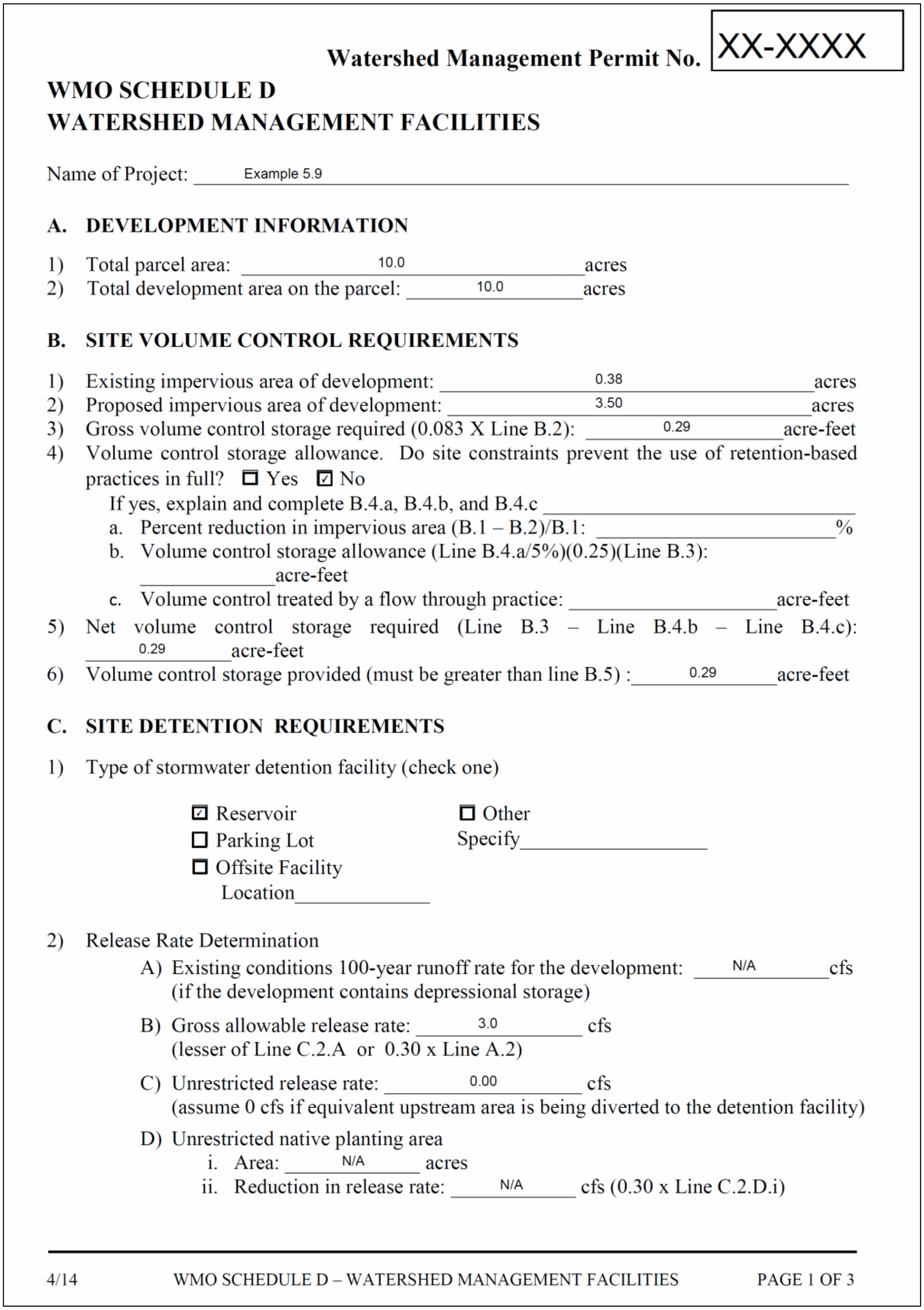
Basin Model: Basin 1
Meteorologic Model: 100YR24HR
Control Specifications: 24HR

Volume Units: ● IN ○ AC-FT

Computed Results

Peak Inflow : 4.50 (CFS)
Peak Outflow : 2.71 (CFS)
Total Inflow : 6.082 (IN)
Total Outflow : 5.760 (IN)

Date/Time of Peak Inflow : 01Jan2000, 15:00
Date/Time of Peak Outflow : 01Jan2000, 19:03
Peak Storage : 1.30 (AC-FT)
Peak Elevation : 702.00 (FT)



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D
WATERSHED MANAGEMENT FACILITIES

Name of Project: Example 5.9

A. DEVELOPMENT INFORMATION

1) Total parcel area: 10.0 acres
2) Total development area on the parcel: 10.0 acres

B. SITE VOLUME CONTROL REQUIREMENTS

1) Existing impervious area of development: 0.38 acres
2) Proposed impervious area of development: 3.50 acres
3) Gross volume control storage required (0.083 X Line B.2): 0.29 acre-feet
4) Volume control storage allowance. Do site constraints prevent the use of retention-based practices in full? ☐ Yes ☑ No
   If yes, explain and complete B.4.a, B.4.b, and B.4.c ____________
   a. Percent reduction in impervious area (B.1 – B.2)/B.1: __________ %
   b. Volume control storage allowance (Line B.4.a/5%)(0.25)(Line B.3): __________ acre-feet
   c. Volume control treated by a flow through practice: __________ acre-feet
5) Net volume control storage required (Line B.3 – Line B.4.b – Line B.4.c): 0.29 acre-feet
6) Volume control storage provided (must be greater than line B.5) : 0.29 acre-feet

C. SITE DETENTION REQUIREMENTS

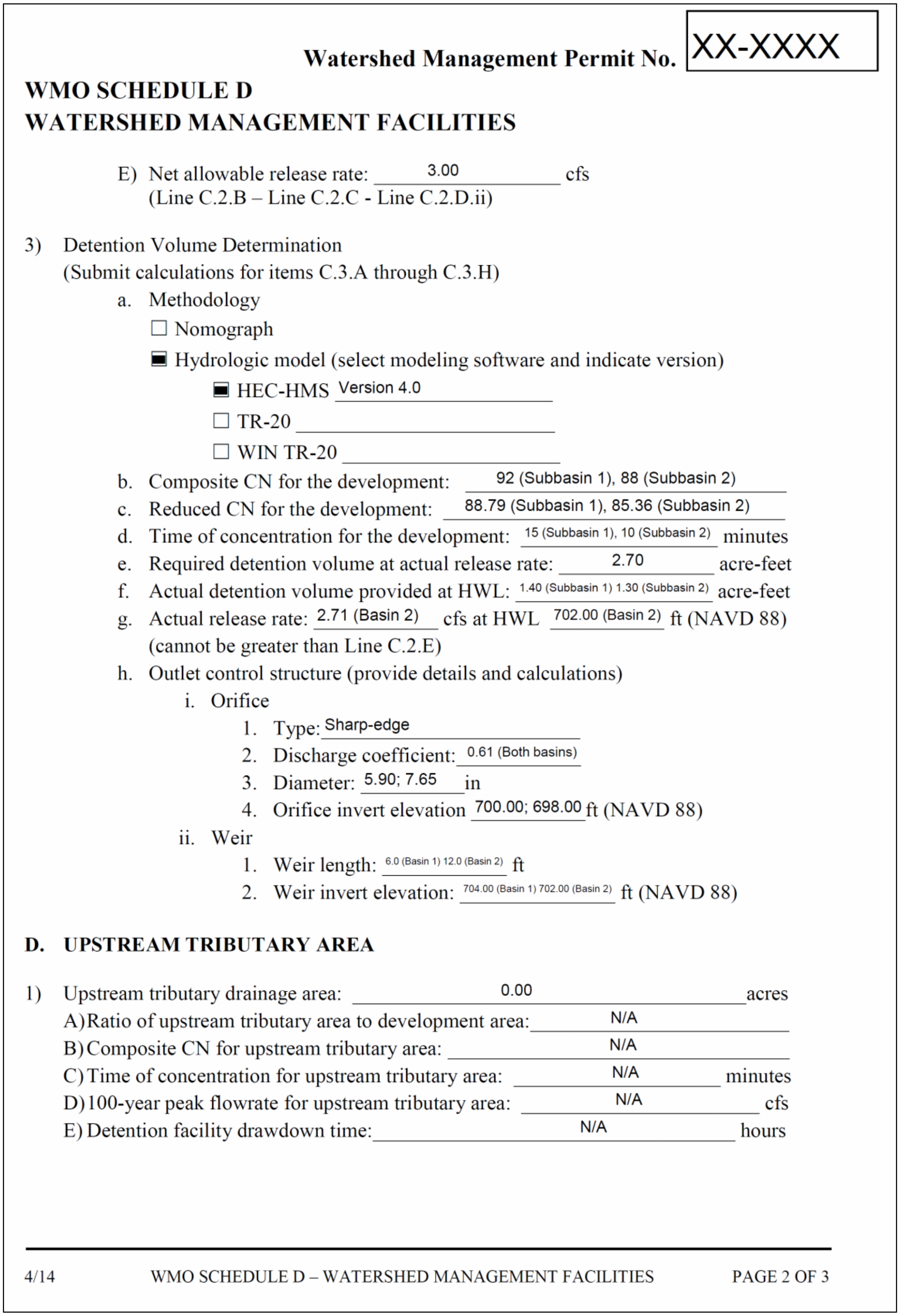
1) Type of stormwater detention facility (check one)

☑ Reservoir	☐ Other
☐ Parking Lot	Specify__________
☐ Offsite Facility Location__________	

2) Release Rate Determination
   A) Existing conditions 100-year runoff rate for the development: N/A cfs
   (if the development contains depressional storage)
   B) Gross allowable release rate: 3.0 cfs
   (lesser of Line C.2.A or 0.30 x Line A.2)
   C) Unrestricted release rate: 0.00 cfs
   (assume 0 cfs if equivalent upstream area is being diverted to the detention facility)
   D) Unrestricted native planting area
      i. Area: N/A acres
      ii. Reduction in release rate: N/A cfs (0.30 x Line C.2.D.i)

4/14 WMO SCHEDULE D – WATERSHED MANAGEMENT FACILITIES PAGE 1 OF 3

Schedule D Form for Example 5.9 (Page 1 of 3)



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D
WATERSHED MANAGEMENT FACILITIES

E) Net allowable release rate: 3.00 cfs
(Line C.2.B – Line C.2.C - Line C.2.D.ii)

3) Detention Volume Determination
(Submit calculations for items C.3.A through C.3.H)

a. Methodology
☐ Nomograph
☒ Hydrologic model (select modeling software and indicate version)
☒ HEC-HMS Version 4.0
☐ TR-20 ______
☐ WIN TR-20 ______

b. Composite CN for the development: 92 (Subbasin 1), 88 (Subbasin 2)

c. Reduced CN for the development: 88.79 (Subbasin 1), 85.36 (Subbasin 2)

d. Time of concentration for the development: 15 (Subbasin 1), 10 (Subbasin 2) minutes

e. Required detention volume at actual release rate: 2.70 acre-feet

f. Actual detention volume provided at HWL: 1.40 (Subbasin 1) 1.30 (Subbasin 2) acre-feet

g. Actual release rate: 2.71 (Basin 2) cfs at HWL 702.00 (Basin 2) ft (NAVD 88)
(cannot be greater than Line C.2.E)

h. Outlet control structure (provide details and calculations)

i. Orifice
1. Type: Sharp-edge
2. Discharge coefficient: 0.61 (Both basins)
3. Diameter: 5.90; 7.65 in
4. Orifice invert elevation 700.00; 698.00 ft (NAVD 88)

