

3.0 Hydrology

3.1 Symbols and Definitions

To provide consistency within this chapter as well as throughout this Manual, the symbols listed in Table 3-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equation.

Symbol	Definition	Units
A	Drainage area, cross sectional area	acres, ft ²
B _f	Baseflow	ac-ft
CN	SCS Curve Number	-
CN _m	Modified SCS Curve Number	-
CP _v	Channel protection volume	ac-ft
E	Evaporation	ft
E _t	Evapotranspiration	ft
G _h	Hydraulic gradient – pressure head/distance	-
I	Percent of impervious cover	%
I	Inflow	ac-ft
I	Infiltration	ac-ft/day
I _a	Initial abstraction	in
K _h	Saturated hydraulic conductivity	ft/day
P	Rainfall	inches
Q, q	Peak inflow or outflow rate, runoff	cfs, in
Q _f	Overflow	ft
Q _p	Peak rate of discharge	cfs
R _v	Volumetric runoff coefficient	-
R _o	Runoff	ac-ft
S	Potential maximum retention	in
t _c	Time of concentration	hours
V	Pond volume	ac-ft
V _r	Runoff volume	ac-ft
WQ _p	Water quality peak discharge	cfs
WQ _v	Water quality protection volume	ac-ft, in

3.2 Runoff Volume and Peak Flow

3.2.1 Introduction

The following is a discussion of the hydrologic methods needed to calculate the water quality protection volume (WQ_v), channel protection volume (CP_v), and large flood event peak flows (Q₁₀), (Q₂₅), (Q₅₀), etc. For more details on hydrologic analysis see Chapter 3, Hydrology, in the Charlotte-Mecklenburg Storm Water Design Manual.

BMPs should be designed to treat and/or control the runoff from the 1-inch storm event (WQ_v) for water quality, and the 1-year, 24-hour (CP_v) for volume control, and 10-year, 6-hour storm event for flood control. More severe storm events may also be required to be analyzed. Table 3-2 summarizes the design storm events for various types of storm drainage systems.

Table 3-2: Summary of Design Standards

Design feature	Design storm
BMPs for water quality	1-inch, 6-hour
BMPs for channel protection	1-year, 24-hour
BMPs for flood protection	10- and 25-year, 6-hour
Closed pipe and channels	10-year, 6-hour
Culverts (subdivision streets)	25-year, 6-hour
Culverts (thoroughfare roads)	50-year, 6-hour
Building footprints	100-year, 6-hour or 100-year, 24-hour on FEMA regulated streams

Conventional SCS methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2 inches. This discrepancy in estimating runoff and discharge rates can lead to several potential design challenges including undersizing of BMPs, inadequately sized diversion structures, and undersized outlet pipes, channels, etc. Therefore, a modified SCS procedure is required for development for the 1-inch, 6-hour storm event hydrograph.

For accurate runoff volume computational results, the 1-inch storm event runoff volume should be computed using methods developed by Schueler, and the 1-year and larger storm event runoff volumes should be computed using methods developed by SCS. The Schueler methods that are published can be used only to compute total runoff volume and cannot be used to develop a hydrograph including a peak flow. In order to overcome the differences of runoff volume results provided by the two methods, a method is presented to adjust the site curve number so that 1-inch storm event runoff volumes computed by SCS methods match the runoff volumes that are computed by methods developed by Schueler.

3.2.2 Water Quality Protection Volume and Peak Flow Calculation

Water Quality Protection Volume Calculation

The Water Quality Protection Volume (WQ_v) is the runoff volume required to be treated when designing a BMP to meet specific pollutant removal targets. This is achieved by intercepting and treating the runoff from a 1-inch rainfall event. The Water Quality Protection Volume (WQ_v) is calculated by method developed by Schueler by multiplying 1.0 inches of rainfall by the volumetric runoff coefficient (R_v) and the drainage area. R_v is defined as:

$$R_v = 0.05 + 0.009(I) \quad (3.1)$$

Where: I = percent of impervious cover of the drainage basin (%)

Using the 1.0 inches of rainfall, the WQ_v is calculated using the following formula:

$$WQ_v = 1.0R_vA/12 \quad (3.2)$$

Where: WQ_v = water quality protection volume (acre-feet)

R_v = volumetric runoff coefficient

A = total drainage area (acres)

WQ_v can be expressed in inches (if the designer wants to compare the result to runoff hydrograph volumes): using the following formula.

$$WQ_v = 1.0(R_v) \quad (3.3)$$

Where: WQ_v = water quality protection volume (inches)

Peak Flow and Runoff Hydrograph Calculation

The peak rate of discharge (Q_p) and/or runoff hydrograph shape for the Water Quality Protection Volume storm is needed for the sizing of most BMPs such as sand filters, bioretention, infiltration trenches, etc. In

addition, the peak rate of discharge (Q_p) and/or runoff hydrograph shape is needed if the designer elects to treat only the Water Quality Runoff Volume (WQ_v) with one BMP such as bioretention and treat the larger storm events with another BMP such as an extended detention basin.

A balanced 6-hour storm event is chosen for use in Mecklenburg County. The method relies on the water quality protection volume and calculation of a modified Curve Number for use in the standard SCS method. A description of the calculation procedure is presented below.

