Appendix 1: Post-Construction Stormwater Design Examples

This appendix uses three hypothetical development sites in order to demonstrate the design of post-construction stormwater practices presented earlier in specifications.

Each practice example utilizes the existing and the proposed developed site and hydrologic characteristics to determine the sizing and configuration of each practice. The base requirements are presumed to be Ohio EPA's Construction General Permit post-construction requirements (detention of the water quality volume) and the detention of the critical storm (see the Critical Storm Method) from the development in order to prevent increases in downstream flooding and streambank erosion.

Each practice use the following steps to proceed through the design:

Step 1 - Calculate Water Quality Volume (WQv)
Step 2 - Compute Peak Discharge Requirements
Step 3 - Identify Other Local Development Criteria/Requirements
Step 4 - Determine if the Site and Soils Are Appropriate for the Practice
Step 5 - Determine Practice Location and Preliminary Geometry to Meet Requirements
Step 6 - Check Design to Ensure All Requirements Are Met

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Section A: Dry Extended Detention Basin

This design example illustrates the design of a dry extended detention stormwater basin that provides water quality treatment and peak discharge control within a highly impervious commercial development.

The layout of the North Country Automotive development is shown in Figure 1.A.1. The development site totals 7.7 acres draining to a single point on the north property line with no offsite watershed area. The site impervious area at completion of construction is estimated to be 5.3 acres. The example assumes that the local community has adopted the Critical Storm Method criteria to control peak discharges[1]. The pre-developed and post-developed site flow paths are shown in Figure 1.A.2. (limited to those used for calculations).

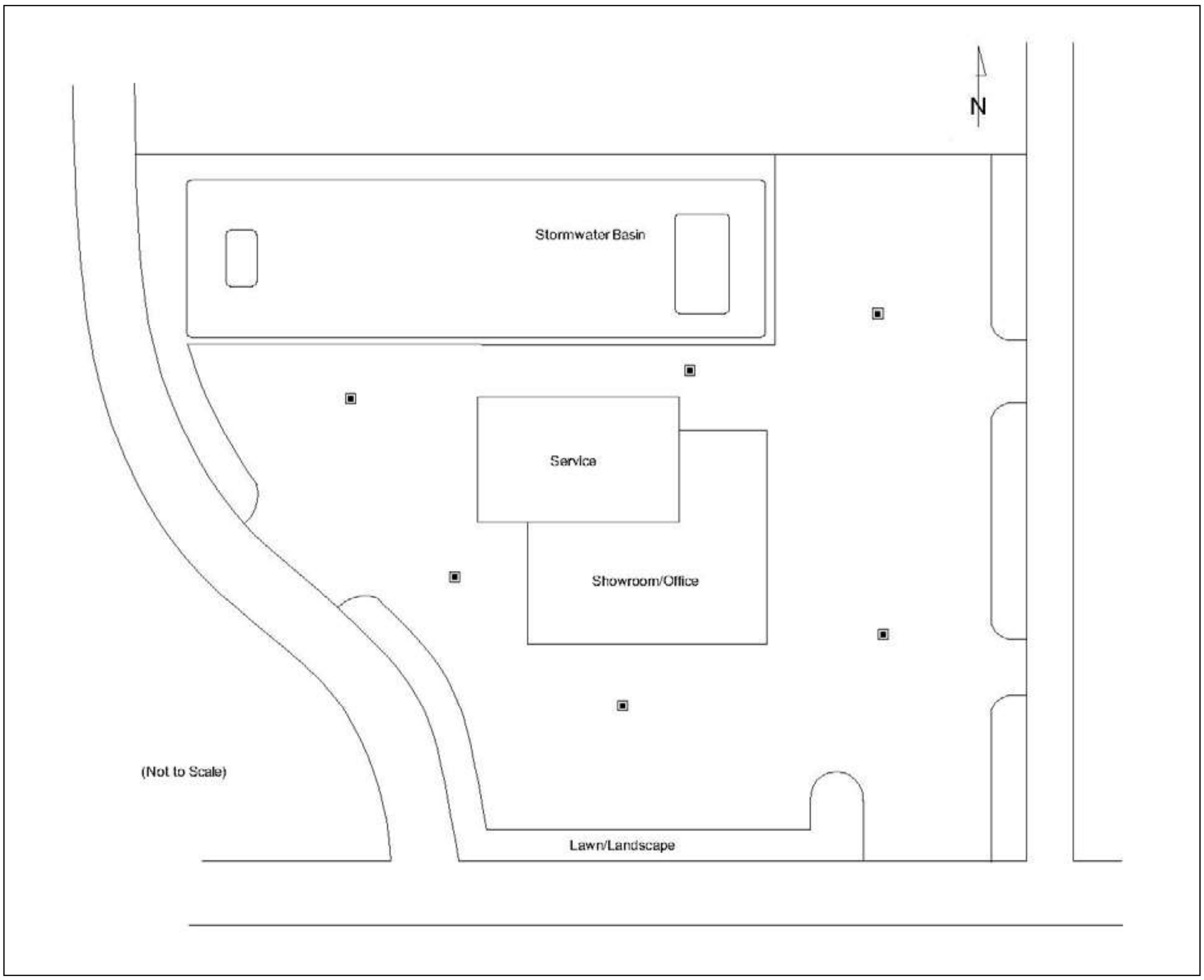

Figure 1.A.1. North Country Automotive Site Plan.

[1] The Critical Storm Method is a set of criteria for controlling the peak discharge of stormwater from large storm events (1 - 100 yr recurrence interval) recommended by ODNR-DSWC since 1980. See Goettemoeller, R.L., D.P. Hanselmann, and J.H. Bassett. 1980. Ohio Stormwater Control Guidebook. Ohio Department of Natural Resources, Division of Soil and Water Districts, p47.

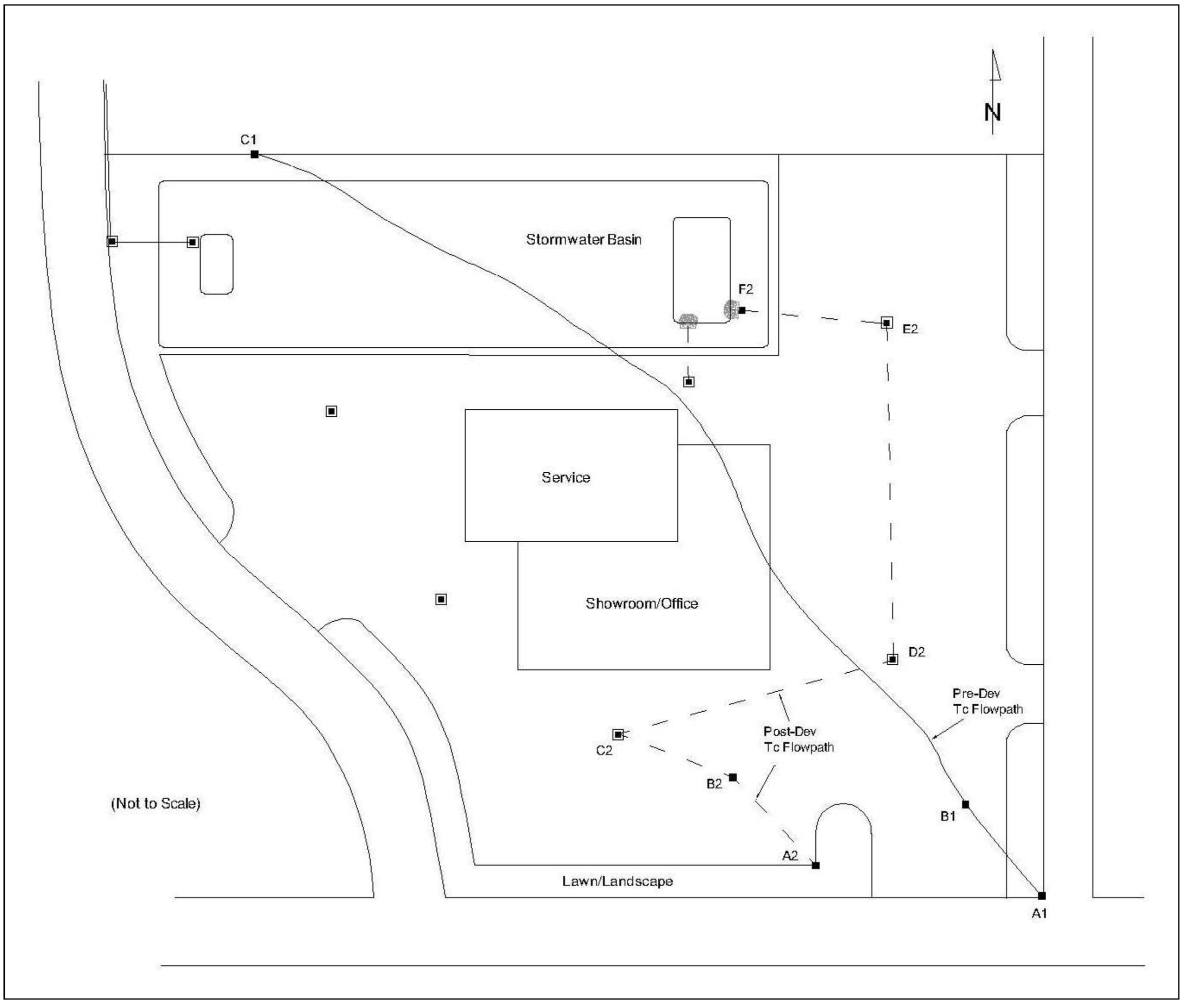

Figure 1.A.2. North Country Automotive Site Plan with pre-developed and post-developed flow paths.

<u>Site Data</u>
Total Drainage Area (A) = 7.7 ac
Estimated Impervious Area = 5.3 ac
Soil Types
Existing: 100% HSG-C
Proposed: 100% HSG-D

<u>Summary Hydrologic Data</u>
WQv = 0.26 ac-ft

	Pre	Post
CN =	70	96
Tc =	33.3 min	3.5 min

Calculation of Preliminary Stormwater Storage Volumes and Peak Discharges

Step 1 - Calculate Water Quality Volume (WQv)

The water quality volume (WQv) is a post-construction stormwater control requirement in Ohio (NPDES Storm Water Construction General Permit; OEPA, 2008). The WQv is determined according to the following equation:

$$WQv = C*P*A \qquad \text{Equation 1.A.1}$$

where:

C = runoff coefficient

P = 0.75 inch precipitation depth

A = drainage area

For open water, C = 1. At this site, the surface area of the detention basin at full WQv is estimated to be 0.4 acres.

For the remainder of the site, the runoff coefficient can either be selected from Table 1 of the NPDES Storm Water Permit, or calculated using the following equation:

$$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \qquad \text{Equation 1.A.2}$$

where i is the fraction of post-construction impervious surface.
For this site, total impervious area = 5.3 acres from a drainage area of 7.3 acres (i.e., total site drainage area - surface area of detention basin at full WQv).

$$i = 5.3/7.3 = 0.73 \qquad \text{Equation 1.A.3}$$

$$C = 0.858(0.73)^3 - 0.78(0.73)^2 + 0.774(0.73) + 0.04 = 0.52 \qquad \text{Equation 1.A.4}$$

Therefore, the WQv is:

$$WQv = [1.0*0.75 \text{ in}*0.4 \text{ ac} + 0.52*0.75 \text{ in}*7.3 \text{ ac}]*(1 \text{ ft}/12 \text{ in}) \qquad \text{Equation 1.A.5}$$

$$= \underline{0.26 \text{ ac-ft}}$$

$$= 11{,}400 \text{ ft}^3$$

Peak Discharge Summary
Project: North Country Automotive

	Cover Description	Soil Name	Hydrologic Group	Drainage Y/N	CN	Area (acres)
Existing Condition	Woods (good condition)	Ellsworth	C		70	7.7
	Existing Conditions				**70**	**7.7**
	Cover Description	Soil Name	Hydrologic Group		CN	Area (acres)
Proposed Condition	Impervious Area				98	5.3
	Open space (good condition)	Ellsworth	D		80	0.8
	Detention Basin				98	1.6
	Proposed Conditions				**96**	**7.7**

Table 1.A.1. Curve Number (CN) for existing (pre-developed) and proposed (post-developed) condition.