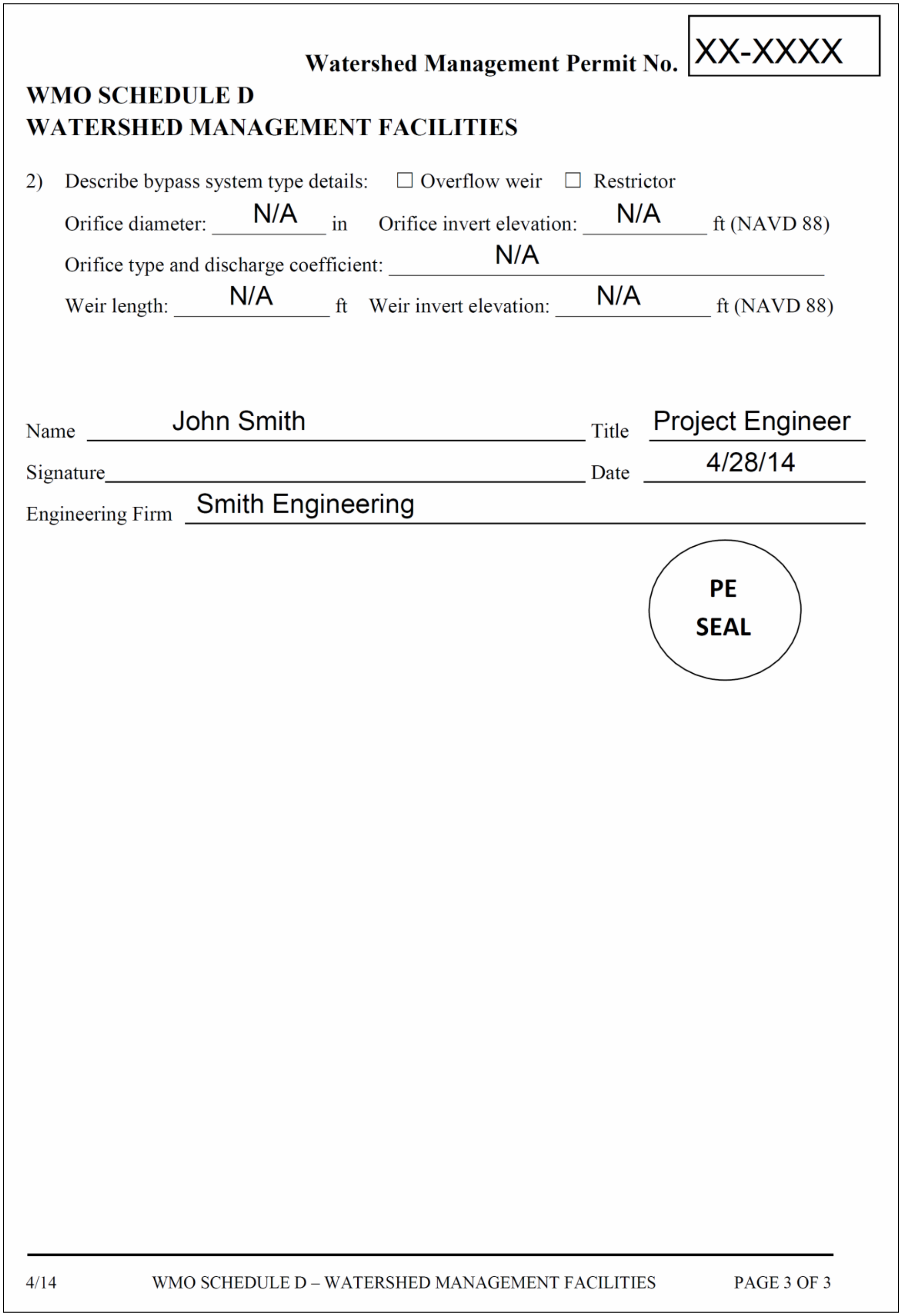
ii. Weir
1. Weir length: 6.0 (Basin 1) 12.0 (Basin 2) ft
2. Weir invert elevation: 704.00 (Basin 1) 702.00 (Basin 2) ft (NAVD 88)

D. UPSTREAM TRIBUTARY AREA

1) Upstream tributary drainage area: 0.00 acres
A) Ratio of upstream tributary area to development area: N/A
B) Composite CN for upstream tributary area: N/A
C) Time of concentration for upstream tributary area: N/A minutes
D) 100-year peak flowrate for upstream tributary area: N/A cfs
E) Detention facility drawdown time: N/A hours

4/14 WMO SCHEDULE D – WATERSHED MANAGEMENT FACILITIES PAGE 2 OF 3

Schedule D Form for Example 5.9 (Page 2 of 3)



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D
WATERSHED MANAGEMENT FACILITIES

2) Describe bypass system type details: ☐ Overflow weir ☐ Restrictor

Orifice diameter: N/A in Orifice invert elevation: N/A ft (NAVD 88)

Orifice type and discharge coefficient: N/A

Weir length: N/A ft Weir invert elevation: N/A ft (NAVD 88)

Name John Smith Title Project Engineer

Signature Date 4/28/14

Engineering Firm Smith Engineering

PE
SEAL

4/14 WMO SCHEDULE D – WATERSHED MANAGEMENT FACILITIES PAGE 3 OF 3

Schedule D Form for Example 5.9 (Page 3 of 3)

Site Detention Facilities within the Floodplain

The WMO (§504.13) allows **site detention facilities** in areas outside of the **regulatory floodway**, but within the **regulatory floodplain** provided that they:

1. Conform to all applicable requirements specified in Article 6, and in particular §602.18; and
2. Store the **site runoff** from the **development** such that the **allowable release rate** determined in §504.3 and adjusted in §504.5 and/or §504.6 is not exceeded, assuming a zero release rate (0 cfs) below the **BFE**.

Other Requirements

The WMO (§504.15) requires that **site detention facilities** be functional before occupancy permits are issued for residential and **non-residential subdivisions** or before **sanitary sewers** are placed in service. In addition, §504.16 requires that **site detention facilities** shall be functional for **developments** before **building** or road construction begins.

Offsite Facilities

If it is not practicable to provide a **detention facility** onsite, an **offsite detention facility** may be constructed if **all** of the following conditions are met:

1. The required **volume control storage** is provided onsite;
2. The **co-permittee** demonstrates that **site** limitations prevent the **development** from providing the full volume of the **detention facility** onsite;
3. The **parcel** area is less than ten (10) acres;
4. **Stormwater** detention is provided in accordance with the following hierarchy:
    a. Partially onsite in a **detention facility** with supplemental storage offsite in an **offsite detention facility** according to (b), (c) and (d) below;
    b. Offsite in an **offsite detention facility** where the **development** conveys the 100-year **storm event** to the **offsite detention facility**;
    c. Offsite in an **offsite detention facility** in a location that is upstream or hydrologically equivalent to the **development** in the same **subwatershed**; or
    d. Offsite in an **offsite detention facility** within the same **subwatershed**;

As described above, offsite detention is only an option for those **parcels** with areas between 3 acres and 10 acres (detention is not required for **parcels** ≤ 3 acres under the WMO), although **parcels** containing the offsite storage can be of unlimited size. When offsite detention is proposed, the **co-permittee** must demonstrate how the hierarchy outlined in (4) was followed for the **development**. An explanation of the navigation through the hierarchy should be included in the narrative in the **stormwater** submittal.

The **offsite detention facility** must be permitted and functional prior to the permitting of the **development** that is seeking detention credit. The WMO allows collaboration with either the Cook County Land Bank Authority or the South Suburban Land Bank and Development Authority to facilitate the trading of detention credits. Concurrence of the **District** or an **authorized municipality** should be sought prior to the start of the design of an **offsite detention facility.**

The design of **offsite detention facilities** must comply with the requirements of §500, §501, §502, §503, and §504 of the WMO, and shall not adversely impact upstream or downstream properties as described in §501.1.

Site Detention Exemptions

The WMO (§504.17) exempts certain types of **development** from the **site** detention requirements. Those **developments** that are tributary to Lake Michigan and provide water quality benefits will be exempt from providing **stormwater** detention. Specifically, the **development** must comply with the following conditions, as outlined in the WMO:

- The **development** discharges **stormwater** to a **storm sewer** tributary to Lake Michigan;
- The downstream receiving **storm sewer** has adequate capacity as determined by the governing **municipality**;
- The **development** complies with the **site** volume control requirements (WMO §503); and
- The **development** intercepts and treats all **stormwater runoff** onsite to improve water quality prior to discharge from the **development**.

The Lake Michigan water quality structure may be a stormwater treatment train of various BMPS, or may consist of a hydrodynamic separator with a settling or separation unit to remove sediments, hydrocarbons, and other pollutants commonly found in stormwater runoff. There are many manufactured hydrodynamic separation systems available, but the suitability of each type will vary based on the tributary area and stormwater runoff characteristics of each site.

The water quality benefits are satisfied by demonstrating the following stormwater runoff pollutant removal standards:

- 80% Total Suspended Solids (TSS) removal, with TSS defined by the Ok-110 particle size distribution (PSD)

- 80% of free floatable hydrocarbons
- 100% of floating trash and debris

The water quality structure must have the capacity to treat peak flowrate/runoff volume for the 2-year, 24-hour storm event (3.04 inches based on Bulletin 70), which is considered the “first flush” storm event. Additionally, the water quality structure must contain an overflow system that safely bypasses flows in excess of the 2-year, 24-hour design flowrate. A typical detail for a Lake Michigan water quality structure is provided in Appendix C.

ALLOWANCES FOR REDEVELOPMENT & DEVELOPMENT SUBJECT TO A LEGACY SEWERAGE SYSTEM PERMIT (§505)

The WMO separates the **stormwater** detention requirements for **redevelopments** into four categories: (1) the **parcel** that is to be redeveloped includes an **existing detention facility** permitted under the **Sewer Permit Ordinance (SPO)**, (2) the **parcel** was not tributary to an **existing detention facility** permitted under the **SPO**, but will be under proposed conditions, (3) the **parcel** to be redeveloped contains an **existing detention facility** that was permitted under a local **ordinance**, but not under the **SPO**; and (4) the **parcel** that is to be redeveloped does not contain a previously permitted **detention facility** and will not be tributary to one under proposed conditions. For those **redevelopments** that involve **parcels** that were never planned to be tributary to or do not contain an **existing detention facility** permitted under the **SPO** ((4) above), the **redevelopment** must comply with the **stormwater** management requirements described in §500 through §504 of the WMO. As detailed below, the WMO makes allowances for the **redevelopment** of **parcels** that involve previously permitted **existing detention facilities** and can meet certain conditions ((1), (2) and (3) above).

Parcels That Involve Existing Detention Facilities Permitted Under the SPO

For those **redevelopments** that involve an **existing detention facility** that was permitted under the **SPO** ((1) and (2) above), the WMO provides allowances if the following conditions are met:

1. Documentation is provided demonstrating the **existing detention facility** was designed, approved, and permitted under the **SPO**. At a minimum, the Schedule D form for the existing **development** must be provided.

2. The actual storage volume provided in the **existing detention facility** either meets or exceeds the permitted storage volume and is verified by a recent survey, signed and sealed by a **Professional Engineer** or **Professional Land Surveyor**.

3. The **redevelopment** meets the volume control requirements of the WMO.

4. The **redevelopment** provides adequate capacity to convey **stormwater runoff** to the **detention facility** for all storms up to the 100-year **storm event**.

If the **redevelopment** can meet these four conditions, the following detention allowances are granted by the WMO:

1. If the composite **runoff** coefficient (C) of the **redevelopment** does not exceed the C value of the existing **development** permitted under the **SPO** (found on the Schedule D form), additional **stormwater** detention volume is not required. When determining increases in C values for detention allowances, the C value should always be analyzed to two decimal places.

2. If the **redevelopment's** C value exceeds the C value of the existing **development** (found on the Schedule D form), the additional **stormwater** detention volume may be calculated using the modified Rational Method and **Bulletin 70** sectional rainfall depths. For these cases, the **allowable release rate** can be calculated using the following:

    a) If the **redevelopment** is within a permitted **parcel** intended to be tributary to an **existing detention facility**, the existing approved release rate and restrictor may be used.

    b) If the **redevelopment** is an area within a permitted **parcel**, which was never intended to be tributary to an **existing detention facility**, but will be tributary upon **redevelopment**, the original release rate for the basin must be recalculated using the proportion of original **tributary area** to new **tributary area**. The total new required detention volume is based on the pro-rated release rate and the existing restrictor may need to be replaced.

If the **redevelopment** only requires a marginal increase in detention storage compared to the provided storage volume in the existing facility, then no additional volume is required. The "marginal" increase in required volume is defined as less than 0.10 acre-feet or within 2% of the existing storage volume provided.

Any modifications to the **existing detention facility** that are necessary to provide the new required storage volume are considered to be **non-qualified development**, and this area of disturbance is not included in the calculation of required detention volume.

It should be noted that the WMO detention allowances are dedicated for **redevelopments** that utilize the existing detention located on the project site. If a **redevelopment** will completely reconfigure the site and relocate the **existing detention facility**, the **redevelopment** detention allowances will not apply to the **site**. Any **redevelopments** that completely reconfigures the existing developed portion of the site and also result in the removal or relocation of 75% or more of the existing detention volume must meet the full detention requirements of the WMO (§504).

A special case of **redevelopment** occurs when a **parcel** was originally permitted as part of a larger **development**. To avoid penalizing a redeveloper of a single **parcel** with providing additional detention volume for the entire **development**, a special methodology has been developed for these cases. Referring to Figure 5.26, Parcels 1-4 were originally permitted as a single **development**. If an applicant wants to redevelop only Parcel 3, the methodology for determining the additional storage volume required is provided below.