Step 1 Using the WQ_v , a modified Curve Number (CN) is computed utilizing the following equation:

$$CN_m = 1000/[10 + 5P + 10WQ_v - 10(WQ_v^2 + 1.25WQ_vP)^{0.5}] \quad (3.4)$$

Where: P = rainfall, in inches (use 1.0 inches)
 WQ_v = Water Quality Volume, in inches ($1.0R_v$)

Step 2 Once a value for CN_m is computed, the time of concentration (t_c) is computed using the standard procedures contained in Chapter 3, Hydrology, in the Charlotte-Mecklenburg Storm Water Design Manual.

Step 3 Using the computed CN_m , t_c and drainage area (A), in acres, the peak discharge (Q_p) can be calculated using the methods outlined in Chapter 3, Hydrology, in the Charlotte-Mecklenburg Storm Water Design Manual. The rainfall values for the 1-inch, 6-hour storm event using the PH record option for a HEC-1 model are shown in Table 3-3.

Table 3-3 1-inch, 6-hour – Balanced Storm Event PH Records

PH records						
Interval	5 min	15 min	60 min	2 hour	3 hour	6 hour
Rainfall	0.188	0.371	0.640	0.769	0.855	1.0 inch

PH records should not be used when implementing a HEC-1 model for smaller sites where a computational interval of less than 4 minutes is necessary. The PI records shown in Table 3-4 can be used for models where a computation interval less than 4 minutes is appropriate. Note that the incremental rainfall data is reported on a 5 minute interval, so the user should use an IN record to set the data read into the HEC-1 model as a 5 minute interval.

Table 3-4 1-inch, 6-hour – Balanced Storm Event PI Records

Time	Rain										
Min	Inch										
0	.000	65	.004	130	.009	195	.043	260	.007	325	.004
5	.003	70	.004	135	.010	200	.028	265	.007	330	.004
10	.003	75	.005	140	.011	205	.023	270	.006	335	.004
15	.003	80	.005	145	.012	210	.020	275	.005	340	.003
20	.003	85	.005	150	.013	215	.014	280	.005	345	.003
25	.003	90	.005	155	.019	220	.012	285	.005	350	.003
30	.003	95	.006	160	.022	225	.011	290	.005	355	.003
35	.004	100	.007	165	.025	230	.010	295	.005	360	.003
40	.004	105	.007	170	.039	235	.009	300	.004	365	.000
45	.004	110	.007	175	.050	240	.009	305	.004		
50	.004	115	.008	180	.108	245	.008	310	.004		
55	.004	120	.008	185	.188	250	.008	315	.004		
60	.004	125	.009	190	.075	255	.007	320	.004		

Example

Using the following information that was derived for a typical site using the methods described in the Charlotte-Mecklenburg Storm Water Design Manual, calculate the Water Quality Protection Volume (WQ_v), water quality peak flow (Q_p), and water quality runoff hydrograph.

- Area = 50 acres
- CN = 72
- $t_c = 0.34$ hours
- $t_{lag} = 0.20$ hours
- Total impervious area = 18 acres

Calculate water quality volume (WQ_v)

Compute volumetric runoff coefficient, R_v using Equation 3.1:

$$R_v = 0.05 + 0.009(I) = 0.05 + (0.009)(18/50 \times 100\%) = 0.37$$

Compute water quality volume, WQ_v using Equations 3.2 and 3.3:

$$WQ_v = 1.0(R_v)(A)/12 = 1.0(0.37)(50)/12 = 1.54 \text{ acre-feet}$$

$$WQ_v = 1.0(R_v) = 1.0(0.37) = 0.37 \text{ inches}$$

Calculate water quality peak flow and runoff hydrograph

Compute modified curve number (CN_m) using Equation 3.4:

$$CN_m = 1000/[10 + 5P + 10WQ_v - 10(WQ_v^2 + 1.25WQ_vP)^{0.5}]$$

$$CN_m = 1000/[10 + 5(1.0) + 10(0.37) - 10\{(0.37)^2 + 1.25(0.37)(1.0)\}^{0.5}] = 91.3$$

The following HEC-1 model output provides the final results of the peak runoff and runoff hydrograph exercise.

```

1*****
* FLOOD HYDROGRAPH PACKAGE (HEC-1) *
* JUN 1998 *
* VERSION 4.1 *
* RUN DATE 05MAR07 TIME 17:40:47 *
*****
* U.S. ARMY CORPS OF ENGINEERS *
* HYDROLOGIC ENGINEERING CENTER *
* 609 SECOND STREET *
* DAVIS, CALIFORNIA 95616 *
* (916) 756-1104 *
*****

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X X XXXXXXX XXXXX X
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XXXXXXXX XXXX X XXXXX X
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THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.
 THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
 THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
 NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
 DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
 KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

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1
HEC-1 INPUT PAGE 1
LINE ID.....1.....2.....3.....4.....5.....6.....7.....8.....9.....10
1 ID MECKLENBURG COUNTY BMP DESIGN MANUAL
2 ID ANALYZED BY ABC ENGINEERING
3 ID DATE: OCTOBER 2006
* *****
* TIME SPECIFICATION CARD
4 IT 1 0 0 360
* DIAGRAM
* TIME INTERVAL CARD
5 IN 5 0 0
*
* OUTPUT CONTROL CARD
6 IO 1 0 0
*
7 KK PRE1
* *****

```


Chapter 3 of the Charlotte-Mecklenburg Storm Water Design Manual. The equation to compute runoff volume is:

$$Q = (P - 0.2S)^2 / (P + 0.8S) \quad (3.5)$$

Where:

- Q = accumulated runoff volume, inches
- P = accumulated rainfall (potential maximum runoff), inches
- S = potential maximum soil retention, inches = 1000/CN - 10
- CN = SCS curve number

Note that the SCS curve number that is used to generate runoff volumes for all storm events other than the 1-inch, 6-hour storm event is the standard curve number and not the modified curve number discussed in Section 3.2.2. The same method as described in Section 3.2.2. to compute the entire runoff hydrograph including the peak discharge should be applied.