	Segment	Flow Type	Surface Cover	Mannings n	Length ft	Slope %	Velocity ft/s	Tt min
Existing Condition	A_1 to B_1	Overland - sheet	Woods - Light Underbrush	0.40	100	3		21.0
	B_1 to C_1	Overland - shallow conc	Woods - Light Underbrush	0.10	700	3.5	0.95	12.3
	Total	**Existing**			**800**			**33.3**
	Segment	Flow Type	Surface Cover	Mannings n	Length ft	Slope %	Veloctiy ft/s	Tt min
Proposed Condition	A_2 to B_2	Overland - sheet	Pavement	0.011	100	2.0		1.4
	B_2 to C_2	Overland - shallow conc	Pavement	0.025	100	2.0	2.9	0.6
	C_2 to D_2	Pipe - storm drain (12")	Pipe	0.013	250	2.0	6.4	0.6
	D_2 to E_2	Pipe - storm drain (15")	Pipe	0.013	300	2.0	7.4	0.7
	E_2 to F_2	Pipe - storm drain (18")	Pipe	0.013	150	4.0	11.9	0.2
	Total	**Proposed**			**900**			**3.5**

Table 1.A..2. Time of Concentration (Tc) for existing ((pre-developed) and proposed (post-developed) condition.

RI years	P in	Q_{pre} in	Q_{post} in	Percent Increase Q	q_{pre} cfs	q_{post} cfs
1	2.00	0.24	1.57	554	0.86	21.3
2	2.40	0.41	1.96	378	1.8	26.1
5	2.98	0.70	2.53	261	3.7	33.1
10	3.47	0.99	3.02	205	5.5	39.0
25	4.17	1.44	3.71	158	8.5	47.3
50	4.76	1.86	4.29	131	11.2	54.3
100	5.38	2.32	4.91	112	14.1	61.6

Table 1.A.3. Summary runoff depth (Q) and peak discharge (q) for existing (pre-developed) and proposed (post-developed) conditions with critical storm (bold type).

Step 2 - Compute Peak Discharge Requirements

> Note: Peak discharge control is typically regulated through local entities (e.g. stormwater district, municipal, township or county governments). The state of Ohio recommends use of the Critical Storm Method[1] for peak discharge control, but the requirements will vary by community. Check local stormwater regulations to determine which peak discharge control method you must use.

This example uses the NRCS Curve Number Methodology to perform hydrologic calculations. TR-20, HEC-HMS or other software that uses NRCS procedures should provide similar results.

Tables 1-1 and 1-2 summarize the inputs necessary to determine the curve number (CN) and time of concentration (Tc) for the existing (pre-development) and proposed (post-development) conditions. Table 1-3 summarizes the existing and proposed runoff depths and peak discharges for the 1-year, 24-hr through 100-year, 24-hr rainfall events.

The *critical storm* is determined from the percent increase in runoff volume from the 1-year, 24-hr storm for the proposed (post-developed) conditions when compared to the existing (pre-developed) conditions (Goettemoeller et al., 1980):

$$Percent\ Increase = \frac{Q_{post} - Q_{pre}}{Q_{pre}} x\, 100 \qquad \text{Equation 1.A.6}$$

From Table 1-3, the percent increase in the 1-year, 24-hr runoff for the proposed development is:

$$Percent\ Increase = \frac{1.57 - 0.24}{0.24} x\ \ 100 = 554\% \qquad \text{Equation 1.A.7}$$

For an increase greater than 500%, the *critical storm* for peak discharge control is the 100-year, 24-hr event - i.e., the 100-year, 24-hr post-developed peak discharge must be less than the existing (pre-developed) 1-year, 24-hr peak discharge. These values are shown in bold type in Table 1.A.3.

Step 3 - Identify Other Local Development Criteria/Requirements

Commercial development in this community is subject to a 5% minimum landscaped area requirement - the proposed design meets this requirement. No additional setback or stormwater requirements were identified.

Step 4 - Determine if the Development Site and Soils Are Appropriate for the Use of a Dry Extended Detention Basin

The site drainage area is 7.7 acres, all of which is mapped as Ellsworth silt loam soil in the county soil survey. Ellsworth silt loam soils are suitable for creation of an extended detention basin with a wet forebay and permanent micropool. The subsoil is silty clay loam derived from glacial till and has slow permeability. Because the soil has slow permeability, there may be extended periods when the basin cannot be mowed. This subsoil is suitable material for construction of the embankment for the stormwater basin.

[1] The Critical Storm Method is a set of criteria for controlling the peak discharge of stormwater from large storm events (1 - 100 yr recurrence interval) recommended by ODNR-DSWC since 1980. See Goettemoeller, R.L., D.P. Hanselmann, and J.H. Bassett. 1980. Ohio Stormwater Control Guidebook. Ohio Department of Natural Resources, Division of Soil and Water Districts, p47.

Step 5 - Determine Pond Location and Develop Preliminary Geometry to Meet WQv and Peak Discharge Requirements

The proposed location of the stormwater basin (see Figure 1.A.2) reflects the best combination of characteristics (landscape position, access to outlet, minimized earth moving, appropriate soils, etc.) for siting the basin. Existing ground elevation at the proposed pond outlet is 935 MSL. An existing 24" storm sewer runs along the west edge of the property, with an invert elevation of 928 MSL at the proposed discharge point. [For more information on siting and planning an extended detention basin, see section 2.6.]

The basin will be designed to include a permanent micropool and wet forebay, an extended detention volume to protect water quality and stream channel stability, and storage necessary to control the peak discharge rate.

The NPDES Storm Water Permit (OEPA, 2008) specifies a dry extended detention basin include a water quality volume (WQv) with a drawdown time of 48 hours. The permit also requires an additional sediment storage volume equal to 20% of the WQv which, for a dry extended detention basin, should consist of a permanent micropool and forebay each sized at 10% of the WQv.

$$V_{micropool} \text{ and } V_{forebay} \geq 0.1*WQv = 0.1*0.26 \text{ ac-ft} = 0.026 \text{ ac-ft} = 1140 \text{ ft}^3 \qquad \text{Equation 1.A.8}$$

A plan view of the basin layout (Figure 1.A.3) reflects the following:

- extended detention water quality volume (WQv)
- a wet forebay with a minimum volume of 0.1*WQv and 3' depth
- permanent micropool with a minimum volume of 0.1*WQv and 4' depth
- a flow length to flow width ratio of 4:1, exceeding the 3:1 requirement
- positive slope (~0.8%) toward the outlet to facilitate surface drainage [Note: this is not enough slope to prevent extended periods of soil wetness.]
- 4:1 side slopes for safety and ease of maintenance
- an emergency spillway constructed in native soil (i.e., not in the constructed embankment)

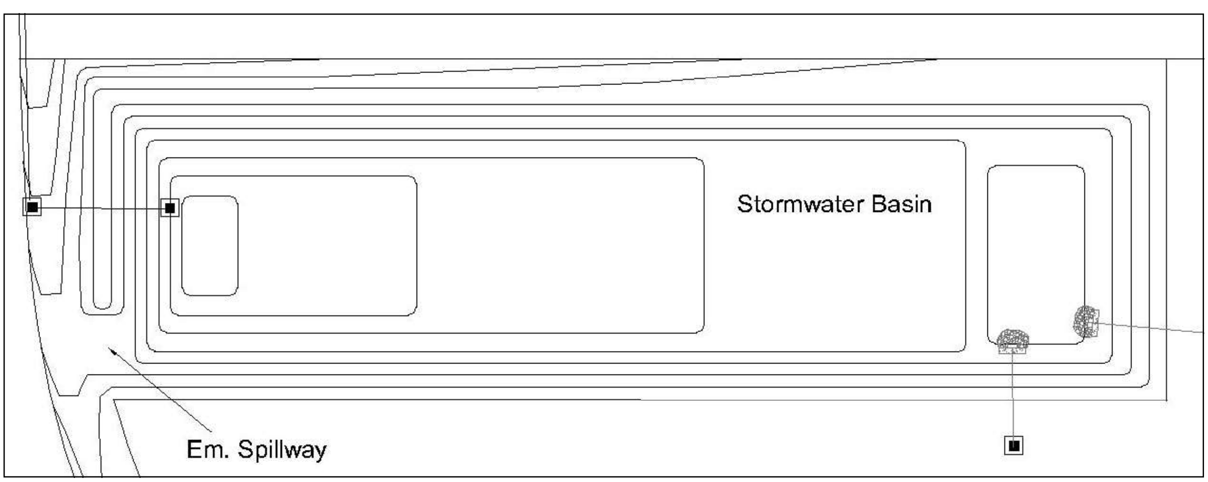

Figure 1.A.3. Plan View of the Basin Layout.

Set elevations for pond structures

- The pond bottom is set at elevation 930.0 and the riser invert is set at 929
- A manhole will be installed in the sewer main with a barrel invert (outfall) elevation at 928 ft

Establish permanent micropool and WQv water surface elevations

A stage-area-storage table (Table 1.A.4) reflects the geometry of the stormwater basin (Figure 1.A.3) designed to meet permanent micropool, forebay, extended detention (WQv) and peak discharge control requirements.

- The permanent micropool volume of 0.05 ac-ft (surface elevation 934.0) exceeds 0.1*WQv
- The extended detention water quality volume WQv of 0.26 ac-ft above the permanent micropool has a top elevation of approximately 935.5

Elevation MSL (ft)	Surface Area (acre)	Average Area (acre)	Incremental Depth (ft)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)	Vol above Perm Pool (ac-ft)
930.0	0.004	-	-	-	-	-
934.0	0.02	0.01	4.0	0.05	0.05	-
934.5	0.10	0.06	0.5	0.03	0.08	0.03
935.0	0.22	0.16	0.5	0.08	0.16	0.11
935.5	0.40	0.31	0.5	0.15	0.31	0.26
936.0	0.52	0.39	0.5	0.20	0.48	0.43
937.0	1.05	0.76	1.0	0.76	1.24	1.19
938.0	1.25	1.15	1.0	1.15	2.39	2.34
939.0	1.36	1.30	1.0	1.30	3.69	3.64
940.0	1.47	1.41	1.0	1.41	5.10	5.05
941.0	1.58	1.53	1.0	1.53	6.63	6.58

Table 1.A.4. Stage-Area-Storage Information for Dry Extended Detention Basin.

Determine orifice size for 48-hour drawdown of WQv

The controlling parameters are WQv = 0.26 ac-ft, depth of WQv = 1.5 ft, and minimum drain time, T_d = 48 hours. Note that "the outlet structure for the post-construction BMP must not discharge more than the first half of the WQv in less than one-third of the drain time" (p22, NPDES Storm Water Construction General Permit; OEPA, 2008).

The average discharge rate for the WQv is:

$$Q_{avg} = \frac{WQv}{T_d} = \frac{(0.26\ ac \cdot ft)\left(\frac{43560\ ft^2}{1\ ac}\right)}{(48\ hr)\left(3600\ \frac{s}{hr}\right)} = 0.065\ cfs \qquad \text{Equation 1.A.9}$$

The discharge equation for an orifice is:

$$Q = ca\sqrt{2gh} \qquad \text{Equation 1.A.10}$$

By rearranging, we can estimate needed orifice area, as:

$$a = \frac{Q}{c\sqrt{2gh}} \qquad \text{Equation 1.A.11}$$

Using an orifice coefficient of c = 0.6, and average head, h = d/2 = (1.5 ft)/2 = 0.75 ft, the required orifice size is:

$$a = \frac{0.065\ \frac{ft^3}{s}}{0.6\sqrt{2(32.2\ \frac{ft}{s^2})(0.75\ ft)}} = 0.0156\ ft^2 \qquad \text{Equation 1.A.12}$$

resulting in an estimated orifice diameter of:

$$d = \left(\frac{4a}{3.14}\right)^{0.5} = \left[\frac{4(0.0156 ft^2)}{3.14}\right]^{0.5} = 0.141 ft \times \frac{12 in}{1 ft} = 1.7 in \qquad \text{Equation 1.A.13}$$

This estimate is a good starting point for selecting the WQv orifice size, because it will always meet the two drawdown requirements: (1) the specified minimum drain time, T_d; and (2) the outlet must not discharge more than the first half of the WQv (or EDv) in less than one-third of the drain time. A larger or smaller orifice should be considered if it will help meet other environmental, cost, or maintenance goals, but must be tested for the two drawdown criteria.