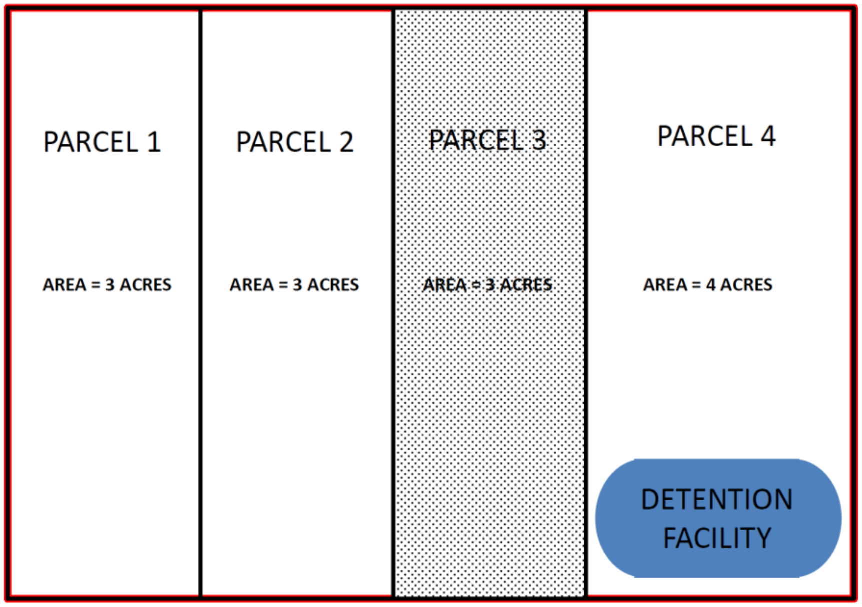


Figure 5.26. Redevelopments of Parcels Part of a Larger Development

The first step is to calculate the proposed C value of the entire **development**, C_{REDEV}, which includes the redeveloped **parcel**. If C_{REDEV} is greater than the permitted C value for the original **development**, C_{PERMIT}, additional storage is required.

The second step is to determine the amount of permitted volume that was allocated just for **Parcel** 3, which is calculated by the following:

$$\frac{V_{PERMIT}}{A_{PERMIT}} \quad x \quad A_{PARCEL\ 3} \quad = \quad V_{PERMIT,3} \text{ (Permitted Storage Volume Allocated to Parcel 3)}$$

Using the modified Rational Method with **Bulletin 70** rainfall depths, the third step is to determine the pro-rated required detention volume for the three-acre **parcel**.

$$\frac{V_{REDEV}}{A_{PERMIT}} \quad x \quad A_{PARCEL\ 3} \quad = \quad V_{REDEV,3} \text{ (Required Storage Volume for Parcel 3)}$$

The last step is to determine the additional storage volume that must be provided for the **redevelopment** of Parcel 3, which can be calculated by:

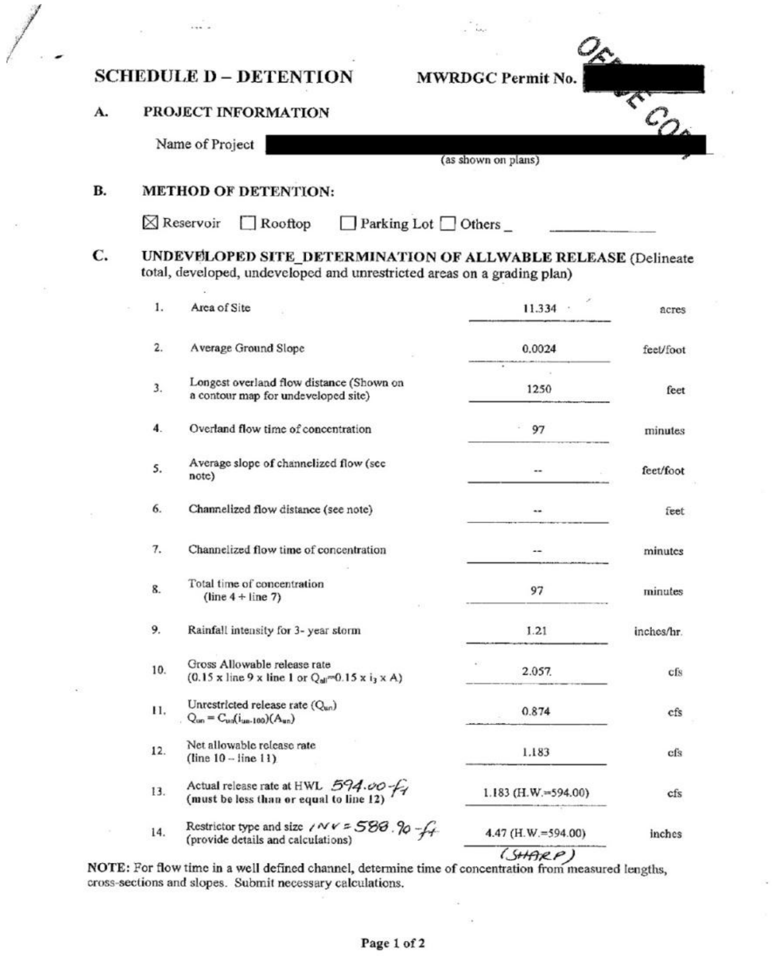
Additional storage volume required = $V_{REDEV,3} - V_{PERMIT,3}$

If the additional storage volume required is less than or equal to 0.10 ac-ft or within 2% of the original volume allocated for the **parcel**, then additional storage volume is not required. Any **volume control storage** that is provided as part of the **redevelopment** of the **parcel** is credited toward the required additional volume.

Example 5.10 – Detention Requirements for Redevelopment

An existing 11.3-acre industrial area is to be redeveloped into a shopping mall. The original **development** contains a **detention facility** that was permitted under the **Sewer Permit Ordinance (SPO)**. The proposed **development** has a C value of 0.90, with 8.0 acres of **impervious area**. It is assumed that the proposed **development** will provide the 1 inch of **volume control storage**. Determine the required detention volume for the **site**.

Step 1: Obtain the Schedule D form for the original **development** to determine the composite **runoff** coefficient (C value) and required detention volume that was permitted. As shown on Page 2 of the Schedule D form, the permitted **development** in this example has a C value of 0.88 and a required detention volume of 3.07 acre-feet.



SCHEDULE D – DETENTION **MWRDGC Permit No.** [redacted]

A. PROJECT INFORMATION

Name of Project [redacted]
(as shown on plans)

B. METHOD OF DETENTION:

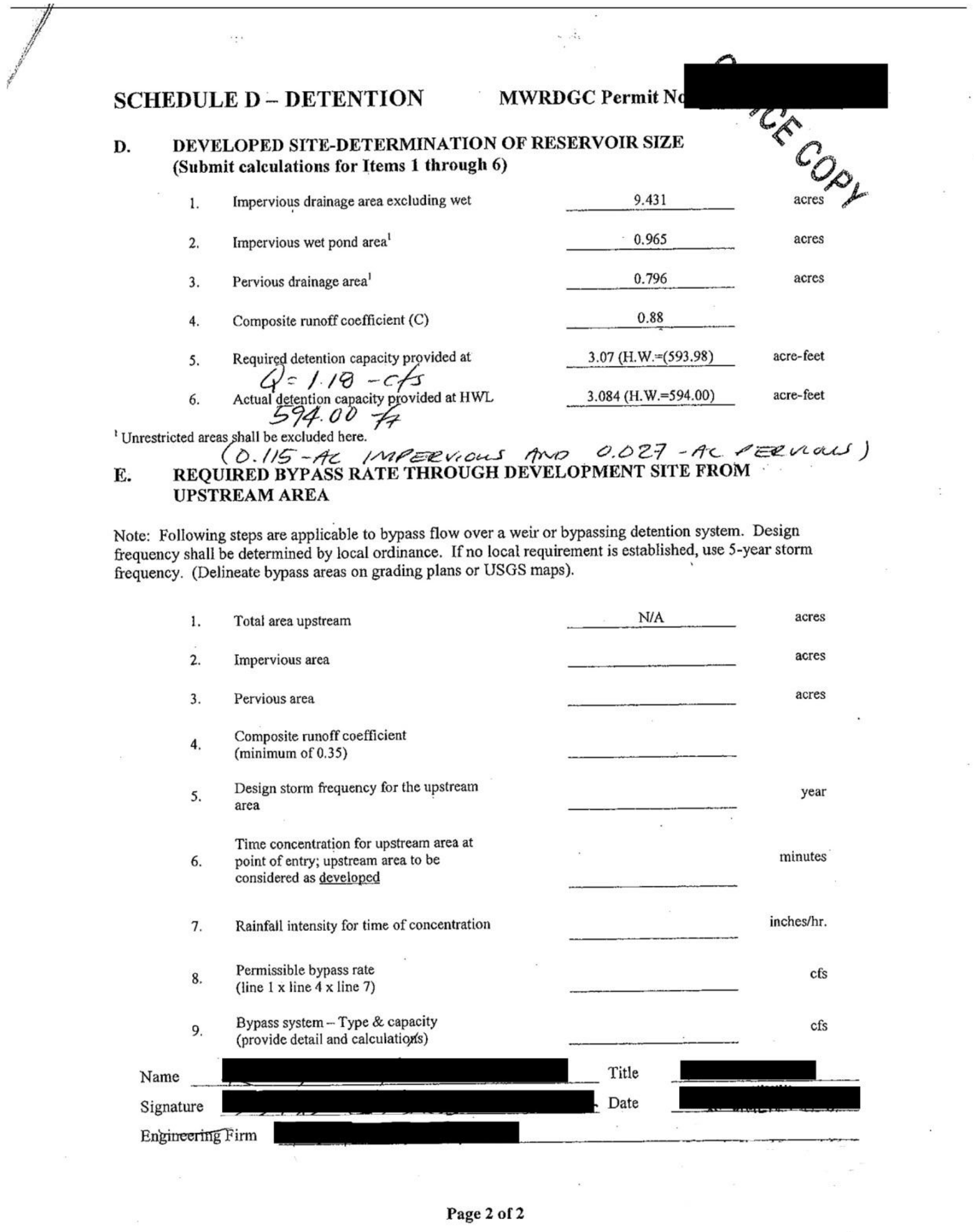
☒ Reservoir ☐ Rooftop ☐ Parking Lot ☐ Others _

C. UNDEVELOPED SITE_DETERMINATION OF ALLWABLE RELEASE (Delineate total, developed, undeveloped and unrestricted areas on a grading plan)

1.	Area of Site	11.334	acres
2.	Average Ground Slope	0.0024	feet/foot
3.	Longest overland flow distance (Shown on a contour map for undeveloped site)	1250	feet
4.	Overland flow time of concentration	97	minutes
5.	Average slope of channelized flow (see note)	--	feet/foot
6.	Channelized flow distance (see note)	--	feet
7.	Channelized flow time of concentration	--	minutes
8.	Total time of concentration (line 4 + line 7)	97	minutes
9.	Rainfall intensity for 3- year storm	1.21	inches/hr.
10.	Gross Allowable release rate (0.15 x line 9 x line 1 or Q_{all}=0.15 x i_3 x A)	2.057	cfs
11.	Unrestricted release rate (Q_{un}) $Q_{un} = C_{un}(i_{un-100})(A_{un})$	0.874	cfs
12.	Net allowable release rate (line 10 – line 11)	1.183	cfs
13.	Actual release rate at HWL 594.00-ft **(must be less than or equal to line 12)**	1.183 (H.W.=594.00)	cfs
14.	Restrictor type and size INV = 588.90-ft (provide details and calculations)	4.47 (H.W.=594.00) (SHARP)	inches

NOTE: For flow time in a well defined channel, determine time of concentration from measured lengths, cross-sections and slopes. Submit necessary calculations.

Page 1 of 2



SCHEDULE D – DETENTION **MWRDGC Permit No** [redacted]

CE COPY

D. DEVELOPED SITE-DETERMINATION OF RESERVOIR SIZE
(Submit calculations for Items 1 through 6)

1. Impervious drainage area excluding wet 9.431 acres
2. Impervious wet pond area[1] 0.965 acres
3. Pervious drainage area[1] 0.796 acres
4. Composite runoff coefficient (C) 0.88
5. Required detention capacity provided at Q= 1.18 -cfs 3.07 (H.W.=(593.98) acre-feet
6. Actual detention capacity provided at HWL 594.00 ft 3.084 (H.W.=594.00) acre-feet

[1] Unrestricted areas shall be excluded here. (0.115-AC IMPERVIOUS AND 0.027-AC PERVIOUS)

E. REQUIRED BYPASS RATE THROUGH DEVELOPMENT SITE FROM UPSTREAM AREA

Note: Following steps are applicable to bypass flow over a weir or bypassing detention system. Design frequency shall be determined by local ordinance. If no local requirement is established, use 5-year storm frequency. (Delineate bypass areas on grading plans or USGS maps).