Table 3-5 SCS Type II Storm Event Distribution PI Records

Time Mins	Rainfall %										
0	0	240	0.0014	480	0.0022	720	0.0951	960	0.0023	1200	0.0013
6	0.001	246	0.0014	486	0.0022	726	0.019	966	0.0023	1206	0.0013
12	0.001	252	0.0014	492	0.0024	732	0.0166	972	0.0022	1212	0.0013
18	0.001	258	0.0015	498	0.0024	738	0.0144	978	0.0023	1218	0.0013
24	0.0011	264	0.0015	504	0.0026	744	0.0122	984	0.0022	1224	0.0013
30	0.001	270	0.0015	510	0.0026	750	0.0098	990	0.0022	1230	0.0012
36	0.0011	276	0.0015	516	0.0028	756	0.0084	996	0.0022	1236	0.0013
42	0.001	282	0.0015	522	0.0029	762	0.008	1002	0.0021	1242	0.0013
48	0.0011	288	0.0015	528	0.0029	768	0.0074	1008	0.0021	1248	0.0012
54	0.0011	294	0.0016	534	0.003	774	0.0068	1014	0.0021	1254	0.0013
60	0.0011	300	0.0016	540	0.0032	780	0.0064	1020	0.0021	1260	0.0012
66	0.0011	306	0.0016	546	0.0032	786	0.006	1026	0.002	1266	0.0013
72	0.0011	312	0.0016	552	0.0032	792	0.0056	1032	0.002	1272	0.0012
78	0.0011	318	0.0017	558	0.0032	798	0.0054	1038	0.002	1278	0.0013
84	0.0012	324	0.0017	564	0.0032	804	0.0052	1044	0.0019	1284	0.0012
90	0.0011	330	0.0016	570	0.0032	810	0.0048	1050	0.002	1290	0.0012
96	0.0012	336	0.0018	576	0.0033	816	0.0046	1056	0.0019	1296	0.0013
102	0.0011	342	0.0017	582	0.0034	822	0.0044	1062	0.0019	1302	0.0012
108	0.0012	348	0.0017	588	0.0036	828	0.0042	1068	0.0018	1308	0.0012
114	0.0012	354	0.0018	594	0.0038	834	0.004	1074	0.0018	1314	0.0012
120	0.0012	360	0.0018	600	0.0039	840	0.0038	1080	0.0018	1320	0.0012
126	0.0012	366	0.0018	606	0.0041	846	0.0037	1086	0.0018	1326	0.0012
132	0.0012	372	0.0018	612	0.0044	852	0.0036	1092	0.0017	1332	0.0012
138	0.0013	378	0.0019	618	0.0046	858	0.0035	1098	0.0018	1338	0.0012
144	0.0012	384	0.0019	624	0.0048	864	0.0034	1104	0.0017	1344	0.0012
150	0.0012	390	0.0018	630	0.0051	870	0.0034	1110	0.0017	1350	0.0011
156	0.0013	396	0.002	636	0.0054	876	0.0033	1116	0.0016	1356	0.0012
162	0.0012	402	0.0019	642	0.0058	882	0.0033	1122	0.0017	1362	0.0012
168	0.0013	408	0.0019	648	0.0062	888	0.0032	1128	0.0016	1368	0.0011
174	0.0013	414	0.002	654	0.0066	894	0.0031	1134	0.0016	1374	0.0012

180	0.0013	420	0.002	660	0.007	900	0.003	1140	0.0015	1380	0.0011
186	0.0013	426	0.002	666	0.0077	906	0.003	1146	0.0016	1386	0.0012
192	0.0013	432	0.002	672	0.0086	912	0.0029	1152	0.0015	1392	0.0011
198	0.0013	438	0.0021	678	0.0096	918	0.0028	1158	0.0015	1398	0.0012
204	0.0014	444	0.0021	684	0.0106	924	0.0027	1164	0.0015	1404	0.0011
210	0.0013	450	0.0021	690	0.0115	930	0.0027	1170	0.0014	1410	0.0011
216	0.0014	456	0.0021	696	0.0238	936	0.0026	1176	0.0014	1416	0.0012
222	0.0014	462	0.0021	702	0.0476	942	0.0026	1182	0.0014	1422	0.0011
228	0.0013	468	0.0021	708	0.0764	948	0.0025	1188	0.0013	1428	0.0011
234	0.0014	474	0.0022	714	0.1371	954	0.0024	1194	0.0014	1434	0.0011
										1440	0.0011

Example

Using the following information, calculate the Channel Protection Volume and peak flow.