Choosing the largest orifice size meeting the criteria lowers the likelihood of a clogged orifice and slightly lessens the storage volume required to meet the peak discharge requirement. In this situation, a 1.7" diameter orifice was the largest orifice that met the above two drawdown requirements (see Figure 1.A.4) and, thus, will be used as the extended detention (WQv) outlet.

Determine storage and outlet configuration to meet peak discharge requirements

As noted under Step 2, this dry detention basin is designed to meet the Critical Storm Method (CSM) for peak discharge control as well as the WQv requirement. Additional storage volume must be added that, with appropriate outlet design, will allow the basin to meet the following requirement:

- The peak rate of discharge from the post-construction 100-year, 24-hour event (the critical storm) must be released at the existing (pre-development) 1-year, 24-hour discharge rate

Proprietary stormwater modeling software was used to try a combination of stage-storage and outlet configurations until the critical storm requirement was satisfied while considering the following:

- use best practices outlined in Section 2.6 of the Rainwater and Land Development manual
- minimize cut/fill and grading

The resulting detention basin geometry is presented in Figure 1.A.3 and Table 1.A.4. The resulting outlet configuration is shown in Figure 1.A.5.

The outlet structure consists of a 3 ft by 3 ft concrete catch basin (e.g., ODOT No. 2-3) with invert at 929 MSL and 2.5'x2.5' iron grate at 938.1 MSL. The following comprise the outlets:

- 1.7" diameter extended detention water quality volume (WQv) orifice (invert 934.0 MSL) drilled into 6" PVC pipe using a non-clogging design
- 4.2" diameter orifice (invert 935.5 MSL) that controls release of the critical storm (100-year, 24-hour)
- 2.5'x2.5' iron grate (invert 938.1 MSL) for emergency overflow and maintenance access

The catch basin will be connected - using a 12" diameter conduit - to the 2' diameter storm sewer at the road along the west property boundary. A tailwater analysis was performed using the modeling software and the storm sewer's design elevation (invert at 928 MSL; 25-yr full pipe flow at 930 MSL) and assumed elevation for the 100-yr event (935 MSL).

In addition, this design includes an emergency spillway excavated into native soil with the following characteristics:

- Invert (crest) elevation of 938.5 MSL
- Level section length of 25 ft, weir length (i.e., crest width) of 25 ft
- Spillway crest perpendicular to flow
- With all other outlets blocked and starting from the permanent pool elevation of 934.0 MSL, will safely convey the 100-yr, 24-hr event with at least 1 ft freeboard below top of embankment

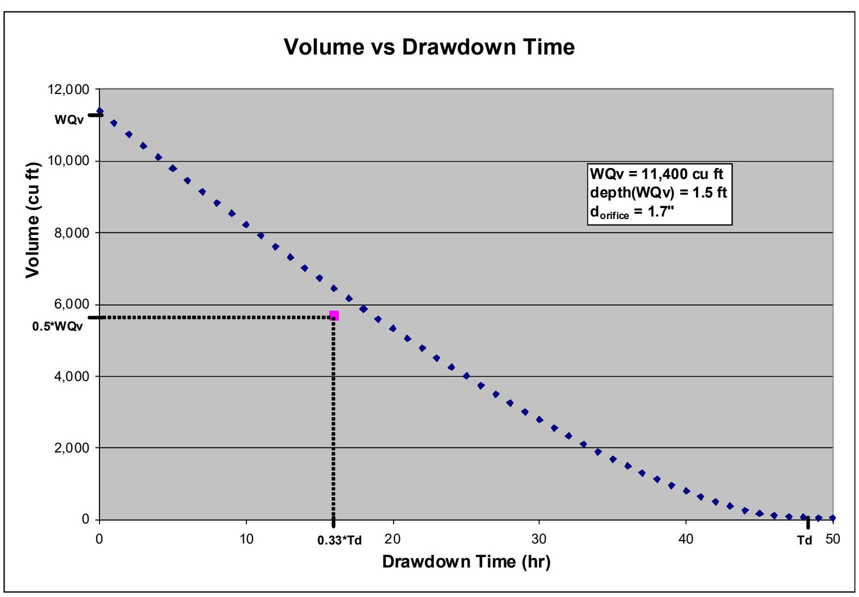

Figure 1.A.4. Dry Extended Detention Basin - Drawdown from Full WQv.

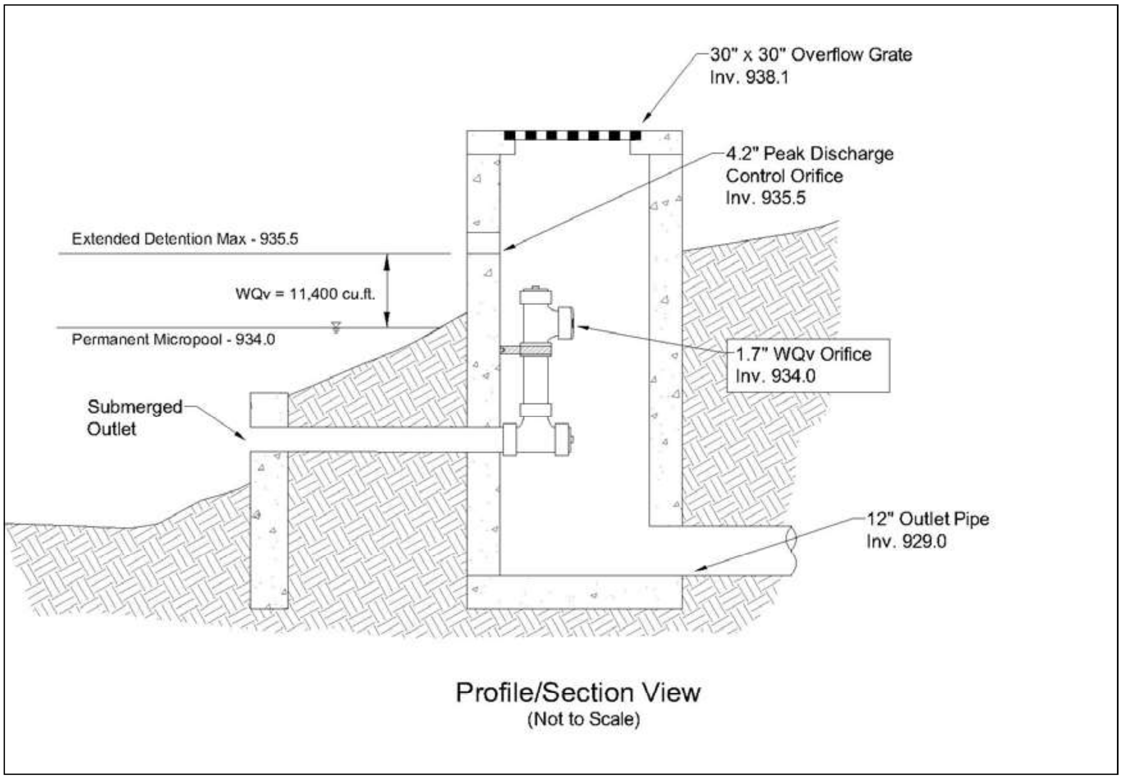

Figure 1.A.5. Outlet Configuration for Dry Extended Detention Basin (not to scale).

Step 6 - Check Design to Ensure All Requirements Are Met

From full WQv, check that WQv meets minimum 48 hour drain time, and discharges no more than 1/2 the water quality volume, 0.5*WQv (5050 ft^3), in the first 1/3 of the drain time, 0.33*T_d (16 hr). This requirement is met and illustrated in Figure 1.A.4.

Check peak discharge for all events (see Table 1.A.5).

RI years	P in	$q_{post\text{-}in}$ cfs	Allowed $q_{post\text{-}out}$ cfs	Estimated $q_{post\text{-}out}$ cfs
1	2.00	21.3	0.86	0.48
2	2.40	26.1	0.86	0.55
5	2.98	33.1	0.86	0.63
10	3.47	39.0	0.86	0.68
25	4.17	47.3	0.86	0.75
50	4.76	54.3	0.86	0.80
100	5.38	61.6	0.86	0.85

Table 1.A.5. Critical Storm Method Peak Discharge Check.

Section B: Wet Extended Detention Basin

This design example illustrates the design of a wet extended detention stormwater basin that provides water quality treatment and peak discharge control within a condominium development. This residential development will consist of 74 units of "active senior" living units and a well-equipped clubhouse for recreation, exercise and social functions. The layout of the development is shown in figure 1.B.1.

The development site consists of 24.2 acres having 10.2 acres of impervious area. An additional 8.5 acres of off-site area drains to the development site. The pre-developed site soils and flow paths are shown in Figure 1.B.2, while the post-developed flow paths (limited to that used for calculations) are shown in figure 1.B.3.

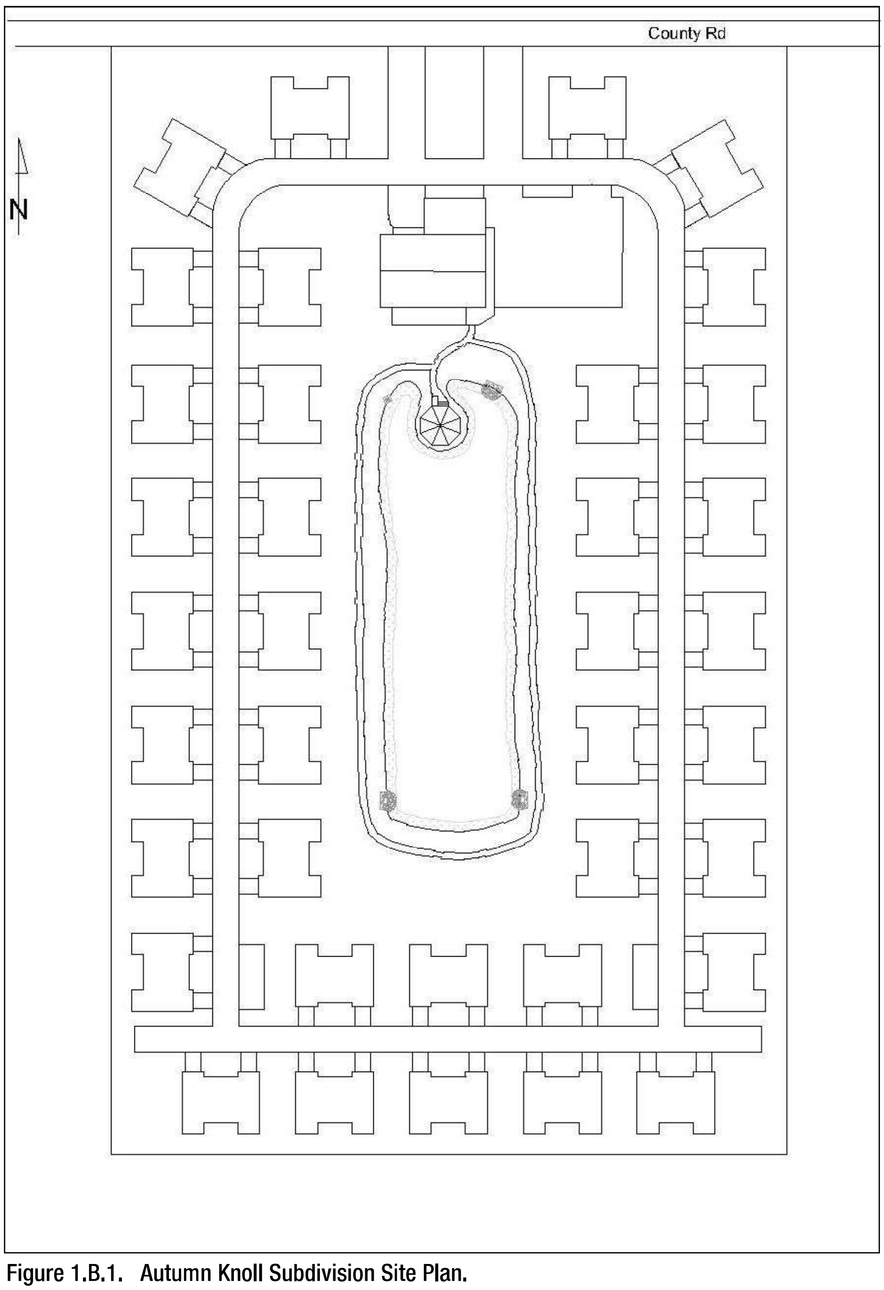

Figure 1.B.1. Autumn Knoll Subdivision Site Plan.