1. Total area upstream N/A acres
2. Impervious area ____ acres
3. Pervious area ____ acres
4. Composite runoff coefficient (minimum of 0.35) ____
5. Design storm frequency for the upstream area ____ year
6. Time concentration for upstream area at point of entry; upstream area to be considered as developed ____ minutes
7. Rainfall intensity for time of concentration ____ inches/hr.
8. Permissible bypass rate (line 1 x line 4 x line 7) ____ cfs
9. Bypass system – Type & capacity (provide detail and calculations) ____ cfs

Name [redacted] Title [redacted]
Signature [redacted] Date [redacted]
Engineering Firm [redacted]

Page 2 of 2

Step 2: Determine if the **redevelopment** can meet the conditions for detention allowances provided in Section §505.3 of the WMO. The four conditions are:

1. Design of the **existing detention facility** is documented and approved under an existing **sewerage system permit**;

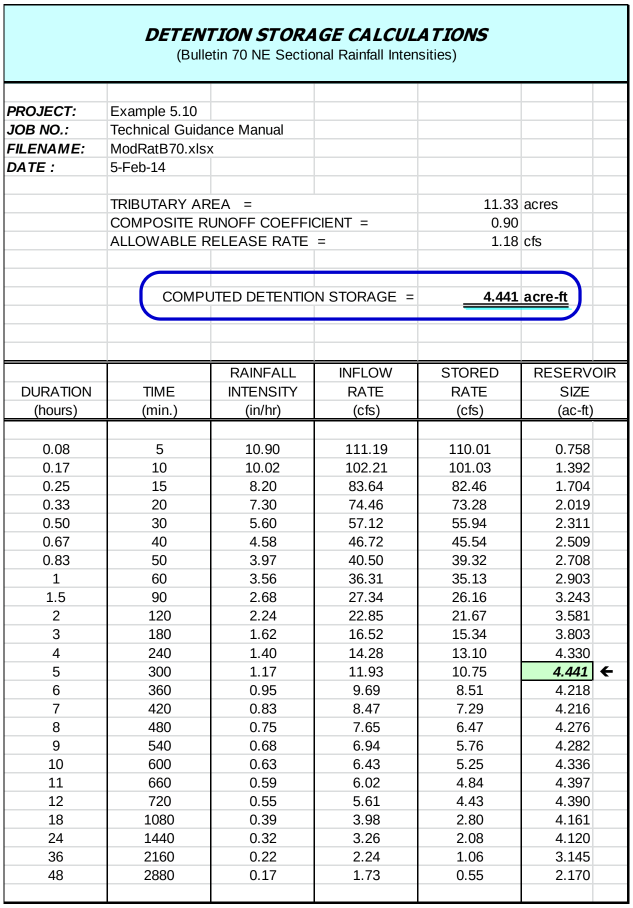
2. The actual storage volume is verified to meet the required permitted volume (3.07 acre-feet in this example) by a survey;

3. The **redevelopment** will meet the volume control requirements of the WMO; and

4. The **redevelopment** provides adequate conveyance to convey the 100-year peak flowrates to the **detention facility**.

Step 3: Assuming the **redevelopment** can meet the four conditions outlined above, calculate the **redevelopment's** C value. Using the values provided in Table 5-2 of the **TGM**, the redeveloped C value is 0.90. Because the redeveloped C value (0.90) is greater than existing (0.88), additional detention storage is required. If the redeveloped C value were to match the permitted C value of 0.88, no additional storage volume would be required.

Step 4: Determine the required storage volume for the **redevelopment** using the modified Rational Method and **Bulletin 70** rainfall depths. Since the **existing detention facility** was previously permitted under the **SPO**, the original release rate and restrictor can be used. From Page 1 of the Schedule D form, the **allowable release rate** is 1.183 cfs. (Note that the release rate of 1.183 cfs calculated for the original **development** included unrestricted releases; if the **redevelopment** causes additional unrestricted releases, or if the applicant wants to use a larger release rate because unrestricted areas have been reduced, a modification to the outlet control **structure** would be required.)



DETENTION STORAGE CALCULATIONS
(Bulletin 70 NE Sectional Rainfall Intensities)

PROJECT: Example 5.10
JOB NO.: Technical Guidance Manual
FILENAME: ModRatB70.xlsx
DATE : 5-Feb-14

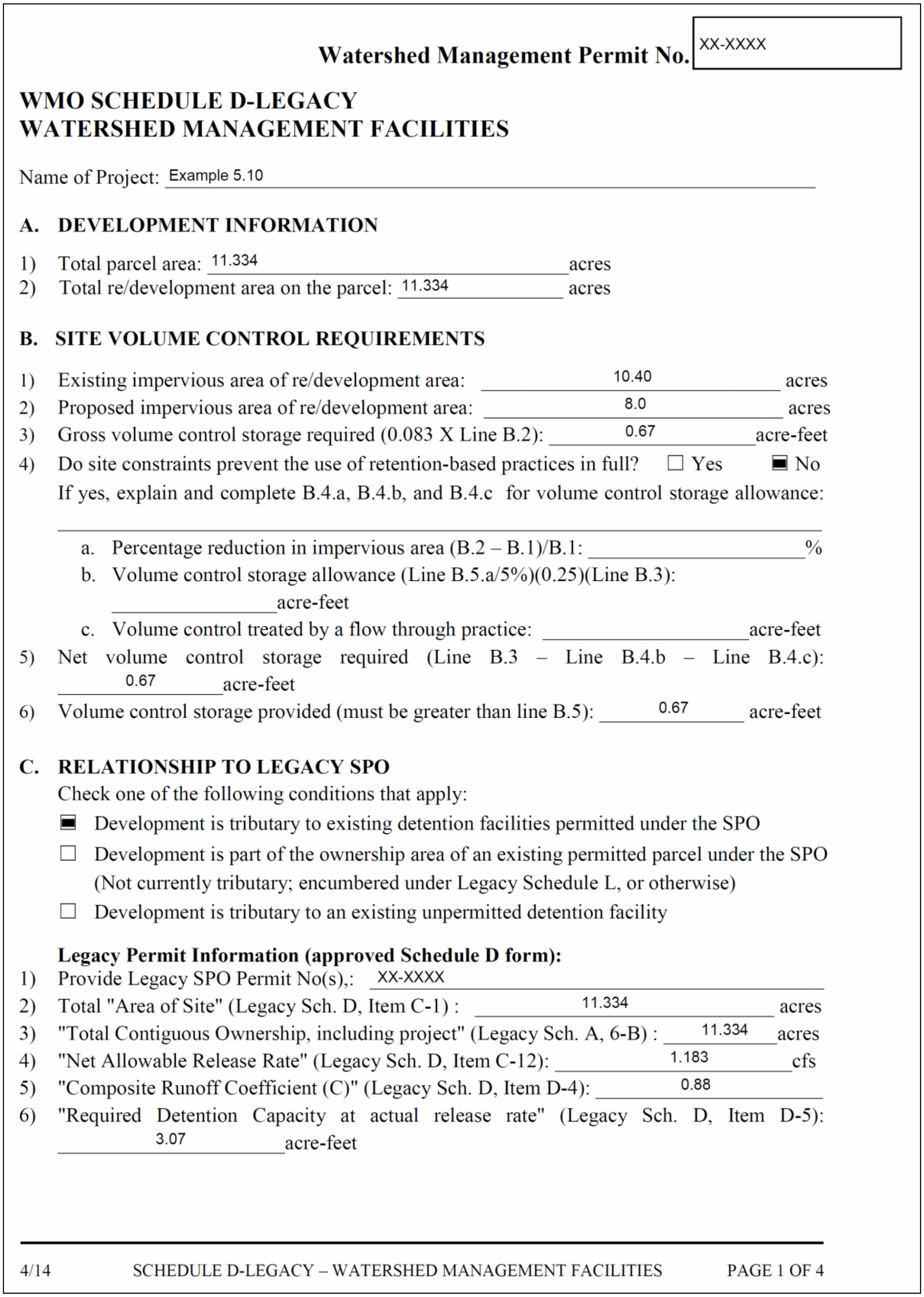
TRIBUTARY AREA = 11.33 acres
COMPOSITE RUNOFF COEFFICIENT = 0.90
ALLOWABLE RELEASE RATE = 1.18 cfs

COMPUTED DETENTION STORAGE = **4.441 acre-ft**

DURATION (hours)	TIME (min.)	RAINFALL INTENSITY (in/hr)	INFLOW RATE (cfs)	STORED RATE (cfs)	RESERVOIR SIZE (ac-ft)
0.08	5	10.90	111.19	110.01	0.758
0.17	10	10.02	102.21	101.03	1.392
0.25	15	8.20	83.64	82.46	1.704
0.33	20	7.30	74.46	73.28	2.019
0.50	30	5.60	57.12	55.94	2.311
0.67	40	4.58	46.72	45.54	2.509
0.83	50	3.97	40.50	39.32	2.708
1	60	3.56	36.31	35.13	2.903
1.5	90	2.68	27.34	26.16	3.243
2	120	2.24	22.85	21.67	3.581
3	180	1.62	16.52	15.34	3.803
4	240	1.40	14.28	13.10	4.330
5	300	1.17	11.93	10.75	***4.441*** ←
6	360	0.95	9.69	8.51	4.218
7	420	0.83	8.47	7.29	4.216
8	480	0.75	7.65	6.47	4.276
9	540	0.68	6.94	5.76	4.282
10	600	0.63	6.43	5.25	4.336
11	660	0.59	6.02	4.84	4.397
12	720	0.55	5.61	4.43	4.390
18	1080	0.39	3.98	2.80	4.161
24	1440	0.32	3.26	2.08	4.120
36	2160	0.22	2.24	1.06	3.145
48	2880	0.17	1.73	0.55	2.170

Using the modified Rational Method and **Bulletin 70** rainfall depths, the required detention volume for the **redevelopment** is 4.44 acre-feet. Since the provided detention storage for the original **development** was 3.07 acre-feet (and was verified by a survey), the additional storage that is required is 1.37 acre-feet. Any **volume control storage** that is provided is credited toward the required storage volume.

It should be noted that while the WMO provides detention allowances for **redevelopments**, all other requirements of the WMO may still be applicable. The overflow weir for the **detention facility**, for example, may need to be retrofitted to meet the design requirements of the WMO if there is a known drainage problem associated with the **parcel**.



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D-LEGACY
WATERSHED MANAGEMENT FACILITIES

Name of Project: Example 5.10

A. DEVELOPMENT INFORMATION

1) Total parcel area: 11.334 acres
2) Total re/development area on the parcel: 11.334 acres

B. SITE VOLUME CONTROL REQUIREMENTS

1) Existing impervious area of re/development area: 10.40 acres
2) Proposed impervious area of re/development area: 8.0 acres
3) Gross volume control storage required (0.083 X Line B.2): 0.67 acre-feet
4) Do site constraints prevent the use of retention-based practices in full? ☐ Yes ☒ No
If yes, explain and complete B.4.a, B.4.b, and B.4.c for volume control storage allowance:
    a. Percentage reduction in impervious area (B.2 – B.1)/B.1: %
    b. Volume control storage allowance (Line B.5.a/5%)(0.25)(Line B.3): acre-feet
    c. Volume control treated by a flow through practice: acre-feet
5) Net volume control storage required (Line B.3 – Line B.4.b – Line B.4.c): 0.67 acre-feet
6) Volume control storage provided (must be greater than line B.5): 0.67 acre-feet

C. RELATIONSHIP TO LEGACY SPO

Check one of the following conditions that apply:

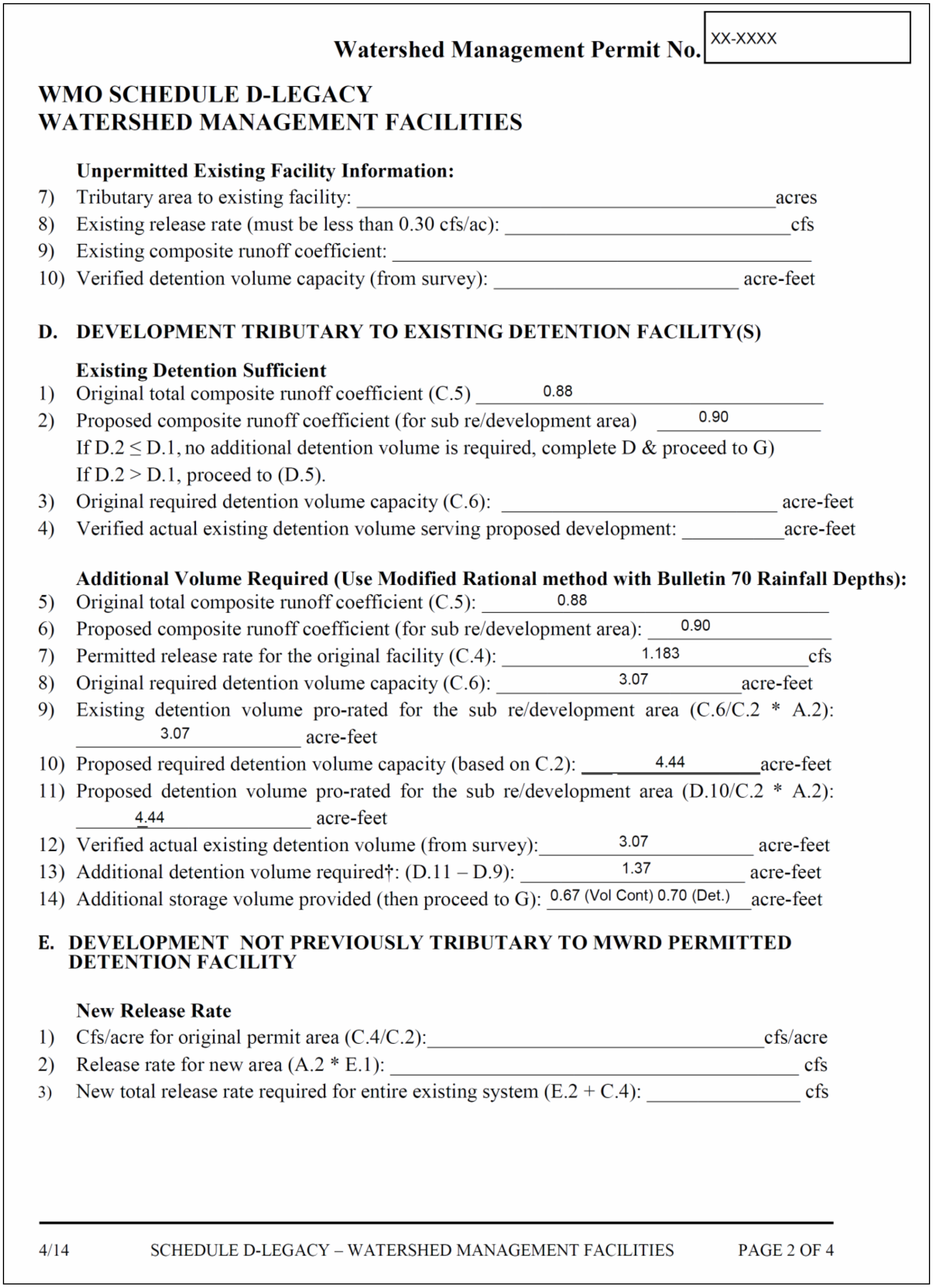
☒ Development is tributary to existing detention facilities permitted under the SPO
☐ Development is part of the ownership area of an existing permitted parcel under the SPO (Not currently tributary; encumbered under Legacy Schedule L, or otherwise)
☐ Development is tributary to an existing unpermitted detention facility