- Area = 50 acres
- CN = 72
- $t_c = 0.34$ hours
- $t_{lag} = 0.20$ hours
- Total impervious area = 18 acres

Calculate channel protection volume (CP_v) using Equation 3.5:

$$S = (1000/CN) - 10 = (1000/72) - 10 = 3.89 \text{ inches}$$

$$Q = (P - 0.2S)^2 / (P + 0.8S) = (2.58 - 0.2(3.89))^2 / (2.58 + 0.8(3.89)) = 0.57 \text{ inches}$$

Q can be converted to total runoff by applying the runoff to the total watershed and converting to appropriate units.

$$CP_v = (0.57 \text{ inches})(1 \text{ foot}/12 \text{ inches})(50 \text{ acres}) = 2.38 \text{ acre-feet}$$

The same hydrograph generation method can be used to compute the entire runoff hydrograph including the peak discharge. The following HEC-1 output file illustrates the result of the computation.

```

1*****
*
* FLOOD HYDROGRAPH PACKAGE (HEC-1)
*   JUN 1998
*   VERSION 4.1
*
* RUN DATE 05MAR07 TIME 18:11:48
*
*****
*
* U.S. ARMY CORPS OF ENGINEERS
* HYDROLOGIC ENGINEERING CENTER
* 609 SECOND STREET
* DAVIS, CALIFORNIA 95616
* (916) 756-1104
*
*****

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X X XXXXXXX XXXXX X
X X X X X XX
X X X X X X
XXXXXXXX XXXX X XXXXX X
X X X X X X
X X X X X X
X X XXXXXXX XXXXX XXX

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THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.
 THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
 THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
 NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
 DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
 KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

3.3 Storage Volume Computations

Design of BMPs to treat and control the Water Quality Volume (WQ_v) and the Channel Protection Volume (CP_v) require level pool routing methods as described in the Charlotte-Mecklenburg Storm Water Design Manual and this Manual. The design process is iterative where a preliminary storage volume and outlet system is assumed, the appropriate design hydrographs are routed through the system, output results are assessed, and refinements to the storage volume and outlet systems are made. This process is continued until all design storm events are properly treated or controlled within the BMP storage volume. The Static Method, described in the following section, can be used to estimate initial conditions so that the number of iterations is reduced.

Static Method

The Static Method assumes that all of the runoff volume enters the storage volume instantaneously and therefore, the storage volume is set equal to the runoff volume. The runoff volume exits the storage volume based solely on the outflow system hydraulics and the holding time duration for the selected design condition.

Example

Using the following information, calculate an initial estimate of the BMP storage volume and outlet system to hold the Channel Protection Volume for 24 hours beyond the center of rainfall (12 hours).

Area = 50 acres
 CN = 72
 $t_c = 0.34$ hours
 $t_{lag} = 0.20$ hours
 Total impervious area = 18 acres

The previous example computed the runoff volume to be 2.38 acre-feet. The storage volume is set equal to the runoff volume and units converted to determine the appropriate outlet flow rate.

$$\begin{aligned} Q_{outlet} &= (2.38 \text{ acre-feet})(43,560 \text{ ft}^2/\text{acre})/(36 \text{ hours})(3,600 \text{ sec/hour}) \\ &= 0.80 \text{ cfs} \end{aligned}$$

It is important to note that within Charlotte and its ETJ, the Channel Protection Volume must be held for 48 hours beyond the center of rainfall. In this example the value of 36 hours would be replaced with 60 hours for sites in the Charlotte area.

Based on the site conditions, the next step is for the designer to fit the 2.38 acre-feet of storage into the site topography. From the site topography, the total and average headwater depth on the outlet structure can be determined. For this example, we assume that the total storage volume depth is 5 feet. The average headwater depth is assumed to be $\frac{1}{2}$ of the maximum depth, or 2.5 feet.

Size the outlet orifice by using the orifice equation to compute cross-sectional area and diameter of outlet.

$$\begin{aligned} Q &= CA(2gh)^{0.5}, \text{ for } Q = 0.80 \text{ cfs, } h = 2.50 \text{ ft, and } C = \text{discharge coefficient} = 0.6 \\ \text{Solve for A: } A &= 0.80 \text{ cfs} / [0.6((2)(32.2 \text{ ft/s}^2)(2.50))^{0.5}] = 0.105 \text{ ft}^2 \\ \text{With } A &= \pi d^2/4, d = 0.37 \text{ ft} = 4.4 \text{ inches} \\ \text{Use 4.4 inch orifice plate.} \end{aligned}$$

Note that the static method provides an over-estimation of the storage volume needed to hold the runoff volume for the design duration. Detailed routing computations are required to complete the design during which the storage volumes, outlet sizes, etc. are altered during the routing computations. The

detailed outlet hydrograph analysis must show that a minimum percentage of the runoff volume is held within the storage volume at the design duration time (see Routing section below).

Routing

Detailed routing computations are required for the design of all BMPs where storage volume and detention time are key design parameters. Chapter 7 of the Charlotte-Mecklenburg Storm Water Design Manual provides a description of methods required to perform routing. In many cases, the designer will perform the required routing computations using computer models.