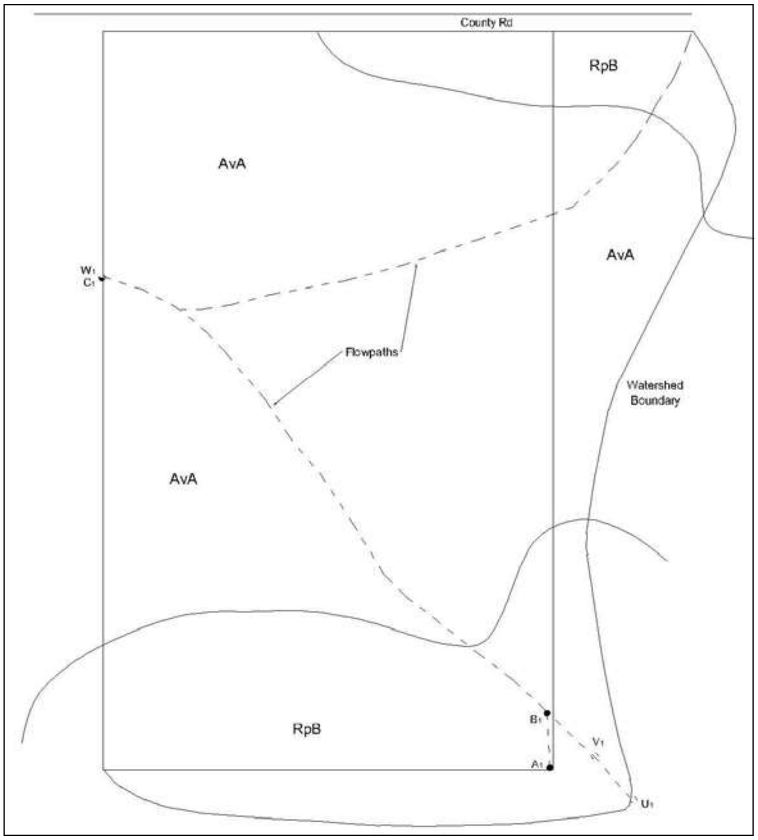

Figure 1.B.2. Pre-Development On-site and Off-site Soils and Drainage.

<u>Development Site Data</u>
Total On-Site Drainage Area (A) = 24.2 ac
Estimated Impervious Area = 10.2 ac
Soil Types
Existing: 25% HSG-C, 75% HSG-D
Proposed: 100% HSG-D

<u>Drainage from Off-site</u>
Off-site Drainage Area (A) = 8.5 ac
Estimated Impervious Area = 0 ac
Soil Types: 60% HSG-C, 40% HSG-D

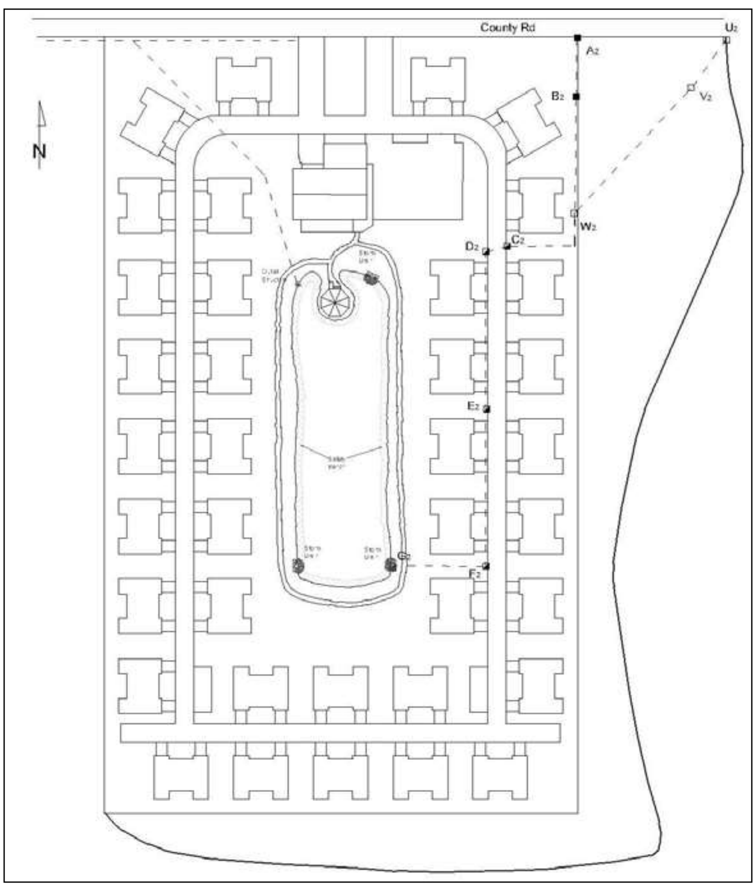

Figure 1.B.3. Post-Development On-site and Off-site Drainage.

<u>Summary Hydrologic Data</u>
WQv = 0.53 ac-ft
EDv = 0.40 ac-ft

	Pre	Post
CN(site) =	84	89
Tc(site) =	66 min	27 min
CN(offsite) =	83	83
Tc(offsite) =	71 min	30 min

Calculation of Preliminary Stormwater Storage Volumes and Peak Discharges

Step 1 - Calculate Water Quality Volume (WQv)

The water quality volume (WQv) is a post-construction stormwater control requirement in Ohio (NPDES Storm Water Construction General Permit; OEPA, 2008). The WQv is determined according to the following equation:

$$WQv = C*P*A \qquad \text{Equation 1.B.1}$$

where:

C = runoff coefficient

P = 0.75 inch precipitation depth

A = drainage area

For open water, C = 1. At this site, the surface area of the detention basin at full extended detention volume is estimated to be 1.5 acres.

For the remainder of the site, the runoff coefficient can either be selected from Table 1 of the NPDES Storm Water Permit, or calculated using the following equation:

$$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \qquad \text{Equation 1.B.2}$$

where i is the fraction of post-construction impervious surface.

For this site, total impervious area = 10.2 acres from a drainage area of 22.7 acres (i.e., total site drainage area - surface area of detention basin).

$$i = 10.2/22.7 = 0.45 \qquad \text{Equation 1.B.3}$$

$$C = 0.858(0.45)^3 - 0.78(0.45)^2 + 0.774(0.45) + 0.04 = 0.31 \qquad \text{Equation 1.B.4}$$

Therefore, the WQv is:

$$WQv = [1.0*0.75 \text{ in}*1.5 \text{ ac} + 0.31*0.75 \text{ in}*22.7 \text{ ac}]*(1 \text{ ft}/12 \text{ in}) \qquad \text{Equation 1.B.5}$$

$$= \underline{0.53 \text{ ac-ft}}$$

$$= 23{,}200 \text{ ft}^3$$

Step 2 - Compute Peak Discharge Requirements

> Note: Peak discharge control is typically regulated through local entities (e.g. stormwater district, municipal, township or county governments). The state of Ohio recommends use of the Critical Storm Method[1] for peak discharge control, but the requirements will vary by community. Check local stormwater regulations to determine which peak discharge control method you must use.

This example uses the NRCS Curve Number Methodology to perform hydrologic calculations. TR-20, HEC-HMS or other software that uses NRCS procedures should provide similar results.

Tables 1.B.1 and 1.B.2 (a and b) summarize the inputs necessary to determine the curve number (CN) and time of concentration (Tc) for the existing (pre-development) and proposed (post-development) conditions. Table 1.B.3 summarizes the existing and proposed runoff depths and peak discharges for the 1-year, 24-hr through 100-year, 24-hr rainfall events.

The *critical storm* is determined from the percent increase in runoff volume generated from the development site for the 1-year, 24-hr storm, comparing proposed (post-developed) conditions to the existing (pre-developed) conditions (Goettemoeller et al., 1980):

$$Percent\ \ Increase = \frac{Q_{post} - Q_{pre}}{Q_{pre}} x\, 100 \qquad \text{Equation 1.B.6}$$

Using data from Table 1.B.3, the percent increase in the 1-year, 24-hr runoff volume for the proposed development site is:

$$Percent\ \ \ Increase\ \ = \frac{1.38 - 1.05}{1.05} x\ \ 100\ \ = 31\,\% \qquad \text{Equation 1.B.7}$$

For an increase greater than 20% but less than 50%, the *critical storm* for peak discharge control is the 5-year, 24-hr event - i.e., the stormwater detention facility must be designed such that the 5-year, 24-hr post-developed peak discharge does not exceed the existing (pre-developed) 1-year, 24-hr peak discharge. These values are shown in bold type in Table 1.B.3. In addition, the proposed peak discharge for the 10-year through 100-year events must not exceed the existing (pre-developed) discharge for like year events.

Step 3 - Identify Other Local Development Criteria/Requirements

The local subdivision regulations included lot size, lot width, road width and setback requirements that affected site layout. No additional stormwater requirements were identified.

Step 4 - Determine if the Development Site and Soils Are Appropriate for the Use of a Wet Extended Detention Basin

The site drainage area is 24.2 acres, all of which is mapped as Rossmoyne silt loam or Avonburg silt loam soil in the county soil survey[2]. The wet basin will be located in area mapped solely as Avonburg silt loam. Avonburg silt loam soils are suitable for creation of an extended detention basin with a permanent pool. The subsoil is clay loam derived from glacial till and has slow permeability. The constructed basin will lie predominantly below existing grade; a small amount of soil material will be used for construction of an embankment along the western and northern edges of the stormwater basin.

[1] The Critical Storm Method is a set of criteria for controlling the peak discharge of stormwater from large storm events (1 - 100 yr recurrence interval) recommended by ODNR-DSWC since 1980. See Goettemoeller, R.L., D.P. Hanselmann, and J.H. Bassett. 1980. Ohio Stormwater Control Guidebook. Ohio Department of Natural Resources, Division of Soil and Water Districts, p47.

[2] Note - Readily available county soil survey data provide excellent planning level information but typically are not accurate enough for engineering design. As part of site evaluation, a certified soil scientist should be contracted to perform an on-site soil investigation to provide an accurate representation of soil conditions and limitations at the development site.

Peak Discharge Summary
Project: Autumn Knoll Senior Living Residential Development

	Cover Description	Soil Name	Hydrologic Group	CN	Area (acres)
Existing Condition Site Only	Agriculture - Row Crop SR & CR	Rossmoyne	C	82	6.1
	Agriculture - Row Crop SR & CR	Avonburg	D	85	18.1
	Existing Conditions			**84**	**24.2**
	Cover Description	**Soil Name**	**Hydrologic Group**	**CN**	**Area (acres)**
Proposed Condition Site Only	Impervious Area			98	10.2
	Open space (good cond)	Rossmoyne/Avonburg	D	80	12.0
	Detention Basin			98	2.0
	Proposed Conditions			**89**	**24.2**
	Cover Description	**Soil Name**	**Hydrologic Group**	**CN**	**Area (acres)**
Off-site Condition	Agriculture - Row Crop SR & CR	Rossmoyne	C	82	5.1
	Agriculture - Row Crop SR & CR	Avonburg	D	85	3.4
	Existing Conditions			**83**	**8.5**

Table 1-B-1. Curve Number for existing and proposed conditions, as well as off-site area.

	Segment	Flow Type	Surface Cover	Mannings n	Length ft	Slope %	Veloctiy ft/s	Tt min
Existing Condition Site Only	A_1 to B_1	Overland - sheet	Cultivated Residue Cover >20%	0.17	100	1.2		14.0
	B_1 to C_1	Overland - shallow conc	Cultivated - Minimum Tillage	0.101	1130	0.5	0.4	52.3
	Total	**Existing**			**1230**			**66.3**
	Segment	**Flow Type**	**Surface Cover**	**Mannings n**	**Length ft**	**Slope %**	**Veloctiy ft/s**	**Tt min**
Proposed Condition Site Only	A_2 to B_2	Overland - sheet	Dense Grass	0.24	100	1.0		19.9
	B_2 to C_2	Overland - shallow conc	Grass Swale	0.050	380	1.0	1.6	3.9
	C_2 to D_2	Pipe - storm drain (18")	Pipe	0.013	40	0.6	3.3	0.2
	D_2 to E_2	Pipe - storm drain (18")	Pipe	0.013	260	0.3	3.3	1.3
	E_2 to F_2	Pipe - storm drain (24")	Pipe	0.013	260	0.2	3.9	1.1
	F_2 to G_2	Pipe - storm drain (30")	Pipe	0.013	170	0.3	4.6	0.7
	Total	**Proposed**			**910**			**27.0**

Table 1-B-2a. Time of Concentration (Tc) for existing and proposed conditions, as well as drainage from the off-site area.