Legacy Permit Information (approved Schedule D form):

1) Provide Legacy SPO Permit No(s),: XX-XXXX
2) Total "Area of Site" (Legacy Sch. D, Item C-1) : 11.334 acres
3) "Total Contiguous Ownership, including project" (Legacy Sch. A, 6-B) : 11.334 acres
4) "Net Allowable Release Rate" (Legacy Sch. D, Item C-12): 1.183 cfs
5) "Composite Runoff Coefficient (C)" (Legacy Sch. D, Item D-4): 0.88
6) "Required Detention Capacity at actual release rate" (Legacy Sch. D, Item D-5): 3.07 acre-feet

4/14 SCHEDULE D-LEGACY – WATERSHED MANAGEMENT FACILITIES PAGE 1 OF 4

Schedule D-Legacy for Example 5.10 (Page 1 of 4)



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D-LEGACY
WATERSHED MANAGEMENT FACILITIES

Unpermitted Existing Facility Information:

7) Tributary area to existing facility: ______________________ acres
8) Existing release rate (must be less than 0.30 cfs/ac): ______________ cfs
9) Existing composite runoff coefficient: ______________________
10) Verified detention volume capacity (from survey): ____________ acre-feet

D. DEVELOPMENT TRIBUTARY TO EXISTING DETENTION FACILITY(S)

Existing Detention Sufficient

1) Original total composite runoff coefficient (C.5) 0.88
2) Proposed composite runoff coefficient (for sub re/development area) 0.90
If D.2 ≤ D.1, no additional detention volume is required, complete D & proceed to G)
If D.2 > D.1, proceed to (D.5).
3) Original required detention volume capacity (C.6): ______________ acre-feet
4) Verified actual existing detention volume serving proposed development: __________ acre-feet

Additional Volume Required (Use Modified Rational method with Bulletin 70 Rainfall Depths):

5) Original total composite runoff coefficient (C.5): 0.88
6) Proposed composite runoff coefficient (for sub re/development area): 0.90
7) Permitted release rate for the original facility (C.4): 1.183 cfs
8) Original required detention volume capacity (C.6): 3.07 acre-feet
9) Existing detention volume pro-rated for the sub re/development area (C.6/C.2 * A.2): 3.07 acre-feet
10) Proposed required detention volume capacity (based on C.2): 4.44 acre-feet
11) Proposed detention volume pro-rated for the sub re/development area (D.10/C.2 * A.2): 4.44 acre-feet
12) Verified actual existing detention volume (from survey): 3.07 acre-feet
13) Additional detention volume required†: (D.11 – D.9): 1.37 acre-feet
14) Additional storage volume provided (then proceed to G): 0.67 (Vol Cont) 0.70 (Det.) acre-feet

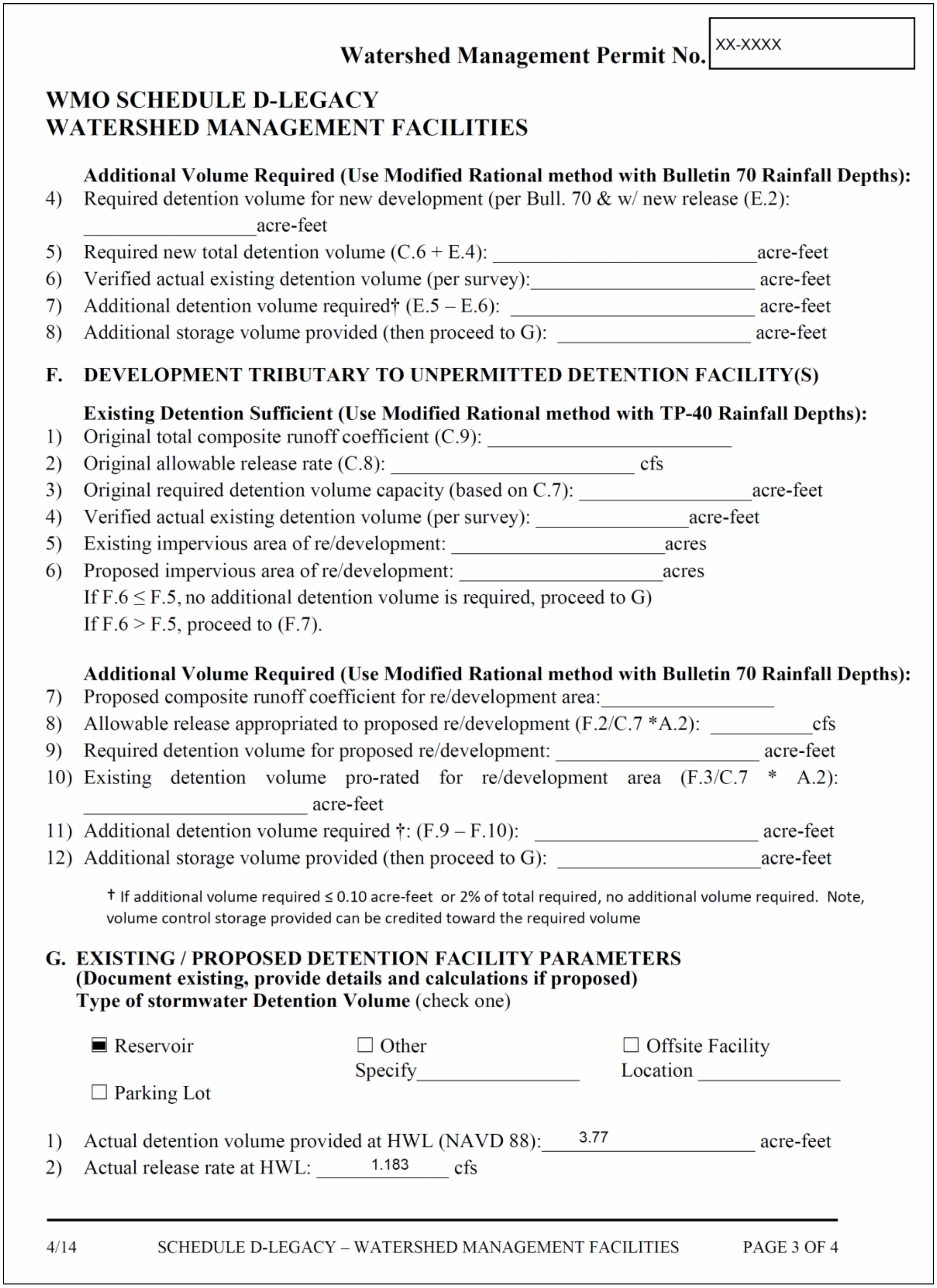
E. DEVELOPMENT NOT PREVIOUSLY TRIBUTARY TO MWRD PERMITTED DETENTION FACILITY

New Release Rate

1) Cfs/acre for original permit area (C.4/C.2): ______________________ cfs/acre
2) Release rate for new area (A.2 * E.1): ______________________ cfs
3) New total release rate required for entire existing system (E.2 + C.4): __________ cfs

4/14 SCHEDULE D-LEGACY – WATERSHED MANAGEMENT FACILITIES PAGE 2 OF 4

Schedule D-Legacy for Example 5.10 (Page 2 of 4)



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D-LEGACY
WATERSHED MANAGEMENT FACILITIES

Additional Volume Required (Use Modified Rational method with Bulletin 70 Rainfall Depths):

4) Required detention volume for new development (per Bull. 70 & w/ new release (E.2): ____________ acre-feet
5) Required new total detention volume (C.6 + E.4): ____________ acre-feet
6) Verified actual existing detention volume (per survey): ____________ acre-feet
7) Additional detention volume required† (E.5 – E.6): ____________ acre-feet
8) Additional storage volume provided (then proceed to G): ____________ acre-feet

F. DEVELOPMENT TRIBUTARY TO UNPERMITTED DETENTION FACILITY(S)

Existing Detention Sufficient (Use Modified Rational method with TP-40 Rainfall Depths):

1) Original total composite runoff coefficient (C.9): ____________
2) Original allowable release rate (C.8): ____________ cfs
3) Original required detention volume capacity (based on C.7): ____________ acre-feet
4) Verified actual existing detention volume (per survey): ____________ acre-feet
5) Existing impervious area of re/development: ____________ acres
6) Proposed impervious area of re/development: ____________ acres

If F.6 ≤ F.5, no additional detention volume is required, proceed to G)
If F.6 > F.5, proceed to (F.7).

Additional Volume Required (Use Modified Rational method with Bulletin 70 Rainfall Depths):

7) Proposed composite runoff coefficient for re/development area: ____________
8) Allowable release appropriated to proposed re/development (F.2/C.7 *A.2): ____________ cfs
9) Required detention volume for proposed re/development: ____________ acre-feet
10) Existing detention volume pro-rated for re/development area (F.3/C.7 * A.2): ____________ acre-feet
11) Additional detention volume required †: (F.9 – F.10): ____________ acre-feet
12) Additional storage volume provided (then proceed to G): ____________ acre-feet

† If additional volume required ≤ 0.10 acre-feet or 2% of total required, no additional volume required. Note, volume control storage provided can be credited toward the required volume

G. EXISTING / PROPOSED DETENTION FACILITY PARAMETERS
(Document existing, provide details and calculations if proposed)
Type of stormwater Detention Volume (check one)

■ Reservoir

☐ Parking Lot

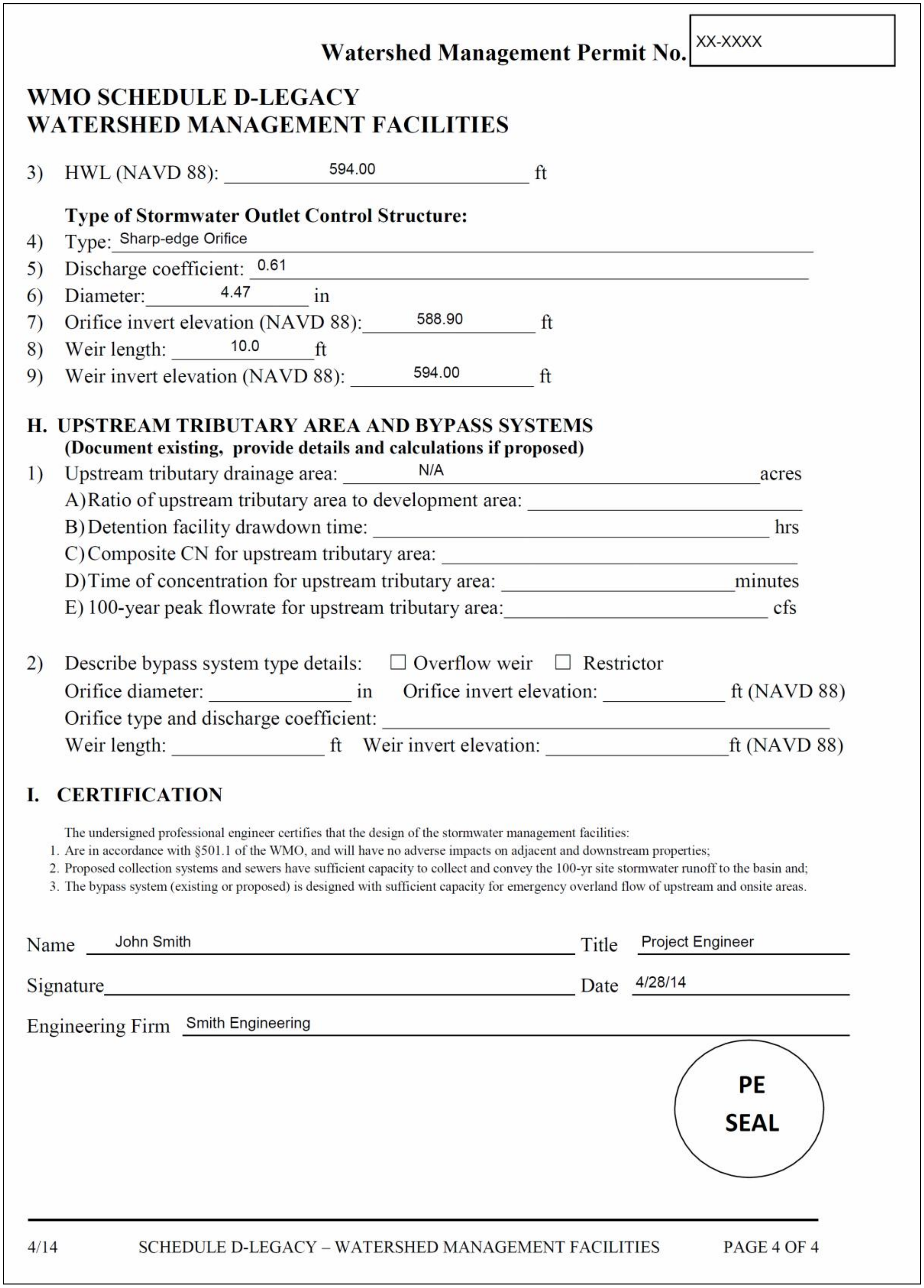
☐ Other
Specify ____________

☐ Offsite Facility
Location ____________

1) Actual detention volume provided at HWL (NAVD 88): 3.77 acre-feet
2) Actual release rate at HWL: 1.183 cfs