The previously described Static Method provides an estimate of the required storage volume to hold the design storm runoff volume for a specific holding duration. Routing computations will be iterative where a preliminary storage volume and outlet system is assumed, the routing computation is performed, and the output is checked to evaluate the success of the BMP meeting the holding duration. In Cornelius, Davidson, Matthews, Mint Hill, Pineville and in areas of Mecklenburg County outside of any planning ETJ's, the detailed outlet hydrograph analysis must show that a minimum of 5 percent of the runoff volume is held within the storage volume at the design duration time (36 hours). In Charlotte and Huntersville, the detailed outlet hydrograph analysis must show that a minimum of 50 percent of the runoff volume is held within the storage volume at 24 hours after the center of the rainfall – 12 hours (total 36 hours) and must show that a portion of the runoff volume is held within the storage volume at the design duration time for those jurisdictions (60 hours).

For example, for the CP_v design, the 1-year, 24-hour storm event must be held for 24 hours (or 48 hours in Charlotte). Therefore, the design should compute the amount of runoff volume that is left within the storage volume at 24 hours after the center of the rainfall (total 36 hours) for all jurisdictions and at 48 hours after the center of the rainfall (total 60 hours) for Huntersville and Charlotte. The remaining volume at 36 hours must be at least 50% of the total runoff volume for Huntersville and Charlotte and at least 5% of the total runoff volume in other jurisdictions. The remaining volume at 60 hours must be at least a portion of the total runoff volume for Huntersville and Charlotte.

3.4 Water Balance Calculations

3.4.1 Introduction

Water balance calculations are used to determine if a drainage area is large enough, or has the right characteristics, to support a BMP with a permanent pool of water during average conditions. Water balance calculations are required for the design of some BMPs included in this manual independent of contributing watershed size and required for the design of some BMPs included in this manual if the BMPs is proposed to serve a contributing watershed less than a specified threshold. The details of a rigorous water balance are beyond the scope of this manual; however, a simplified procedure is described in this section that will provide an estimate of pool viability and point to the need for more rigorous analysis.

The entire water year should be checked with water balance computations, but the summer season is typically the most critical because high outflows are experienced and are combined with the lowest inflows. Each month must be assessed during the analysis. Two starting conditions should be checked, the water year starting in July and the calendar year starting in January.

3.4.1 Basic Equations

Water balance is defined as the change in volume of the permanent pool resulting from the total inflow minus the total outflow (actual or potential).

$$\Delta V = \Sigma I - \Sigma O \quad (3.6)$$

Where: Δ = "change in"
 V = pond volume (acre-feet)
 Σ = "sum of"
 I = inflows (acre-feet)
 O = outflows (acre-feet)

The inflows consist of rainfall, runoff and baseflow into the pond or holding area. The outflows consist of infiltration, evaporation, evapotranspiration, and surface overflow out of the pond or wetland. Equation 3.6 can be changed to reflect these factors.

$$\Delta V = P + R_o + B_f - I - E - E_t - O_f \quad (3.7)$$

Where: P = precipitation (ac-ft)
 R_o = runoff (ac-ft)
 B_f = baseflow (ac-ft)
 I = infiltration (ac-ft)
 E = evaporation (ac-ft)
 E_t = evapotranspiration (ac-ft)
 O_f = overflow (ac-ft)

Rainfall (P)

Monthly values are commonly used for calculations of values over a season. Rainfall is the direct amount that falls on the pond surface for the period in question. When multiplied by the pond surface area (in acres) it becomes acre-feet of volume. Table 3-6 shows monthly rainfall values for the Mecklenburg County area.

Table 3-6 Average Monthly Rainfall Values for Mecklenburg County, North Carolina

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Precipitation (in)	3.72	3.84	4.44	2.64	3.84	3.36	3.96	3.72	3.48	3.36	3.24	3.48	43.08

Runoff (R_o)

Runoff is equivalent to the rainfall for the period multiplied by the "efficiency" of the watershed, which is equal to the ratio of runoff to rainfall. The following method can be used for BMP design in Mecklenburg County. Equation 3.1 gives a ratio of runoff to rainfall volume for a particular storm. If it can be assumed that the average storm that produces runoff has a similar ratio, then the R_v value can serve as the ratio of rainfall to runoff (R_v is defined in section 3.2).

Not all storms produce runoff in an urban setting. Typically initial losses (often called "initial abstractions") are normally estimated to be between 0.1 and 0.2 inches. When compared to the rainfall records in Mecklenburg County, this is equivalent to about a 10% runoff volume loss. Thus a factor of 0.9 should be applied to the calculated R_v value to account for storms that produce no runoff. Equation 3.8 reflects this approach. Total runoff volume (acre-feet) is the product of runoff depth ($0.9PR_v$) times the drainage area.

$$R_o = 0.9PR_vA/12 \quad (3.8)$$

Where: R_o = runoff volume (acre-feet)