Peak Discharge Summary (cont'd)
Project: Autumn Knoll Senior Living Residential Development

	Segment	Flow Type	Surface Cover	Mannings n	Length ft	Slope %	Velociiy ft/s	Tt min
Existing Condition Off-site Condition	U_1 to V_1	Overland - sheet	Cultivated Residue Cover >20%	0.17	100	1.5		12.8
	V_1 to W_1	Overland - shallow conc	Cultivated - Minimum Tillage	0.101	1250	0.5	0.4	57.9
	Total	**Existing**			**1350**			**70.7**
Proposed Condition Off-site Condition	**Segment**	**Flow Type**	**Surface Cover**	**Mannings n**	**Length ft**	**Slope %**	**Velociiy ft/s**	**Tt min**
	U_2 to V_2	Overland - sheet	Cultivated Residue Cover >20%	0.17	100	1.5		12.8
	V_2 to W_2	Overland - shallow conc	Cultivated - Minimum tillage	0.101	300	0.7	0.4	11.9
	W_2 to C_2	Overland - shallow conc	Grass Swale	0.050	180	1.0	1.6	1.9
	C_2 to D_2	Pipe - storm drain (18")	Pipe	0.013	40	0.3	3.3	0.2
	D_2 to E_2	Pipe - storm drain (18")	Pipe	0.013	260	0.3	3.3	1.3
	E_2 to F_2	Pipe - storm drain (24")	Pipe	0.013	260	0.3	3.9	1.1
	F_2 to G_2	Pipe - storm drain (30")	Pipe	0.013	170	0.3	4.6	0.6
	Total	**Proposed**			**1310**			**29.8**

Table 1-B-2b. Time of Concentration (Tc) for existing and proposed condition for off-site drainage only.

RI years	P in	Q_{pre} in	Q_{post} in	$Q_{off\text{-}site}$ in	Q_{pre} Ac-ft	Q_{post} Ac-ft	q_{pre} cfs	q_{post} cfs
1	2.42	1.05	1.38	1.00	2.8	3.5	16.1	38.1
2	2.90	1.43	1.81	1.37	3.9	4.6	22.4	50.4
5	3.56	1.99	2.41	1.91	5.4	6.2	31.4	67.7
10	4.07	2.43	2.89	2.35	6.6	7.5	38.6	81.3
25	4.77	3.06	3.55	2.97	8.3	9.3	48.6	100.0
50	5.32	3.56	4.08	3.47	9.6	10.7	56.6	114.7
100	5.89	4.09	4.63	3.99	11.1	12.2	65.0	130.0

Table 1-B-3. Summary runoff depth or volume (Q) and peak discharge (q) for existing (pre-developed) and proposed (post-

Step 5 - Determine Pond Location and Develop Preliminary Geometry to Meet WQv and Peak Discharge Requirements

The proposed location of the stormwater basin (see Figure 1.B.1) reflects several goals for this development project (including appropriate soils). In particular, the wet basin is considered the centerpiece of this development, with "waterfront condos" selling for a premium. The basin will also be over-excavated to provide fill material to raise the elevation of the condo structures. Existing ground elevation at the proposed pond outlet is 829 MSL. As part of this development, a storm sewer will be installed along the county road to convey site runoff to a receiving stream to the west. At the connection point, the storm sewer is 36" and has an invert elevation of 818.5 MSL. [For more information on siting and planning an extended detention basin, see section 2.6.]

The stormwater basin includes a permanent pool, an extended detention volume to protect water quality and stream channel stability, and storage necessary to control the peak discharge rate.

The NPDES Storm Water Permit (OEPA, 2008) specifies a wet extended detention basin must include both a permanent pool (designated PPv below) and an extended detention volume (EDv) equal to 75% of the water quality volume (WQv), with an EDv drawdown time of 24 hours. The permit also requires that the permanent pool contain an additional sediment storage volume equal to 20% of the WQv.

EDv = 0.75* WQv = 0.75*0.53 ac-ft = 0.40 ac-ft = 17,400 ft^3 Equation 1-B-8

PPv $\geq$ (0.75 + 0.2)*WQv = 0.95*0.53 ac-ft = 0.50 ac-ft = 22,000 ft^3 Equation 1-B-9

A plan view of the basin layout (Figure 1-B-4) reflects the following:

- extended detention volume equal to 0.75*WQv
- permanent pool with a minimum volume of (0.75+0.2)*WQv and 6 foot minimum depth
- 4:1 sideslopes for safety and ease of maintenance
- shallow, submerged wetland safety benches around the perimeter
- an emergency spillway constructed in native soil
- 3 storm drain outlets draining subareas within the development site (note: the length to width ratio for each of the two drains at the far end of the basin (draining approximately 90% of the site) exceeds 3:1, whereas the storm drain for the clubhouse/parking lot (drains approximately 10% of the site) was located on the other side of the gazebo peninsula to extend flow pathway to minimize short-cutting.

<u>Set elevations for pond structures</u>

- The pond bottom and riser invert are set at elevation 820 MSL
- A pond drain will be included to facilitate drawdown for maintenance or repairs.

<u>Establish permanent pool and WQv water surface elevations</u>

A stage-area-storage table (Table 1.B.4) reflects the geometry of the stormwater basin (Figure 1.B.4) designed to meet permanent pool, extended detention (EDv) and peak discharge control requirements.

- The permanent pool volume (PPv) of 4.5 ac-ft (surface elevation 826.0) exceeds 0.95*WQv
- The extended detention volume (EDv) of 0.40 ac-ft above the permanent pool has a top elevation of approximately 826.26.

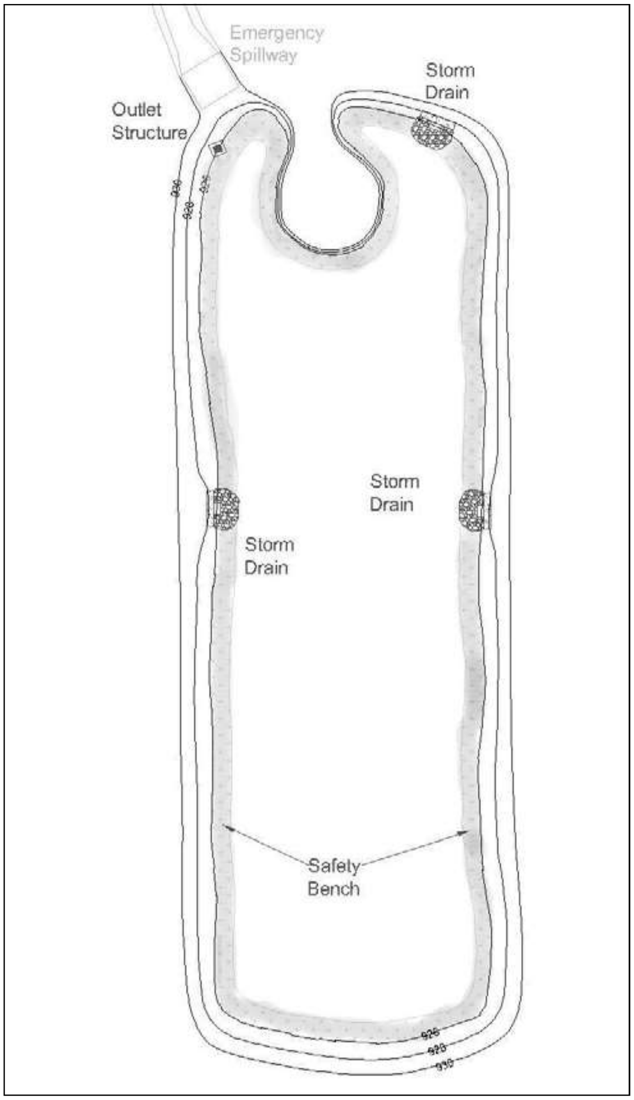

Figure 1.B.4. Preliminary Plan View of Wet Extended Detention Basin (not to scale).

Elevation MSL (ft)	Surface Area (acre)	Average Area (acre)	Incremental Depth (ft)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)	Vol above Perm Pool (ac-ft)
820.0	0.004	-	-	-	-	-
826.0	1.50	0.75	6.0	4.5	4.5	-
826.3	1.53	1.51	0.3	0.4	4.9	0.4
827.0	1.61	1.57	0.7	1.2	6.1	1.6
828.0	1.72	1.66	1.0	1.7	7.8	3.3
829.0	1.83	1.78	1.0	1.8	9.6	5.1
830.0	1.98	1.90	1.0	1.9	11.5	7.0
831.0	2.20	2.09	1.0	2.1	13.6	9.1

Table 1.B.4. Stage-Area-Storage Information for Wet Extended Detention Basin.

Determine outlet geometry for 24-hour drawdown of EDv

The controlling parameters are EDv = 0.40 ac-ft, depth of EDv = 0.26 ft, and minimum drain time, T_d = 24 hours. Note that "the outlet structure for the post-construction BMP must not discharge more than the first half of the WQv in less than one-third of the drain time" (p22, NPDES Storm Water Construction General Permit; OEPA, 2008). This same criterion applies to the EDv.

When a wet detention basin has a large surface area (and thus the EDv depth is small), the designer has a wide variety of outlet options that will meet the two criteria above[3]. In this situation, combining a v-notch weir ("V" depth equal to or exceeding the depth of the EDv) with the peak discharge (critical storm) outlet, the designer was able to simplify and optimize the outlet while meeting both EDv criteria (see Figure 1.B.5) and peak discharge criteria (Table 1.B.5).

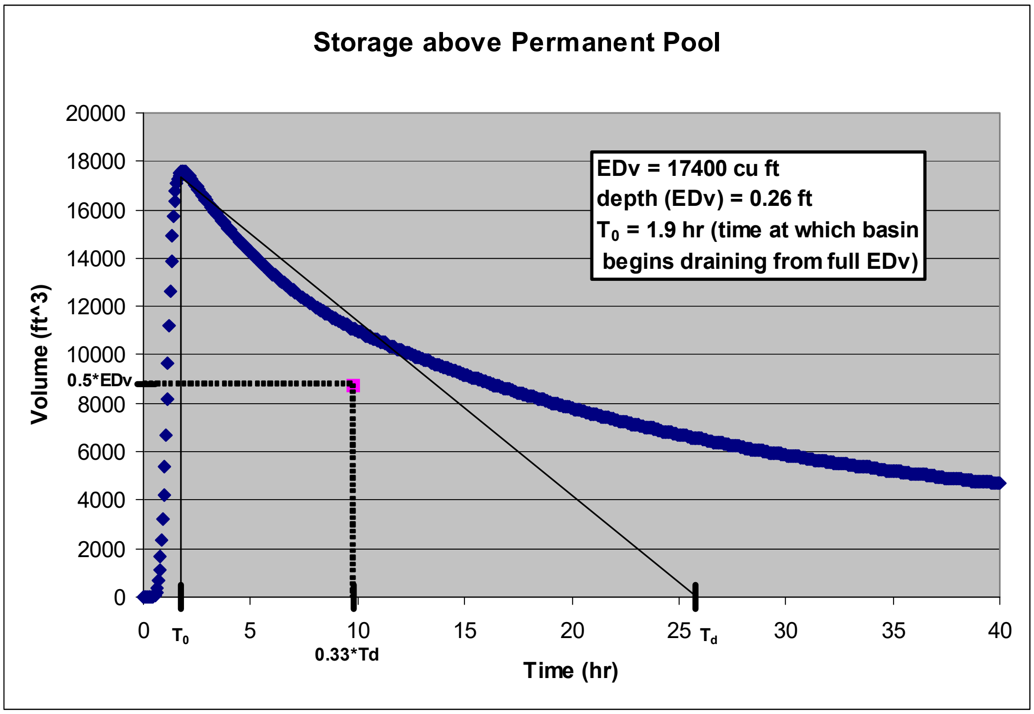

Figure 1.B.5. Wet Extended Detention Basin - Drawdown from Full EDv.