4/14 SCHEDULE D-LEGACY – WATERSHED MANAGEMENT FACILITIES PAGE 3 OF 4

Schedule D-Legacy for Example 5.10 (Page 3 of 4)



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D-LEGACY
WATERSHED MANAGEMENT FACILITIES

3) HWL (NAVD 88): 594.00 ft

Type of Stormwater Outlet Control Structure:

4) Type: Sharp-edge Orifice
5) Discharge coefficient: 0.61
6) Diameter: 4.47 in
7) Orifice invert elevation (NAVD 88): 588.90 ft
8) Weir length: 10.0 ft
9) Weir invert elevation (NAVD 88): 594.00 ft

H. UPSTREAM TRIBUTARY AREA AND BYPASS SYSTEMS

(Document existing, provide details and calculations if proposed)

1) Upstream tributary drainage area: N/A acres
   A) Ratio of upstream tributary area to development area: ______
   B) Detention facility drawdown time: ______ hrs
   C) Composite CN for upstream tributary area: ______
   D) Time of concentration for upstream tributary area: ______ minutes
   E) 100-year peak flowrate for upstream tributary area: ______ cfs

2) Describe bypass system type details: ☐ Overflow weir ☐ Restrictor
   Orifice diameter: ______ in Orifice invert elevation: ______ ft (NAVD 88)
   Orifice type and discharge coefficient: ______
   Weir length: ______ ft Weir invert elevation: ______ ft (NAVD 88)

I. CERTIFICATION

The undersigned professional engineer certifies that the design of the stormwater management facilities:

1. Are in accordance with §501.1 of the WMO, and will have no adverse impacts on adjacent and downstream properties;
2. Proposed collection systems and sewers have sufficient capacity to collect and convey the 100-yr site stormwater runoff to the basin and;
3. The bypass system (existing or proposed) is designed with sufficient capacity for emergency overland flow of upstream and onsite areas.

Name John Smith Title Project Engineer
Signature ______ Date 4/28/14
Engineering Firm Smith Engineering

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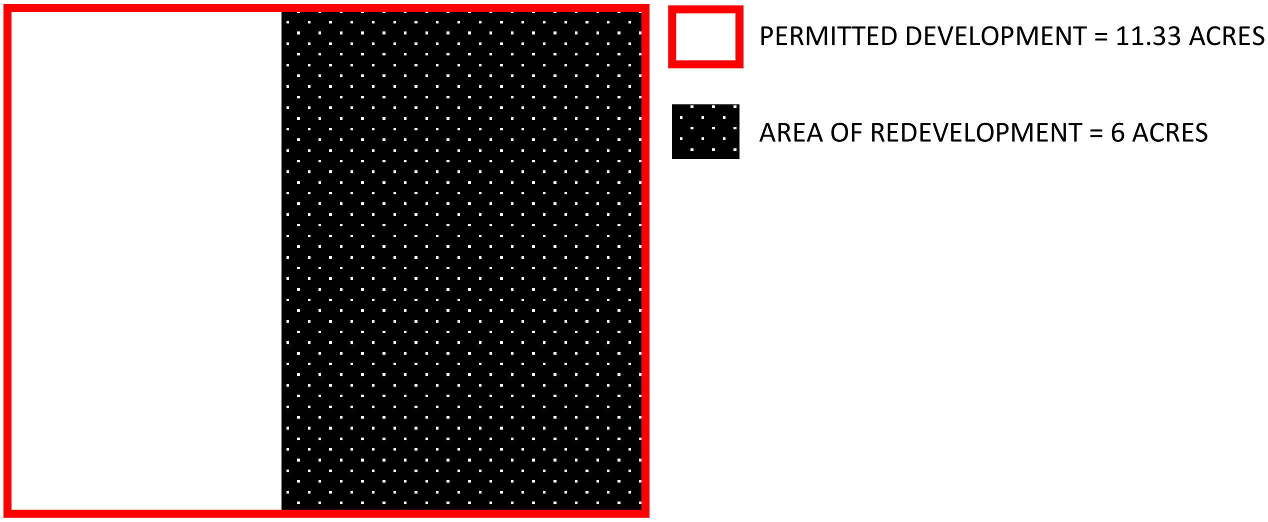
4/14 SCHEDULE D-LEGACY – WATERSHED MANAGEMENT FACILITIES PAGE 4 OF 4

Schedule D-Legacy for Example 5.10 (Page 4 of 4)

Example 5.11 –Redevelopment of a Parcel Permitted as Part of Larger Development

For the **site** in Example 5.10, determine the required detention volume if only a 6-acre portion of the **site** is to be redeveloped. Of the 6 acres that is to be redeveloped, 5.4 acres are impervious.



Step 1: Calculate the proposed C value of entire **development**, C_{REDEV}, which includes redeveloped **parcel**. If C_{REDEV} is greater than the permitted C value for the **development**, C_{PERMIT}, additional storage volume is required.

C_{PERMIT} = 0.88 (from Schedule D form for original **development**)

C_{REDEV} = 0.89 (entire **development** including redeveloped 6-acre **parcel**)

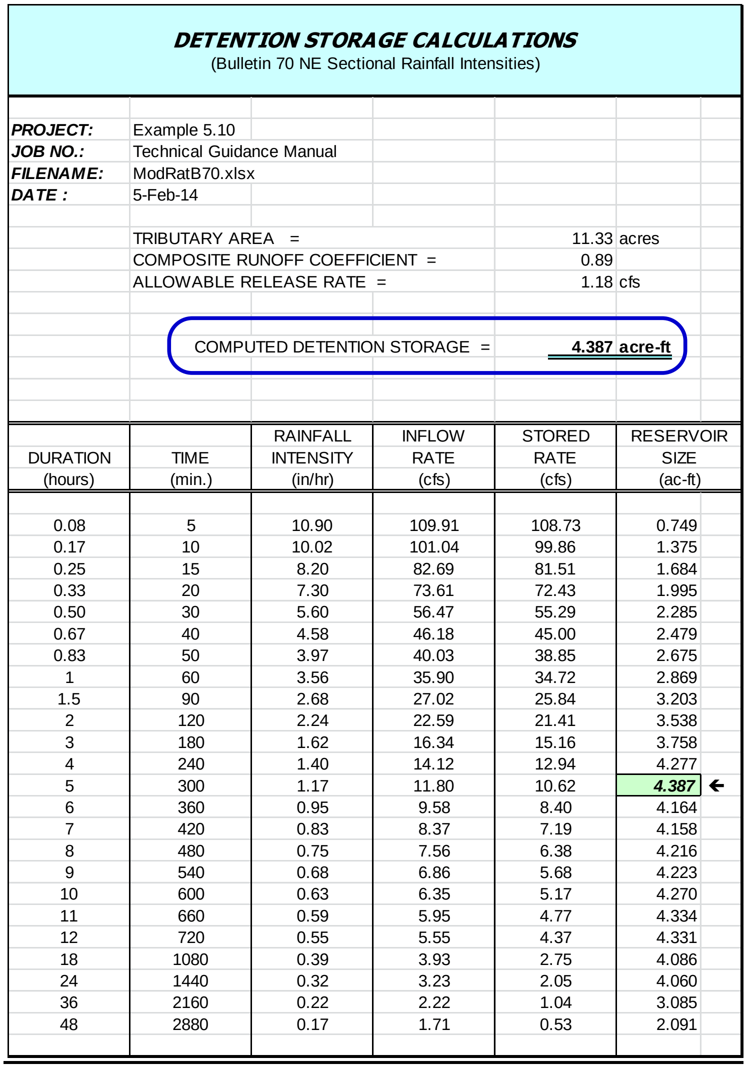
Since C_{REDEV} is greater than C_{PERMIT}, additional storage volume is required.

Step 2: Determine the pro-rated permitted detention volume for the 6-acre **parcel**. From Schedule D for the original **development**, 3.07 acre-feet is required for the 11.33-acre **development**. The pro-rated detention volume for the 6-acre **parcel** is calculated as such:

$$\frac{3.07 \text{ acre-feet}}{11.33 \text{ acres}} \quad x \quad 6 \text{ acres} \quad = \quad 1.63 \text{ acre-feet}$$

Step 3: Using the modified Rational Method with **Bulletin 70** rainfall depths, determine the pro-rated required detention volume for the 6-acre **parcel**.

$$\frac{4.39 \text{ acre-feet}}{11.33 \text{ acres}} \quad x \quad 6 \text{ acres} \quad = \quad 2.32 \text{ acre-feet}$$



DETENTION STORAGE CALCULATIONS

(Bulletin 70 NE Sectional Rainfall Intensities)

PROJECT:	Example 5.10
JOB NO.:	Technical Guidance Manual
FILENAME:	ModRatB70.xlsx
DATE :	5-Feb-14

TRIBUTARY AREA = 11.33 acres

COMPOSITE RUNOFF COEFFICIENT = 0.89

ALLOWABLE RELEASE RATE = 1.18 cfs

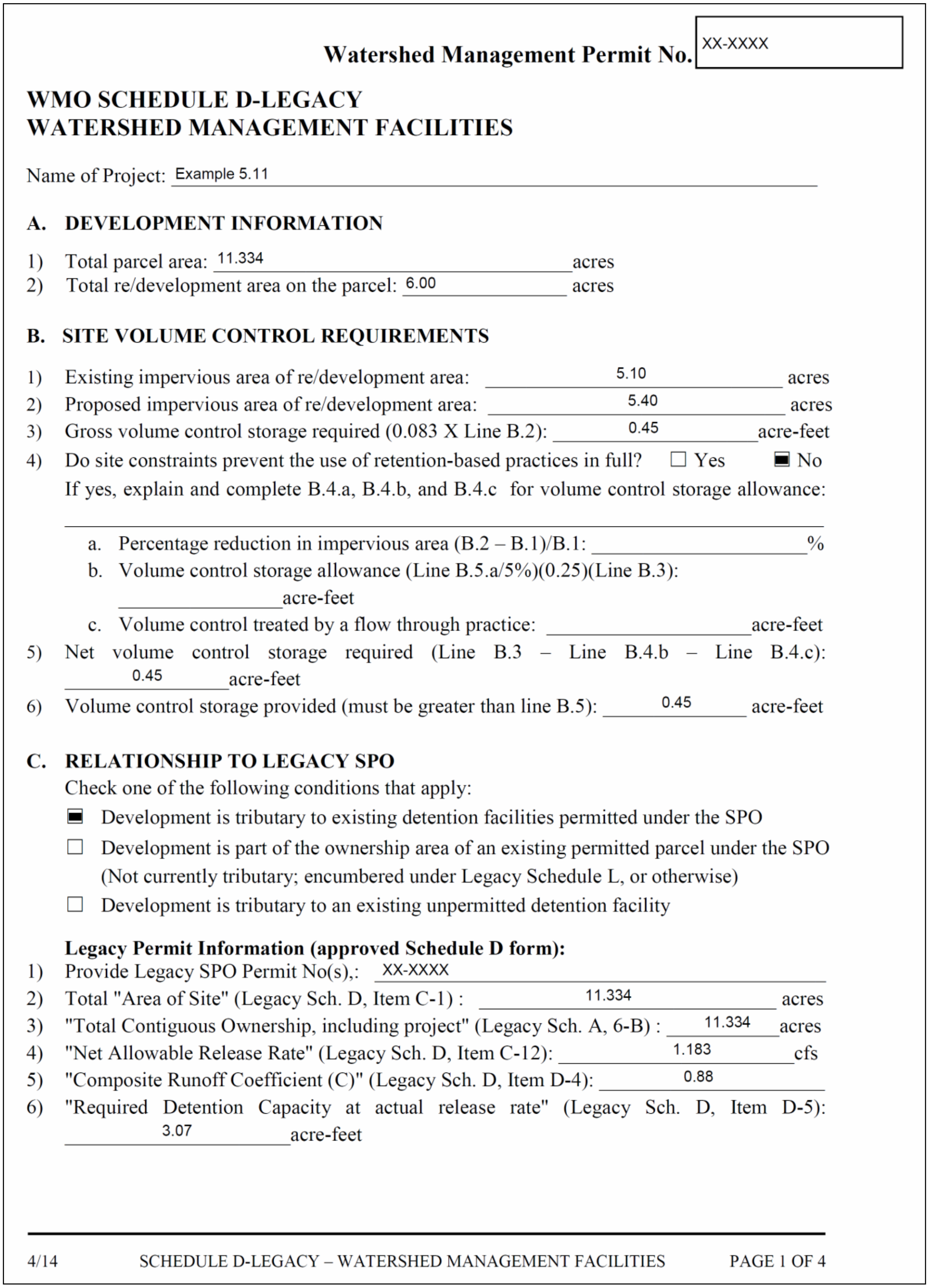
COMPUTED DETENTION STORAGE = **4.387 acre-ft**