 P = precipitation (in)
 R_v = volumetric runoff coefficient
 A = total drainage area – pond area (acres)

Baseflow (B_f)

Most storm water ponds and wetlands have little, if any, baseflow, as they are rarely placed across perennial streams. If so placed, baseflow must be estimated from observation or through theoretical

estimates. Review of the USGS Annual Data Report for North Carolina indicates that the watersheds in Mecklenburg County experience a baseflow in the range of 0.2 cfs (cubic feet per second per square mile of contributing drainage area) for urbanized watersheds and 0.4 cfs for rural watersheds. The urbanized value 0.2 cfs should be used for water balance computations when baseflow is present.

Infiltration (I)

Infiltration is a very complex subject and cannot be covered in detail here. The amount of infiltration depends on soils, water table depth, rock layers, surface disturbance, the presence or absence of a liner in the pond, and other factors. The infiltration rate is governed by the Darcy equation as:

$$I = Ak_h G_h \quad (3.9)$$

Where: I = infiltration (ac-ft/day)

A = cross sectional area through which the water infiltrates (acres)

For the purposes of this analysis, use ponding area at the permanent pool

k_h = saturated hydraulic conductivity or infiltration rate (ft/day)

G_h = hydraulic gradient = pressure head/distance

G_h can be set equal to 1.0 for the typical BMP application in the Mecklenburg County area. Infiltration rates can be established through testing, though not always accurately.

The value of k_h should be based on site specific soil condition testing if available. If site specific soil condition testing is not available, the values shown in Table 3-7 can be used based on the NRCS soil type that is present on site. If the proposed site grading is such that significant cut and fill will occur and therefore displace the NRCS soil type, then the site soil type with the highest k_h value shall be selected for use in the water balance computation.

Table 3-7 Saturated Hydraulic Conductivity

NRCS Soil Type	Hydraulic Conductivity In/hr	Hydraulic Conductivity ft/day
Appling	1.3	2.6
Cecil	1.3	2.6
Davidson	1.3	2.6
Enon	0.1	0.2
Georgeville	1.3	2.6
Goldston	4.0	8.0
Helena	0.4	0.8
Iredell	0.1	0.2
Lignum	0.3	0.6
Mecklenburg	0.1	0.2
Monacan	1.3	2.6
Pacolet	1.3	2.6
Pits	0.1	0.2
Vance	0.1	0.2
Wilkes	0.4	0.8

Evaporation (E)

Evaporation is from an open lake water surface. Evaporation rates are dependent on differences in vapor pressure, which, in turn, depend on temperature, wind, atmospheric pressure, water purity, and shape and depth of the pond. It is estimated or measured in a number of ways, which can be found in most hydrology textbooks. Table 3-8 gives evaporation rates for a typical 12-month period based on information from a station in Union, SC. The values in Table 3-8 are converted to volume of evaporation by multiplying by the pond surface area and converting to acre-feet.

Table 3-8 Monthly Evaporation Rates (inches)

Mon	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Lake	1.2	1.5	2.5	3.7	4.4	4.6	4.8	4.5	3.3	2.4	1.5	1.1	35.5

Evapotranspiration (E_t)

Evapotranspiration consists of the combination of evaporation and transpiration by plants. The estimate of E_t for crops is well documented and has become standard practice. However, for wetlands the estimating methods are not well documented, nor are there consistent studies to assist the designer in estimating the demand wetland plants would put on water volumes. Literature values for various places in the United States vary around the free water surface lake evaporation values. Estimating E_t only becomes important when wetlands are being designed and emergent vegetation covers a significant portion of the pond surface. In these cases conservative estimates of lake evaporation should be compared to crop-based E_t estimates. Crop-based E_t can be obtained from typical hydrology textbooks and from related web sites.

Overflow (O_f)

Overflow is considered as excess runoff and is defined by the volume of water temporarily held in the pond above the “permanent” pool elevation. In the simplified water balance calculations presented in this Manual, all water volume above the permanent pool volume is considered lost at the end of each month (no more than the maximum permanent pool volume is carried forward to the next month).

Example

A 23 acre site in Mecklenburg County is being developed along with an estimated 0.5 acre surface wet pond. There is no baseflow. The desired pond volume to the overflow point is 2.0 acre-feet at a pond stage of 705.0 feet. The designer must show that the site be able to support the pond volume. From the basic site data it is determined that the site is 40% impervious and has Enon NRCS soil type throughout the entire site.

- From Equation 3.1, $R_v = 0.05 + 0.009(40) = 0.41$. With the correction factor of 0.9 the watershed efficiency is 0.37.
- The annual lake evaporation is 35.5 inches. (Table 3-8)
- For a Enon soil the infiltration rate is $I = 0.20$ ft/day (Table 3-7)

Table 3-9 shows summary calculations for this site for each month of the year based on a calendar year computation and water year computation.

Table 3-9 Water Balance Result Summary

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1												
2	Days/month	31.0	28.0	31.0	30.0	31.0	30.0	31.0	30.0	31.0	30.0	31.0