Determine storage and outlet configuration to meet peak discharge requirements

As noted under Step 2, this wet detention basin is designed to meet the Critical Storm Method (CSM) for peak discharge control as well as the WQv requirement. Storage volume must be incorporated that, with appropriate outlet design, will allow the basin to meet the following requirements:

- The peak rate of discharge from the post-construction 5-year, 24-hour event (the *critical storm*) must be less than the existing (pre-development) 1-year, 24-hour discharge peak rate
- The peak rate of discharge from the post-construction 10-, 25-, 50- and 100-year, 24-hour events must be no more than the existing (pre-development) discharge peak rate for the corresponding recurrence interval events

Proprietary stormwater modeling software was used to try a combination of stage-storage and outlet

[3] The methodology laid out in the Ohio NPDES Post Construction Q&A Document (Guidance Regarding Post-Construction Storm Water Management Requirements of Ohio; Ohio EPA, 2007) item #22.is a good starting point for selecting the EDv orifice size because it will always meet the two drawdown requirements: (1) the specified minimum drain time, Td; and (2) the outlet must not discharge more than the first half of the WQv (or EDv) in less than one-third of the drain time. A larger or smaller orifice should be considered if it will help meet other environmental, cost, or maintenance goals but must be tested for the two drawdown criteria.

configurations until the *critical storm* requirement was satisfied while considering the following:

- use best practices outlined in Section 2.6 of the Rainwater and Land Development manual
- optimize cut/fill and grading
- meet safety and aesthetic goals for the “lake” and waterfront properties

The resulting detention basin geometry is presented in Figure 1.B.4 and Table 1.B.4. The resulting outlet configuration is shown in Figure 1.B.6.

The outlet structure (see Figure 1.B.6) consists of a 3 ft by 3 ft concrete catch basin (e.g., ODOT No. 2-3) with invert at 820 MSL and 2.5’x2.5’ iron grate at 828 MSL. The following comprise the outlets:

- A 30” wide orifice combined with a V-notch weir (invert 826 MSL) that controls release of both the extended detention volume (EDv) and the *critical storm* (5-year, 24-hour)
- 2.5’x2.5’ iron grate (effective orifice area 490 sq. in.; invert 828 MSL) for maintenance access and to help manage discharge between the 10-yr and 100-yr, 24-hr events

The catch basin will be connected - using a 30” diameter conduit - to the 36” diameter storm sewer at the road along the north property boundary. A tailwater analysis was performed using the modeling software and the storm sewer’s design elevation (invert at 818.5 MSL; 10-yr full pipe flow at 821.5 MSL) and assumed elevation for the 100-yr event (827.5 MSL).

In addition, this design includes an emergency spillway excavated into native soil with the following characteristics:

- Invert (crest) elevation of 829.2 MSL
- Spillway crest perpendicular to flow
- Level section length of 25 ft, weir length (i.e., width of crest perpendicular to flow) of 25 ft
- Exit channel flows to road ditch at elevation 827.5 MSL
- With all other outlets blocked and starting from the permanent pool elevation of 826 MSL, will safely convey the 100-yr, 24-hr event

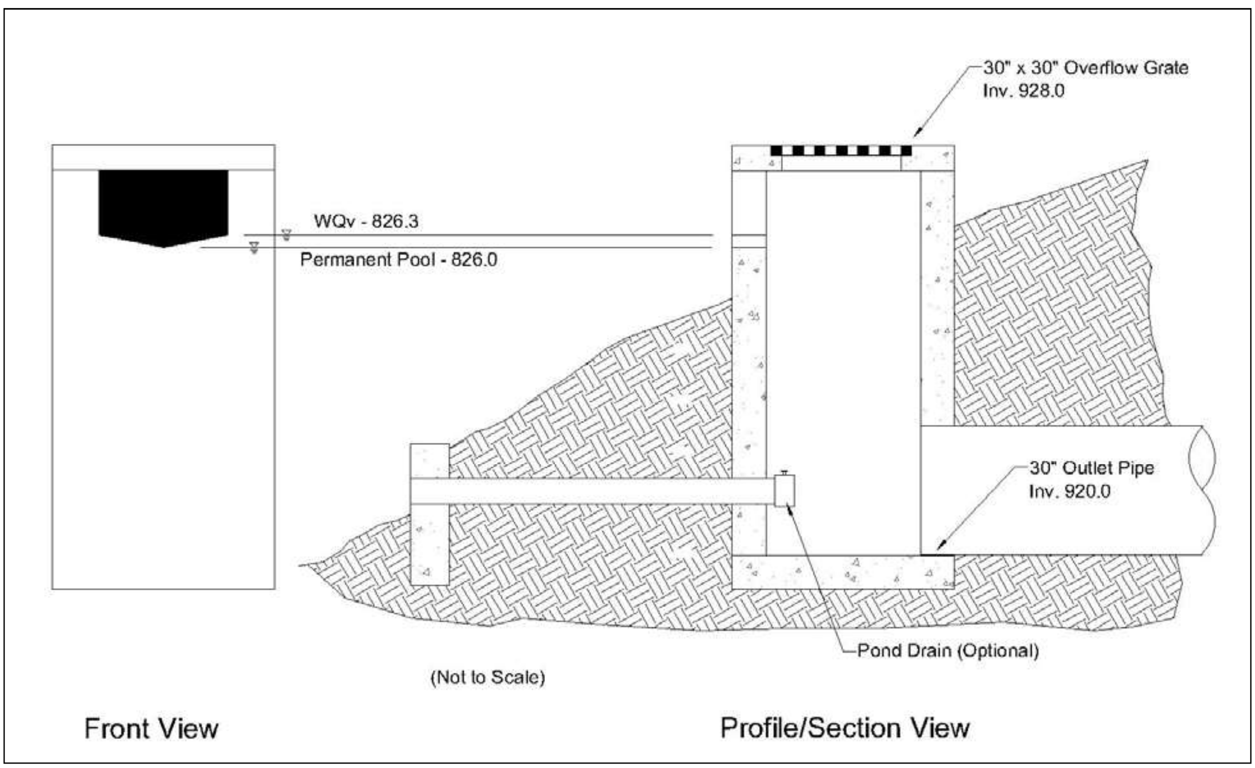

Figure 1.B.6. Outlet Configuration for Wet Extended Detention Basin (not to scale).

Step 6 - Check Design to Ensure All Requirements Are Met

From full EDv, check that EDv meets minimum 24 hour drain time, and discharges no more than 1/2 the extended detention volume, 0.5*EDv (9150 ft^3), in the first 1/3 of the drain time, 0.33*T_d (8 hr). This requirement is met and illustrated in Figure 1.A.5[4].

Check peak discharge for all events (see Table 1.B.5).

RI years	P in	$q_{post\text{-}in}$ cfs	Allowed $q_{post\text{-}out}$ cfs	Estimated $q_{post\text{-}out}$ cfs
1	2.42	38.1	16.1	7.2
2	2.90	50.4	16.1	11.1
5	3.56	67.7	16.1	15.9
10	4.07	81.3	38.6	24.0
25	4.77	100.0	48.6	32.3
50	5.32	114.7	56.6	37.3
100	5.89	130.0	65.0	41.8

Table 1.B.5. Critical Storm Method Peak Discharge Check.

[4] Note - Through trial and error, it was determined using a constant intensity 1-hour rainfall event of 0.83” depth in the hydrologic model would raise the water surface elevation of the wet basin to 826.26 providing a just-full EDv of 0.40 ac-ft (17,400 ft^3) above permanent pool, allowing evaluation of the drawdown from a full EDv (Figure 1.B.5). The depth of rainfall event necessary to just fill the EDv or WQv for other stormwater basins using CN methodology will vary based on watershed characteristics, pond geometry and outlet configuration, but can be determined through trial and error.

Section C: Extended Detention Wetland Basin

This design example illustrates the design of a extended detention wetland basin that provides water quality treatment and peak discharge control for a single family residential development, consisting of 101 residential lots on 46.0 acres (parcel and drainage area). The layout of the Beech Ridge subdivision is shown below in Figure 1.C.1.

The impervious area of the site at completion of construction is estimated to be 13.2 acres. The pre-developed site soils and flow paths are shown in Figure 1.C.2, while the post-developed flow path (limited to that used for calculations) is shown in Figure 1.C.3. This example assumes that the local community has adopted the Critical Storm Method criteria to control peak discharges.

Figure 1.C.1. Beech Ridge Subdivision Site Plan.

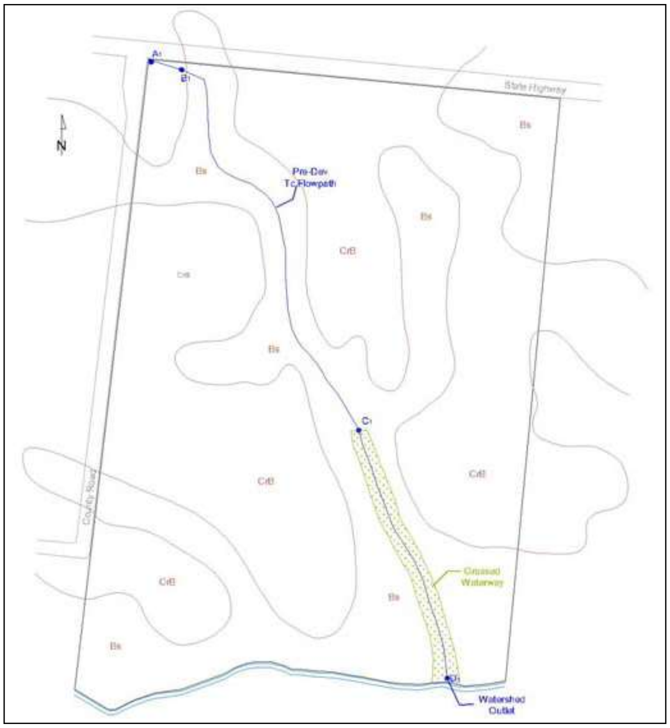

Figure 1.C.2. Pre-Development and Soils and Flow Path.

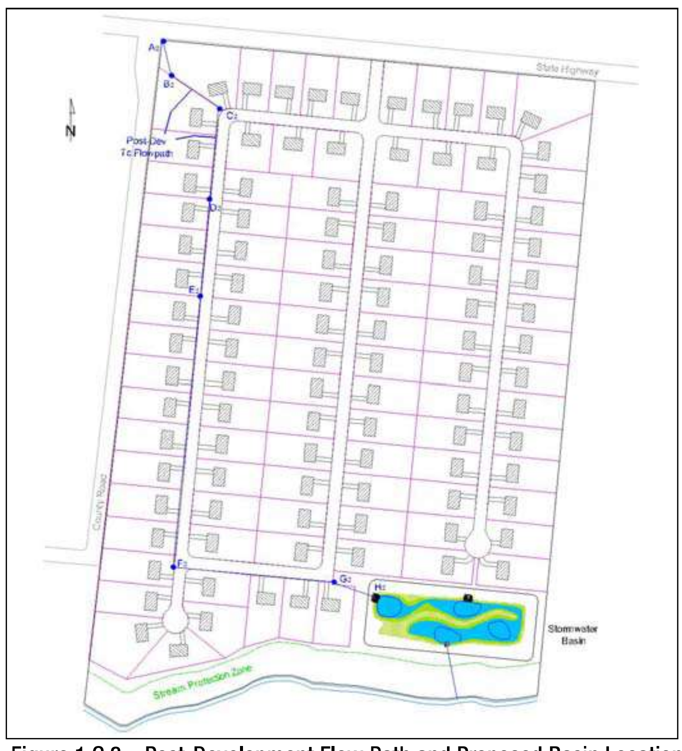

Figure 1.C.3. Post-Development Flow Path and Proposed Basin Location

<u>Site Data</u>
Zoning: Residential, 16,000 ft^2 minimum lot size (0.37 ac)
Total Drainage Area (A) = 46.0 ac
Estimated Impervious Area = 13.2 ac
Pre-Development Soil Types: 60% HSG-C, 40% HSG-B/D

<u>Summary Hydrologic Data</u>
WQv = 0.69 ac-ft

	Pre	Post
CN =	79	86
Tc =	69.5 min	25.4 min

Calculation of Preliminary Stormwater Storage Volumes and Peak Discharges

Step 1 - Calculate Water Quality Volume (WQv)

The water quality volume (WQv) is a post-construction stormwater control requirement in Ohio (OEPA-CGP, 2008). The WQv is determined according to the following equation:

$$WQv = C*P*A \qquad \text{Equation 1.C.1}$$

where:

C = runoff coefficient

P = 0.75 inch precipitation depth

A = drainage area

For open water, C = 1. At this site, the surface area of the detention basin at full WQv is estimated to be 1.2 acres.