DURATION (hours)	TIME (min.)	RAINFALL INTENSITY (in/hr)	INFLOW RATE (cfs)	STORED RATE (cfs)	RESERVOIR SIZE (ac-ft)
0.08	5	10.90	109.91	108.73	0.749
0.17	10	10.02	101.04	99.86	1.375
0.25	15	8.20	82.69	81.51	1.684
0.33	20	7.30	73.61	72.43	1.995
0.50	30	5.60	56.47	55.29	2.285
0.67	40	4.58	46.18	45.00	2.479
0.83	50	3.97	40.03	38.85	2.675
1	60	3.56	35.90	34.72	2.869
1.5	90	2.68	27.02	25.84	3.203
2	120	2.24	22.59	21.41	3.538
3	180	1.62	16.34	15.16	3.758
4	240	1.40	14.12	12.94	4.277
5	300	1.17	11.80	10.62	***4.387*** ←
6	360	0.95	9.58	8.40	4.164
7	420	0.83	8.37	7.19	4.158
8	480	0.75	7.56	6.38	4.216
9	540	0.68	6.86	5.68	4.223
10	600	0.63	6.35	5.17	4.270
11	660	0.59	5.95	4.77	4.334
12	720	0.55	5.55	4.37	4.331
18	1080	0.39	3.93	2.75	4.086
24	1440	0.32	3.23	2.05	4.060
36	2160	0.22	2.22	1.04	3.085
48	2880	0.17	1.71	0.53	2.091

<u>Step 4</u>: Determine the additional storage volume required.

Additional storage volume required = 2.32 ac-ft – 1.63 ac-ft = 0.69 acre-feet

Since additional storage volume required is greater than 0.10 acre-feet and is not within 2% of the existing volume allocated for the **parcel**, the additional storage volume must be provided. Assuming 0.45 acre-feet of **volume control storage** was provided as part of the **redevelopment** of the **parcel** (1 inch over the **impervious area**), the net volume that is required is 0.24 ac-ft.



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D-LEGACY
WATERSHED MANAGEMENT FACILITIES

Name of Project: Example 5.11

A. DEVELOPMENT INFORMATION

1) Total parcel area: 11.334 acres
2) Total re/development area on the parcel: 6.00 acres

B. SITE VOLUME CONTROL REQUIREMENTS

1) Existing impervious area of re/development area: 5.10 acres
2) Proposed impervious area of re/development area: 5.40 acres
3) Gross volume control storage required (0.083 X Line B.2): 0.45 acre-feet
4) Do site constraints prevent the use of retention-based practices in full? ☐ Yes ■ No
If yes, explain and complete B.4.a, B.4.b, and B.4.c for volume control storage allowance:
a. Percentage reduction in impervious area (B.2 – B.1)/B.1: ______ %
b. Volume control storage allowance (Line B.5.a/5%)(0.25)(Line B.3): ______ acre-feet
c. Volume control treated by a flow through practice: ______ acre-feet
5) Net volume control storage required (Line B.3 – Line B.4.b – Line B.4.c): 0.45 acre-feet
6) Volume control storage provided (must be greater than line B.5): 0.45 acre-feet

C. RELATIONSHIP TO LEGACY SPO

Check one of the following conditions that apply:

■ Development is tributary to existing detention facilities permitted under the SPO

☐ Development is part of the ownership area of an existing permitted parcel under the SPO (Not currently tributary; encumbered under Legacy Schedule L, or otherwise)

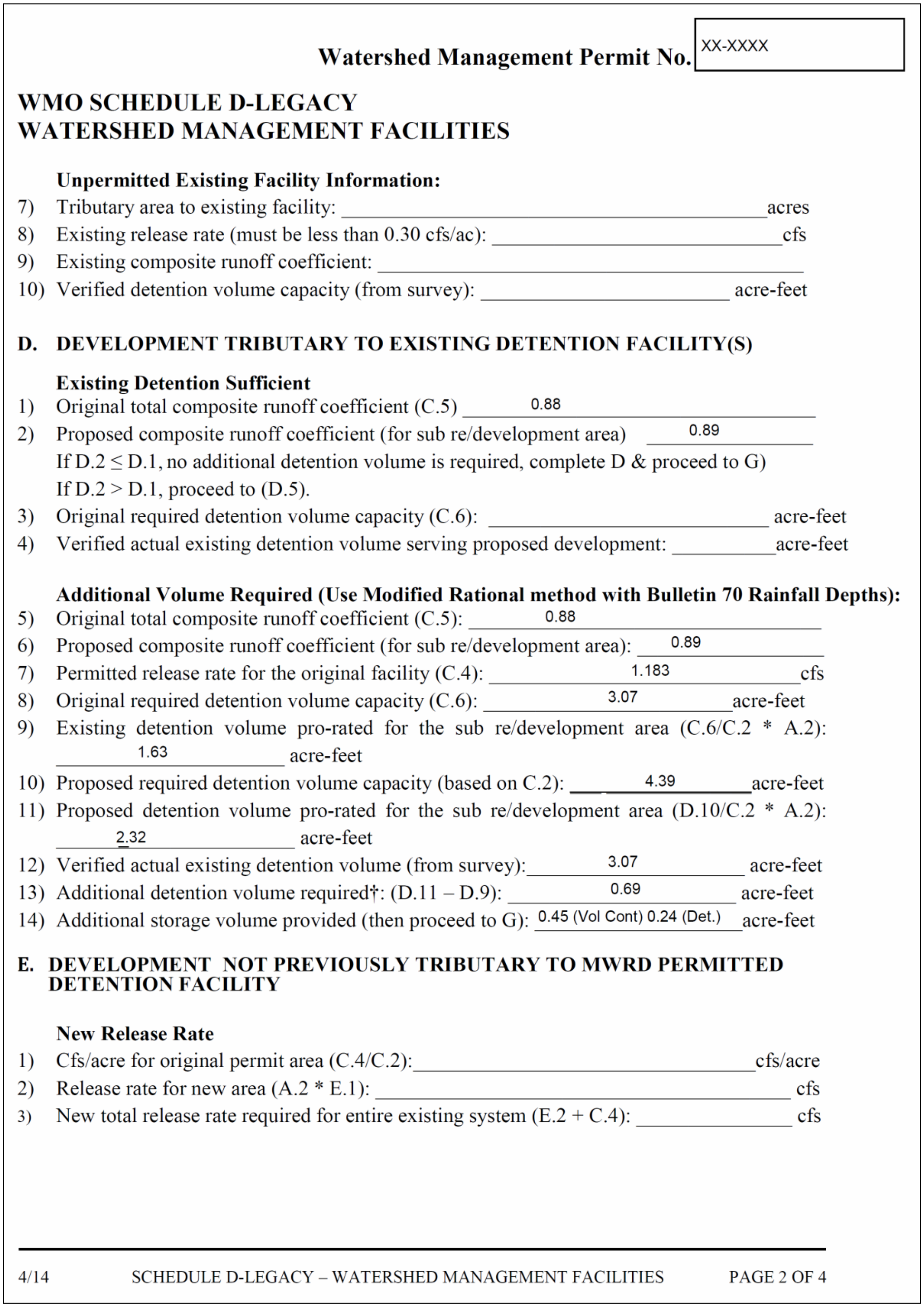
☐ Development is tributary to an existing unpermitted detention facility

Legacy Permit Information (approved Schedule D form):

1) Provide Legacy SPO Permit No(s),: XX-XXXX
2) Total "Area of Site" (Legacy Sch. D, Item C-1) : 11.334 acres
3) "Total Contiguous Ownership, including project" (Legacy Sch. A, 6-B) : 11.334 acres
4) "Net Allowable Release Rate" (Legacy Sch. D, Item C-12): 1.183 cfs
5) "Composite Runoff Coefficient (C)" (Legacy Sch. D, Item D-4): 0.88
6) "Required Detention Capacity at actual release rate" (Legacy Sch. D, Item D-5): 3.07 acre-feet

4/14 SCHEDULE D-LEGACY – WATERSHED MANAGEMENT FACILITIES PAGE 1 OF 4

Schedule D-Legacy for Example 5.11 (Page 1 of 4)



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D-LEGACY
WATERSHED MANAGEMENT FACILITIES

Unpermitted Existing Facility Information:

7) Tributary area to existing facility: ____________ acres
8) Existing release rate (must be less than 0.30 cfs/ac): ____________ cfs
9) Existing composite runoff coefficient: ____________
10) Verified detention volume capacity (from survey): ____________ acre-feet

D. DEVELOPMENT TRIBUTARY TO EXISTING DETENTION FACILITY(S)

Existing Detention Sufficient

1) Original total composite runoff coefficient (C.5) 0.88
2) Proposed composite runoff coefficient (for sub re/development area) 0.89

If D.2 ≤ D.1, no additional detention volume is required, complete D & proceed to G)
If D.2 > D.1, proceed to (D.5).

3) Original required detention volume capacity (C.6): ____________ acre-feet
4) Verified actual existing detention volume serving proposed development: ____________ acre-feet

Additional Volume Required (Use Modified Rational method with Bulletin 70 Rainfall Depths):

5) Original total composite runoff coefficient (C.5): 0.88
6) Proposed composite runoff coefficient (for sub re/development area): 0.89
7) Permitted release rate for the original facility (C.4): 1.183 cfs
8) Original required detention volume capacity (C.6): 3.07 acre-feet
9) Existing detention volume pro-rated for the sub re/development area (C.6/C.2 * A.2): 1.63 acre-feet
10) Proposed required detention volume capacity (based on C.2): 4.39 acre-feet
11) Proposed detention volume pro-rated for the sub re/development area (D.10/C.2 * A.2): 2.32 acre-feet
12) Verified actual existing detention volume (from survey): 3.07 acre-feet
13) Additional detention volume required†: (D.11 – D.9): 0.69 acre-feet
14) Additional storage volume provided (then proceed to G): 0.45 (Vol Cont) 0.24 (Det.) acre-feet