3	Precipitation (inches)	3.72	3.84	4.44	2.64	3.84	3.36	3.96	3.72	3.48	3.36	3.48
4	Evaporation (inches)	1.20	1.50	2.50	3.70	4.40	4.60	4.80	4.50	3.30	2.40	1.10
5	Runoff (acre-feet)	2.57	2.66	3.07	1.83	2.66	2.32	2.74	2.57	2.41	2.32	2.41
6	Pond Precipitation (acre-feet)	0.16	0.16	0.19	0.11	0.16	0.14	0.17	0.16	0.15	0.14	0.15
7	Evaporation (acre-feet)	0.05	0.06	0.10	0.15	0.18	0.19	0.20	0.19	0.14	0.10	0.05
8	Infiltration (acre-feet)	3.10	2.80	3.10	3.00	3.10	3.00	3.10	3.10	3.00	3.10	3.10
9	Monthly Balance (acre-feet)	-0.42	-0.05	0.05	-1.22	-0.47	-0.73	-0.40	-0.56	-0.58	-0.74	-0.69
10	Running Balance of Retained Pond Volume (acre-feet)	0.00	0.00	0.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
11	Pond water surface elevation (feet)	699.8	699.8	700.0	699.8	699.8	699.8	699.8	699.8	699.8	699.8	699.8

12	Water surface with respect to permanent pool elevation (feet)	-5.2	-5.2	-5.0	-5.2	-5.2	-5.2	-5.2	-5.2	-5.2	-5.2	-5.2	-5.2
1		Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun
2	Days/month	31.0	31.0	30.0	31.0	30.0	31.0	31.0	28.0	31.0	30.0	31.0	30.0
3	Precipitation (inches)	3.96	3.72	3.48	3.36	3.24	3.48	3.72	3.84	4.44	2.64	3.84	3.36
4	Evaporation (inches)	4.80	4.50	3.30	2.40	1.50	1.10	1.20	1.50	2.50	3.70	4.40	4.60
5	Runoff (acre-feet)	2.74	2.57	2.41	2.32	2.24	2.41	2.57	2.66	3.07	1.83	2.66	2.32
6	Pond Precipitation (acre-feet)	0.17	0.16	0.15	0.14	0.14	0.15	0.16	0.16	0.19	0.11	0.16	0.14
7	Evaporation (acre-feet)	0.20	0.19	0.14	0.10	0.06	0.05	0.05	0.06	0.10	0.15	0.18	0.19
8	Infiltration (acre-feet)	3.10	3.10	3.00	3.10	3.00	3.10	3.10	2.80	3.10	3.00	3.10	3.00
9	Monthly Balance (acre-feet)	-0.40	-0.56	-0.58	-0.74	-0.69	-0.59	-0.42	-0.05	0.05	-1.22	-0.47	-0.73
10	Running Balance of Retained Pond Volume (acre-feet)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.00	0.00	0.00
11	Pond water surface elevation (feet)	699.8	699.8	699.8	699.8	699.8	699.8	699.8	699.8	700.0	699.8	699.8	699.8
12	Water surface with respect to permanent pool elevation (feet)	-5.2	-5.2	-5.2	-5.2	-5.2	-5.2	-5.2	-5.2	-5.0	-5.2	-5.2	-5.2

Explanation of Table:

1. Months of the year.
2. Days per month.
3. Monthly precipitation from Table 3-6.
4. Evaporation by month from Table 3-8.
5. Watershed efficiency of 0.37 times the rainfall and converted to acre-feet.
6. Precipitation volume directly into pond equals precipitation depth times pond surface area divided by 12 to convert to acre-feet.
7. Evaporation from line 4 converted to acre-feet.
8. Infiltration converted to acre-feet.
9. Lines 5 and 6 minus lines 7 and 8.
10. Accumulated total from line 9 keeping in mind that all volume above 2 acre-feet overflows and is lost in the trial design.
11. Pond water surface elevation based on stage-storage data for pond.
12. Pond water surface elevation with respect to permanent pool elevation.

For this example the wet pond has the potential to go dry throughout the year. This could be remedied in a number of ways including compacting the pond bottom in areas that are not planned to be planted, placing a liner of clay or geosynthetics, and changing the pond geometry to decrease surface area. Table 3-10 shows revised water balance computations for the same site using an infiltration rate of 0.1 ft/day based on a clay liner.

Table 3-10 Revised Water Balance Result Summary (Clay Liner)

1		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2	Days/month	31.0	28.0	31.0	30.0	31.0	30.0	31.0	31.0	30.0	31.0	30.0	31.0
3	Precipitation (inches)	3.72	3.84	4.44	2.64	3.84	3.36	3.96	3.72	3.48	3.36	3.24	3.48
4	Evaporation (inches)	1.20	1.50	2.50	3.70	4.40	4.60	4.80	4.50	3.30	2.40	1.50	1.10
5	Runoff (acre-feet)	2.57	2.66	3.07	1.83	2.66	2.32	2.74	2.57	2.41	2.32	2.24	2.41
6	Pond Precipitation (acre-feet)	0.16	0.16	0.19	0.11	0.16	0.14	0.17	0.16	0.15	0.14	0.14	0.15
7	Evaporation (acre-feet)	0.05	0.06	0.10	0.15	0.18	0.19	0.20	0.19	0.14	0.10	0.06	0.05
8	Infiltration (acre-feet)	1.55	1.40	1.55	1.50	1.55	1.50	1.55	1.55	1.50	1.55	1.50	1.55

9	Monthly Balance (acre-feet)	1.13	1.35	1.60	0.28	1.08	0.77	1.15	0.99	0.92	0.81	0.81	0.96
10	Running Balance of Retained Pond Volume (acre-feet)	1.13	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
11	Pond water surface elevation (feet)	703.0	705.0	705.0	705.0	705.0	705.0	705.0	705.0	705.0	705.0	705.0	705.0
12	Water surface with respect to permanent pool elevation (feet)	-2.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1		Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun
2	Days/month	31.0	31.0	30.0	31.0	30.0	31.0	31.0	28.0	31.0	30.0	31.0	30.0
3	Precipitation (inches)	3.96	3.72	3.48	3.36	3.24	3.48	3.72	3.84	4.44	2.64	3.84	3.36
4	Evaporation (inches)	4.80	4.50	3.30	2.40	1.50	1.10	1.20	1.50	2.50	3.70	4.40	4.60
5	Runoff (acre-feet)	2.74	2.57	2.41	2.32	2.24	2.41	2.57	2.66	3.07	1.83	2.66	2.32
6	Pond Precipitation (acre-feet)	0.17	0.16	0.15	0.14	0.14	0.15	0.16	0.16	0.19	0.11	0.16	0.14
7	Evaporation (acre-feet)	0.20	0.19	0.14	0.10	0.06	0.05	0.05	0.06	0.10	0.15	0.18	0.19
8	Infiltration (acre-feet)	1.55	1.55	1.50	1.55	1.50	1.55	1.55	1.40	1.55	1.50	1.55	1.50
9	Monthly Balance (acre-feet)	1.15	0.99	0.92	0.81	0.81	0.96	1.13	1.35	1.60	0.28	1.08	0.77
10	Running Balance of Retained Pond Volume (acre-feet)	1.15	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
11	Pond water surface elevation (feet)	703.0	705.0	705.0	705.0	705.0	705.0	705.0	705.0	705.0	705.0	705.0	705.0
12	Water surface with respect to permanent pool elevation (feet)	-2.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