For the remainder of the site, the runoff coefficient can either be selected from Table 1 of the NPDES Storm Water Permit, or calculated using the following equation:

$$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \qquad \text{Equation 1.C.2}$$

where i is the fraction of post-construction impervious surface.

For this site, total impervious area = 13.2 acres from a drainage area of 44.8 acres (i.e., total site drainage area - surface area of detention basin).

$$i = 13.2/44.8 = 0.295 \qquad \text{Equation 1.C.3}$$

$$C = 0.858(0.295)^3 - 0.78(0.295)^2 + 0.774(0.295) + 0.04 = 0.22 \qquad \text{Equation 1.C.4}$$

Therefore, the WQv is:

$$WQv = [(1.0 * 0.75\text{ in} * 1.2\text{ ac}) + (0.22 * 0.75\text{ in} * 44.8\text{ ac})] * (1\text{ ft}/12\text{ in}) \qquad \text{Equation 1.C.5}$$

$$= \underline{0.69\text{ ac-ft}}$$

$$= 30{,}100\text{ ft}^3$$

Step 2 - Compute Peak Discharge Requirements

Note: Peak discharge control is typically regulated through local entities (e.g. stormwater district, municipal, township or county governments). The state of Ohio recommends use of the Critical Storm Method[1] for peak discharge control, but the requirements will vary by community. Check local stormwater regulations to determine which peak discharge control method you must use.

This example uses the SCS Curve Number Methodology to perform hydrologic calculations. TR-20, HEC-HMS or other software that uses SCS procedures should provide similar results.

Tables 1.C.1 and 1.C.2 summarize the inputs necessary to determine the curve number (CN) and time of concentration (Tc) for the existing (pre-development) and proposed (post-development) conditions. The property receives no runoff from off-site. Table 1.C.3 summarizes the existing and proposed runoff depths and peak discharges for the 1-year, 24-hr through 100-year, 24-hr rainfall events.

The *critical storm* is determined from the percent increase in runoff volume from the 1-year, 24-hr storm for the proposed (post-developed) conditions when compared to the existing (pre-developed) conditions (Goettemoeller et al., 1980):

$$Percent\ Increase = \frac{Q_{post} - Q_{pre}}{Q_{pre}} x\, 100$$

Equation 1.C.6

From Table 1.C.3, the percent increase in the 1-year, 24-hr runoff for the proposed development is:

$$Percent\ Increase = \frac{0.98 - 0.62}{0.62} x\ 100 = 58.1\%$$

Equation 1.C.7

For a percentage increase between 50% and 100%, the critical storm for peak discharge control is the 10 -year, 24-hr event—that is, the 10-year, 24-hr post-developed peak discharge must be less than the existing (pre-developed) 1-year, 24-hr peak discharge. These values are shown in bold type in Table 1.C.3. In addition, the post-developed peak discharge from the 25, 50 and 100 year events must be less than the existing peak discharge for each of those events.

Step 3 - Identify Other Local Development Criteria/Requirements

This site is located within a community that has incorporated a stream corridor protection requirement (i.e., stream setback) in its subdivision regulations. Review of the regulations has determined that the stream protection zone at this site extends 100 ft from the ordinary high water mark of the adjacent stream. This protection zone is noted on the map in Figure 1.C.1. All construction activities, including the wetland stormwater basin and embankment, must be outside of the stream protection zone. Also note this stream protection area, since it does not drain to the detention facility, was excluded from the hydrologic analysis.

[1] The Critical Storm Method is a set of criteria for controlling the peak discharge of stormwater from large storm events (1 - 100 yr recurrence interval) recommended by ODNR-DSWC since 1980. See Goettemoeller, R.L., D.P. Hanselmann, and J.H. Bassett. 1980. Ohio Stormwater Control Guidebook. Ohio Department of Natural Resources, Division of Soil and Water Districts, p47.

Peak Discharge Summary
Project: Beech Ridge Subdivision

	Cover Description	Soil Name	Hydrologic Group	Drainage Y/N	CN	Area (acres)
Existing Condition Site Only	Row crop, SR + CR (good condition)	Crosby	C		82	27.6
	Row crop, SR + CR (good condition)	Brookston	B/D	Y	75	18.4
	Pre-development Conditions - All				**79**	**46.0**
	Cover Description	**Soil Name**	**Hydrologic Group**		**CN**	**Area (acres)**
Proposed Condition Site Only	Impervious Area				98	13.2
	Open space (good condition)	Crosby	D		80	18.5
	Open space (good condition)	Brookston	D		80	12.3
	Open Water				98	2.0
	Post-development Conditions - All				**86**	**46.0**

Table 1.C.1. Curve Number (CN) for existing (pre-developed) condition.

	Segment	Flow Type	Surface Cover	Mannings n	Length ft	Slope %	Velocity ft/s	Tt min
Existing Condition Site Only	A_1 to B_1	Overland - sheet	Min Tillage	0.17	100	1.2		14.8
	B_1 to C_1	Overland - shallow conc	Min Tillage	0.1	1250	0.75	0.44	47.6
	C_1 to D_1	Overland - shallow conc	Grassed waterway	0.05	750	1.2	1.8	7.1
	Total	Pre-developed			2100			69.5
	Segment	**Flow Type**	**Surface Cover**	**Mannings n**	**Length ft**	**Slope %**	**Velociiy ft/s**	**Tt min**
Proposed Condition Site Only	A_2 to B_2	Overland - sheet	Grass	0.24	100	1.5		17.8
	B_2 to C_2	Overland - shallow conc	Grassed waterway	0.05	160	1.5	2.0	1.3
	C_2 to D_2	Pipe - storm drain (15")	Pipe	0.013	250	0.5	3.7	1.1
	D_2 to E_2	Pipe - storm drain (18")	Pipe	0.013	270	0.5	4.2	1.1
	E_2 to F_2	Pipe - storm drain (24")	Pipe	0.013	750	0.5	5.1	2.5
	F_2 to G_2	Pipe - storm drain (30")	Pipe	0.013	450	0.5	5.9	1.3
	G_2 to H_2	Pipe - storm drain (36")	Pipe	0.013	130	0.5	6.7	0.3
	Total	Post-developed			2150			25.4

Table 1.C.2. Time of Concentration (Tc) for existing (pre-developed) and proposed (post-developed) condition.

RI years	P in	Q_{pre} in/acre	Q_{post} in/acre	q_{pre} cfs	q_{post} cfs
1	2.17	0.62	0.98	12.1	42.8
2	2.59	0.90	1.32	18.3	58.0
5	3.18	1.32	1.82	28.1	80.4
10	3.67	1.70	2.25	36.7	99.4
25	4.35	2.25	2.87	49.2	126.2
50	4.91	2.72	3.38	59.9	148.4
100	5.50	3.24	3.94	71.3	171.9

Table 1.C.3. Summary runoff depth (Q) and peak discharge (q) for existing (pre-developed) and proposed (post-developed) condition with critical storm (bold type).

Step 4 - Determine if the Development Site and Soils Are Appropriate for the Use of an Extended Detention Wetland Basin

The site drainage area is 46.0 acres. Brookston and Crosby soils are suitable for creation of an extended detention wetland. The subsoil is silty clay loam derived from high-lime glacial till and has slow permeability. This subsoil is suitable material for construction of the embankment for the stormwater basin.

It is known that subsurface tiles currently drain the proposed property. All tiles need to be removed from the wetland basin site[2].

Step 5 - Determine Pond Location and Develop Preliminary Geometry to Meet WQv and Peak Discharge Requirements

The proposed location of the stormwater basin (see Figure 1.C.3) reflects the best combination of characteristics (landscape position, access to outlet, minimize earth moving, appropriate soils, etc.) for siting the basin. Existing ground elevation at the proposed pond outlet is 907 MSL. The invert of the receiving stream at the proposed discharge point is 896 MSL.

The basin will be designed to include a permanent pool, an extended detention volume equivalent to the WQv, and the storage necessary to control the peak discharge rate. [For more information on siting and planning a wetland basin, see section 2.6.]

An analysis of site hydrology (drainage area/wetland surface area ratio >>20, HSG-D soil with seasonal high water table, etc.) has determined that a permanent pool equivalent to the WQv (~0.69 ac-ft) up to 2 ac-ft should be sufficient to maintain basic wetland hydrology and function. In addition, an additional sediment storage volume equal to 20% of the WQv (0.2*WQv = 0.2*0.69 = 0.14 ac-ft) is added to the permanent pool with this volume concentrated in the forebay.

A preliminary plan view of the basin layout (Figure 1.C.4) reflects the following:

- permanent pool (includes forebay and outlet micropool) with a volume in excess of 1.2*WQv
- permanent pool forebay equal to 0.2* WQv and a minimum depth of 3 ft
- permanent micropool at outlet with a minimum depth of 3 ft
- total area of deep pools (including forebay and outlet mircropool) representing between 20 and 25 percent of total permanent pool surface area with deep pools interspersed through wetland to provide refugia and wetland function during drought periods - depth of deep pools should range between 18 and 36 (or more) inches[3]
- balance of permanent pool with average depth of 0.75 ft, and range of depths from 6” to 18”
- a low constructed peninsula, with an elevation approximately 1 ft above the extended detention (WQv) storage volume, to extend the flow path and minimize short-circuiting during the WQv event
- maximum 4:1 side slopes for safety and maintenance
- an emergency spillway constructed in native soil (i.e., not located in the constructed embankment)

Note: The high organic matter topsoil should be removed and stockpiled before excavation and construction of the wetland, and then replaced on peninsulas and benches.

[2] Functional drainage systems are essential for the productivity of agriculture in much of Ohio, and to prevent flooding of upgradient property. It is the responsibility of the developer to maintain drainage infrastructure (surface and subsurface drainage mains) disrupted by construction activities. As an example, if a subsurface tile main conveys water from upgradient properties, that main should be protected or re-routed to maintain the same drainage capacity.

[3] Recent guidance from North Carolina (Hunt et al, 2007) recommends “deep pools (including the forebay) should occupy between 20 and 25 percent of the total wetland surface area”. For most wetlands this will result in a permanent pool volume (ac-ft) between about 1.1 and 1.3 times the surface area (acres) of the permanent pool.

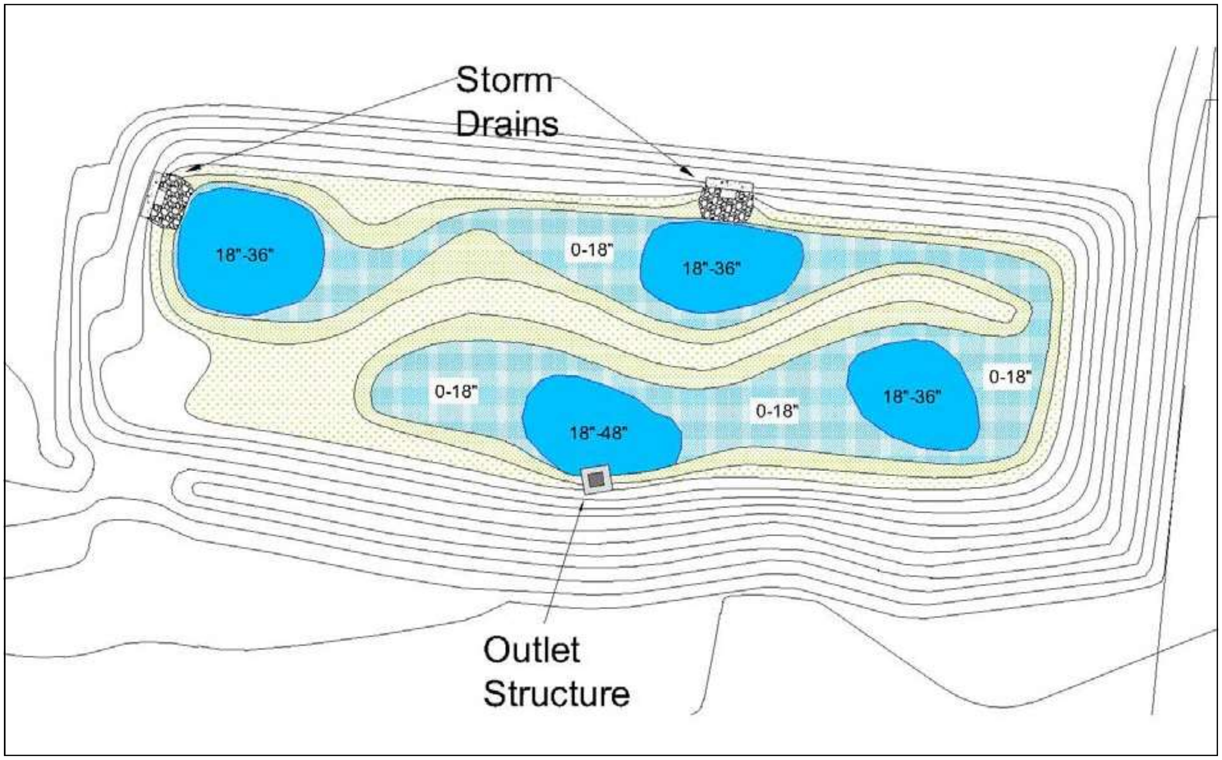

Figure 1.C.4. Preliminary Plan View of Wetland (not to scale).