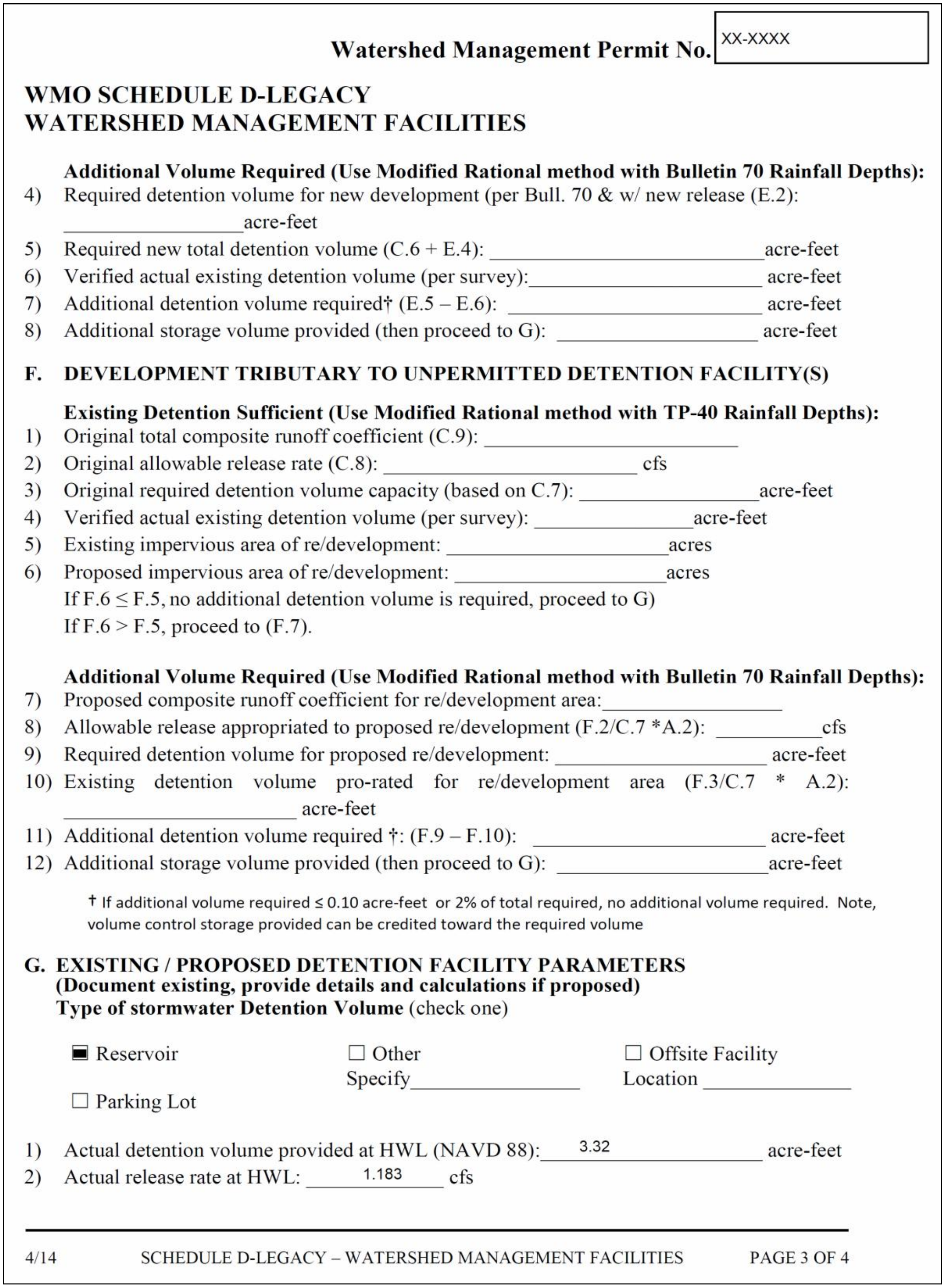
E. DEVELOPMENT NOT PREVIOUSLY TRIBUTARY TO MWRD PERMITTED DETENTION FACILITY

New Release Rate

1) Cfs/acre for original permit area (C.4/C.2): ____________ cfs/acre
2) Release rate for new area (A.2 * E.1): ____________ cfs
3) New total release rate required for entire existing system (E.2 + C.4): ____________ cfs

4/14 SCHEDULE D-LEGACY – WATERSHED MANAGEMENT FACILITIES PAGE 2 OF 4

Schedule D-Legacy for Example 5.11 (Page 2 of 4)



Watershed Management Permit No. XX-XXXX

WMO SCHEDULE D-LEGACY
WATERSHED MANAGEMENT FACILITIES

Additional Volume Required (Use Modified Rational method with Bulletin 70 Rainfall Depths):

4) Required detention volume for new development (per Bull. 70 & w/ new release (E.2): ________________acre-feet
5) Required new total detention volume (C.6 + E.4): ________________________acre-feet
6) Verified actual existing detention volume (per survey):_____________________ acre-feet
7) Additional detention volume required† (E.5 – E.6): _______________________ acre-feet
8) Additional storage volume provided (then proceed to G): __________________ acre-feet

F. DEVELOPMENT TRIBUTARY TO UNPERMITTED DETENTION FACILITY(S)

Existing Detention Sufficient (Use Modified Rational method with TP-40 Rainfall Depths):

1) Original total composite runoff coefficient (C.9): _______________________
2) Original allowable release rate (C.8): _______________________ cfs
3) Original required detention volume capacity (based on C.7): ________________acre-feet
4) Verified actual existing detention volume (per survey): ______________acre-feet
5) Existing impervious area of re/development: ____________________acres
6) Proposed impervious area of re/development: ___________________acres

If F.6 ≤ F.5, no additional detention volume is required, proceed to G)
If F.6 > F.5, proceed to (F.7).

Additional Volume Required (Use Modified Rational method with Bulletin 70 Rainfall Depths):

7) Proposed composite runoff coefficient for re/development area:________________
8) Allowable release appropriated to proposed re/development (F.2/C.7 *A.2): __________cfs
9) Required detention volume for proposed re/development: ___________________ acre-feet
10) Existing detention volume pro-rated for re/development area (F.3/C.7 * A.2): ______________________ acre-feet
11) Additional detention volume required †: (F.9 – F.10): ______________________ acre-feet
12) Additional storage volume provided (then proceed to G): ___________________acre-feet

† If additional volume required ≤ 0.10 acre-feet or 2% of total required, no additional volume required. Note, volume control storage provided can be credited toward the required volume

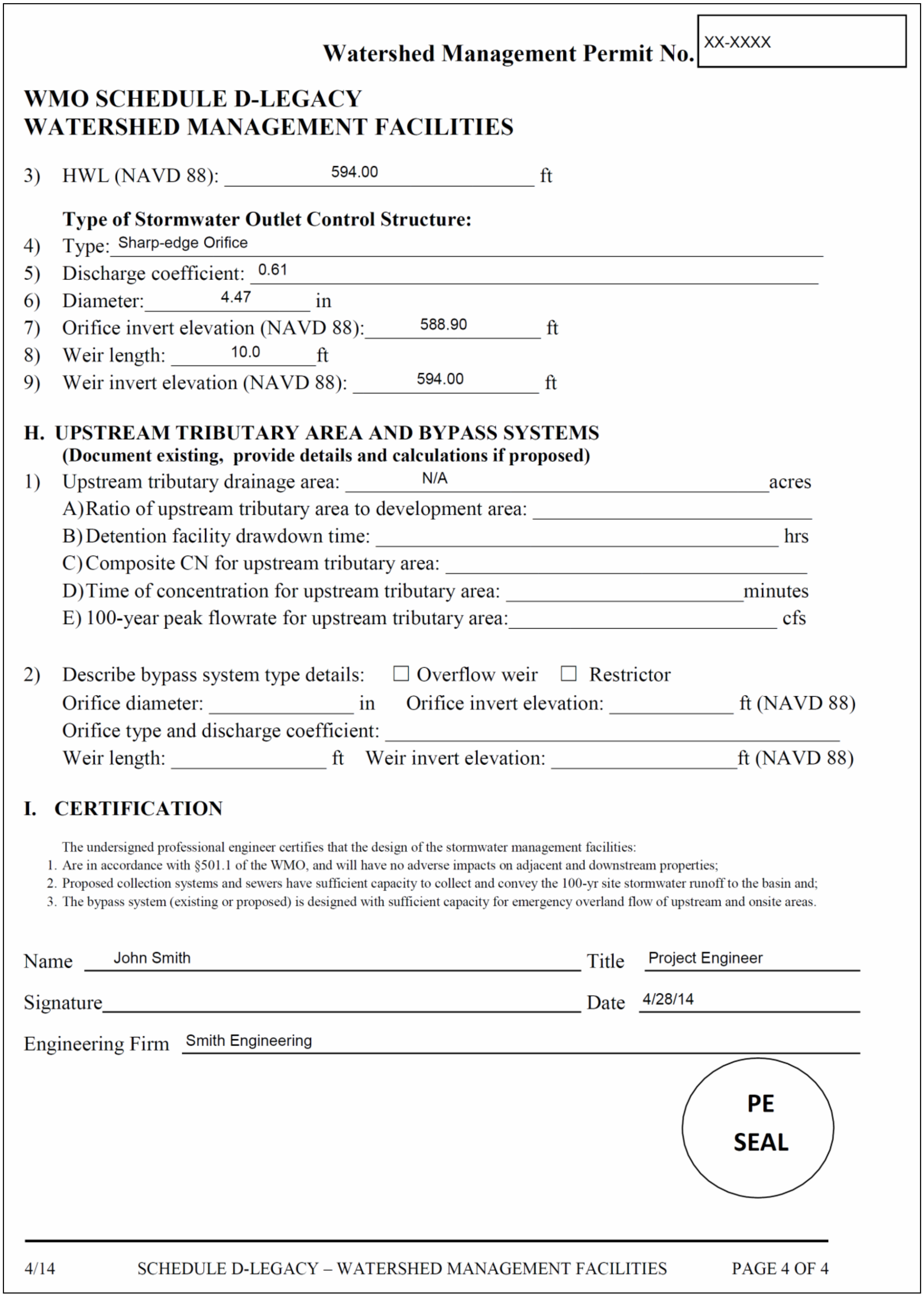
G. EXISTING / PROPOSED DETENTION FACILITY PARAMETERS
(Document existing, provide details and calculations if proposed)
Type of stormwater Detention Volume (check one)

☒ Reservoir
☐ Other Specify________________
☐ Offsite Facility Location ______________
☐ Parking Lot

1) Actual detention volume provided at HWL (NAVD 88): 3.32 acre-feet
2) Actual release rate at HWL: 1.183 cfs

4/14 SCHEDULE D-LEGACY – WATERSHED MANAGEMENT FACILITIES PAGE 3 OF 4

Schedule D-Legacy for Example 5.11 (Page 3 of 4)



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WMO SCHEDULE D-LEGACY
WATERSHED MANAGEMENT FACILITIES

3) HWL (NAVD 88): 594.00 ft

Type of Stormwater Outlet Control Structure:

4) Type: Sharp-edge Orifice
5) Discharge coefficient: 0.61
6) Diameter: 4.47 in
7) Orifice invert elevation (NAVD 88): 588.90 ft
8) Weir length: 10.0 ft
9) Weir invert elevation (NAVD 88): 594.00 ft

H. UPSTREAM TRIBUTARY AREA AND BYPASS SYSTEMS

(Document existing, provide details and calculations if proposed)

1) Upstream tributary drainage area: N/A acres
   - A) Ratio of upstream tributary area to development area: ________
   - B) Detention facility drawdown time: ________ hrs
   - C) Composite CN for upstream tributary area: ________
   - D) Time of concentration for upstream tributary area: ________ minutes
   - E) 100-year peak flowrate for upstream tributary area: ________ cfs

2) Describe bypass system type details: ☐ Overflow weir ☐ Restrictor
Orifice diameter: ________ in Orifice invert elevation: ________ ft (NAVD 88)
Orifice type and discharge coefficient: ________
Weir length: ________ ft Weir invert elevation: ________ ft (NAVD 88)

I. CERTIFICATION

The undersigned professional engineer certifies that the design of the stormwater management facilities:

1. Are in accordance with §501.1 of the WMO, and will have no adverse impacts on adjacent and downstream properties;
2. Proposed collection systems and sewers have sufficient capacity to collect and convey the 100-yr site stormwater runoff to the basin and;
3. The bypass system (existing or proposed) is designed with sufficient capacity for emergency overland flow of upstream and onsite areas.

Name John Smith Title Project Engineer
Signature ________ Date 4/28/14
Engineering Firm Smith Engineering

PE SEAL

4/14 SCHEDULE D-LEGACY – WATERSHED MANAGEMENT FACILITIES PAGE 4 OF 4

Schedule D-Legacy for Example 5.11 (Page 4 of 4)

Parcels with Existing Detention not Permitted Under the SPO

Parcels in **combined sewer areas** were not required to provide **detention** under the **SPO**, however, a local ordinance may have required **detention** for these **developments**. Similarly, a local ordinance may have required detention in a **separate sewer area** when the **SPO** did not require it (for example, if the **development** was constructed prior to 1972 and a local ordinance was in place). Therefore, it is possible that **parcels** may have **existing detention facilities** that were never permitted by the **District**. The WMO also makes allowances for the **redevelopment** of these **parcel**s. These allowances are outlined in §505.4 of the WMO.

If the **redevelopment** meets **all** of the following conditions:

- Actual detention volume is verified to meet or exceed the detention volume calculated according to standards set under the **SPO**, and signed and sealed by either a **Professional Engineer** or a **Professional Land Surveyor**;
- Actual release rate from the existing control **structure** is verified to be less than the requirements set under the **SPO**, and the calculations are signed and sealed by either a **Professional Engineer**;
- The **redevelopment** provides treatment of the **volume control storage** as required in the WMO; and
- The **redevelopment** provides adequate capacity to convey **stormwater runoff** to the **existing detention facility** for all storms up to and including the 100-year **storm event**.

Then, the following **redevelopment** allowances may be granted:

- If the **redevelopment's** proposed **impervious area** does not exceed the existing **impervious area**, additional **stormwater** detention volume is not required;
- If the **redevelopment's** proposed **impervious area** exceeds the existing **impervious area**, additional **stormwater** detention volume shall be provided for the **redevelopment**. In such situations, the modified Rational Method using **Bulletin 70** rainfall data may be used to calculate the additional required storage volume. The release rate for the **redevelopment** will be based on a pro-rata share of **redevelopment's** portion of the actual release rate of the control **structure**.

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