Elevation MSL (ft)	Surface Area (acre)	Average Area (acre)	Incremental Depth (ft)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)	Vol above Perm Pool (ac-ft)
900.0	0.08	-	-	-	-	-
902.5	0.16	0.12	2.5	0.30	0.30	-
903.0	0.24	0.20	0.5	0.10	0.40	-
903.5	0.48	0.36	0.5	0.18	0.58	-
904.0	0.74	0.61	0.5	0.31	0.89	-
904.7	1.24	0.99	0.7	0.69	1.58	0.69
905.0	1.50	1.37	0.3	0.41	1.99	1.10
906.0	1.60	1.55	1.0	1.55	3.54	2.65
907.0	1.71	1.65	1.0	1.65	5.19	4.30
908.0	1.82	1.76	1.0	1.76	6.95	6.07
909.0	1.93	1.87	1.0	1.87	8.82	7.94
910.0	2.05	1.99	1.0	1.99	10.81	9.93
911.0	2.20	2.13	1.0	2.13	12.94	12.05

Table 1.C.4. Stage-Area-Storage Information for Wetland Basin.

Set elevations for pond structures

- The basin bottom is set at elevation 900.0
- To allow gravity flow for the pond drain, set the riser invert at 898.0
- The outfall at the receiving stream has invert elevation 896.5

Set permanent pool and WQv water surface elevations

A stage-area-storage table (Table 1-4) reflects geometry of the stormwater wetland basin (Figure 1-3) designed to meet permanent pool, extended detention WQv and peak discharge control requirements.

- To meet NPDES Construction Stormwater Permit minimums, the permanent pool, surface elevation 904.0, is sized to exceed 1.2*WQv = 1.2*0.69 ac-ft = 0.83 ac-ft (see footnote below)
- The extended detention WQv of 0.69 ac-ft above permanent pool has a top elevation of approximately 904.7

Calculate required orifice size for 24-hour drawdown of WQv

The controlling parameters are WQv = 0.69 ac-ft, depth of WQv = 0.7 ft, and minimum drain time, t_d, of 24 hours. Note that "the outlet structure for the post-construction BMP must not discharge more than the first half of the WQv in less than one-third of the drain time" (p22, NPDES Storm Water Construction General Permit).

The average discharge rate for the WQv is:

$$Q_{avg} = \frac{WQv}{t_d} = \frac{(0.69\ ac\cdot ft)\left(\frac{43560\ \ ft^2}{1\ ac}\right)}{(24\ hr)\left(3600\ \ \frac{s}{hr}\right)} = 0.35\ cfs$$ Equation 1-C-8

The discharge equation for an orifice is:

$$Q=ca\sqrt{2gh}$$ Equation 1-C-9

By rearranging, we can estimate needed orifice area:

$$a = \frac{Q}{c\sqrt{2gh}}$$ Equation 1-C-10

Using an orifice coefficient, c = 0.6, and average head, h = d/2 = (0.7 ft)/2 = 0.35 ft, the required orifice size is:

$$a = \frac{0.35\frac{ft^3}{s}}{0.6\sqrt{2(32.2\frac{ft}{s^2})(0.35\,ft)}} = 0.12\ ft^2$$ Equation 1-C-11

Resulting in an orifice diameter of:

$$d = \left(\frac{4a}{3.14}\right)^{0.5} = \left[\frac{4(0.12\ ft^2)}{3.14}\right]^{0.5} = 0.39\ ft \times \frac{12}{1\ ft} = 4.7\ in$$ Equation 1-C-12

This estimate is a good starting point for selecting the WQv or EDv orifice size because it will always meet the two drawdown requirements: (1) the specified minimum drain time, Td; and (2) the outlet must discharge less than the first half of the WQv in the first one-third of the drain time (8 hours in this case). A larger or smaller orifice should be considered if it will help meet other environmental, cost, or maintenance goals but must be tested for the two drawdown criteria. In this situation, trial and error showed that a 6.0" diameter orifice will meet the above two drawdown requirements (see Figure 1.C.4) and will be used as the WQv outlet.

Determine storage and outlet configuration to meet peak discharge requirements

As noted under Step 2, this wetland basin is designed to meet the Critical Storm Method (CSM) for peak discharge control as well as the WQv requirement. Additional storage volume must be added that, with appropriate outlet design, will allow the basin to meet the following requirements:

- The peak rate of discharge from the post-construction 10-year, 24-hour event (the *critical storm*) must be released at the existing (pre-development) 1-year, 24-hour discharge rate
- The peak rate of discharge from the post-construction 25-, 50- and 100-year, 24-hour events must be released at the existing (pre-development) discharge rate for the corresponding recurrence interval events

Proprietary stormwater modeling software was used to try a combination of stage-storage and outlet configurations until the Table 1.C.5 requirements were satisfied while considering the following:

- maximize wetland function
- minimize the "footprint" of the basin
- optimize cut/fill

RI years	P in	$q_{post\text{-}in}$ cfs	Allowed $q_{post\text{-}out}$ cfs
1	2.17	42.7	12.1
2	2.59	57.9	12.1
5	3.18	80.2	12.1
10	3.67	99.1	12.1
25	4.35	125.9	49.2
50	4.91	148.0	59.9
100	5.50	171.4	71.3

Table 1.C.5. Critical Storm Method Peak Discharge Requirements.

The resulting wetland basin geometry is presented in Figure 1.C.3 and Table 1.C.4. The resulting outlet configuration is shown in Figure 1.C.5.

The outlet structure consists of a 4 ft by 4 ft concrete catch basin (e.g., ODOT No 2-4) with invert at 899 MSL and 3.7'x3.7' iron grate at 908.33 MSL. The following comprise the outlets:

- 36" barrel outlet with invert at 899 MSL
- 6.0" extended detention (WQv) orifice (invert 904 MSL) with submerged entrance
- Two (2) 12" diameter orifices (invert 904.7 MSL) that control release of the *critical storm* (10-year, 24-hour)
- Four 36" L x 9" H rectangular orifices (invert 907.25 MSL) and 3.7'x3.7' iron grate (invert 908.33) with 868 in^2 of clear opening area that control release of the 25- through 100-year, 24-hour events

In addition, this design includes an emergency spillway excavated into native soil that has the following characteristics:

- Invert (crest) elevation of 909.3 MSL
- Level section length of 25 ft, weir length (i.e., crest width) of 30 ft
- Spillway crest perpendicular to flow
- Exit channel aligned with level section well beyond downstream toe of dam, and a 4 percent slope
- With all other outlets blocked and starting from the permanent pool elevation of 904 MSL, will safely convey the 100-yr, 24-hr event with 1 ft freeboard from top of embankment

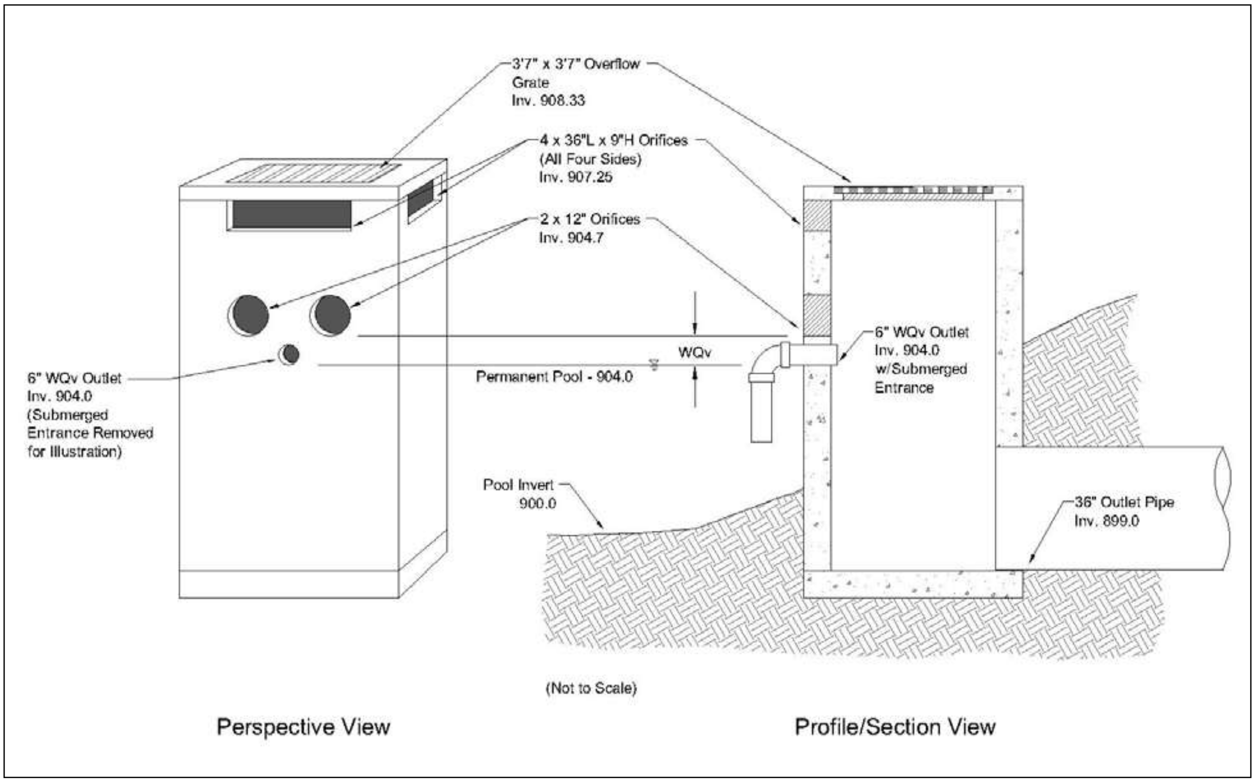

Figure 1.C.5. Outlet configuration for Wetland Basin (not to scale).

Step 6 - Check Design to Ensure All Requirements Are Met

From "brimfull", check that WQv meets minimum 24 hour drain time, and discharges no more than 1/2 the water quality volume, 0.5*WQv (= 15,550 ft^3), in the first 1/3 of the drain time, 0.33*T_d (8 hr). Figure 1.C.4 shows the wetland basin meets this requirement.

Check peak discharge for all events. Table 1.C.6 shows the wetland basin meets the peak discharge requirements.

RI years	P in	$q_{post\text{-}in}$ cfs	Allowed $q_{post\text{-}out}$ cfs	Estimated $q_{post\text{-}out}$ cfs
1	2.17	42.7	12.1	5.1
2	2.59	57.9	12.1	7.6
5	3.18	80.2	12.1	10.2
10	3.67	99.1	12.1	12.1
25	4.35	125.9	49.2	26.5
50	4.91	148.0	59.9	42.7
100	5.50	171.4	71.3	62.4

Table 1.C.6. Critical Storm Method Peak Discharge Check.

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