3.5 Downstream Flood Analysis

3.5.1 Introduction

The purpose of the flood control detention requirements (10- and 25-year, 6-hour storm event control) is to protect downstream properties from increased flooding due to upstream development. Current standard policies require that 10- and 25-year storms are controlled at the outlet of a site such that post-development peak discharge equals pre-development peak discharge. For commercial developments, some (or all) of the 25-year storm event control requirement can be waived if a downstream analysis indicates that the receiving drainage system meets design standards. For residential developments, some (or all) of the 10- and 25-year storm event control requirement can be waived if a downstream analysis indicates that the receiving drainage system meets design standards. The flood control detention waiver if a project discharges directly to a FEMA floodplain no longer applies. Even though the waiver no longer applies, most developments along the FEMA regulated floodplain will pass the 10 times the site area test described in Section 3.5.3. For example, any project with an area less than 64 acres that discharges directly to the floodway will discharge to a stream system with a drainage area larger than 640 acres and therefore would comply with the 10% rule requirement for downstream analysis at the system outfall.

The portion of the 10- and/or 25-year storm event control requirement that can be waived is based on how well the receiving system meets design standards. The 10- and/or 25-year storm event control cannot be waived, if through the waiver portions of the downstream system do not meet design standards.

Design standards are defined as all storm drainage system requirements described in the Charlotte-Mecklenburg Storm Water Design Manual. These include providing pipe capacity for the 10- and 25-year storm events using Mannings formula and hydraulic gradeline computations, culvert capacity for design storms based on roadway type, and providing protection for first floor flooding for the 100-year storm

event. The storms that will need to be analyzed will be based on what features are located within the limits of this study area. Design storms for various features are listed in Table 3-11.

Table 3-11: Summary of Design Standards for Downstream Analysis

Design feature	Design storm for analysis (all 6-hr)
Closed pipe/Channel systems/Driveway culverts	10-year
Culverts (subdivision streets)	25-year
Culverts (thoroughfare roads)	50-year
Building footprints	100-year

The design storms to be analyzed will depend upon what is downstream from the project site. For example, if there is only a local street crossing within the study area, only the 25-yr 6-hr storm would need to be modeled. The other storms would have to be analyzed as needed.

3.5.2. Reasons for Downstream Problems

Many of the sites that are being developed are upstream from existing urbanized areas where the storm drainage infrastructure has been in place for some time. In some cases, this storm drainage infrastructure does not meet current design standards or is showing some signs of failure due to deterioration, poor installation, etc. Design standards for these systems vary based on the potential for flood damage from the 10-year flood event for the pipe and channel systems up to the 100-year flood event for the protection of living space. Current detention requirements require the attenuation of some of these flood events, but not all of the flood events. Therefore, there may be some downstream systems that experience increased discharges due to upstream development for flood events that are not addressed by the current detention requirements. Therefore, the flood damages on these properties may be exacerbated by the upstream development.

3.5.3 The Ten-Percent Rule

The “ten-percent” rule is a practical, flexible and effective approach for ensuring all potential impacts of flooding or increased volume from development are assessed. The ten-percent rule recognizes the fact that development and/or a structural BMP control providing detention has a “zone of influence” downstream where its effectiveness can be felt. Beyond this zone of influence the development and/or structural control becomes relatively small and insignificant compared to the runoff from the total drainage area to that point. Based on studies and master planning results, that zone of influence is considered to be the point where the drainage area controlled by the detention facility comprises 10% of the total drainage area (Debo & Reese, 1990). Although the detention exemption along floodplains will no longer be in the ordinance, most developments along the FEMA regulated floodplain will pass the 10 times the site area test. For example, any project with an area less than 64 acres that discharges directly to the floodway will discharge to a stream system with a drainage area larger than 640 acres and therefore would be immediately complying with the 10% rule requirement for downstream analysis.

For example, if the structural BMP control drains 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater.

Typical steps in the application of the ten-percent rule are:

1. Using a topographic map determine the lower limit of the zone of influence (10% point).
2. Using a hydrologic model determine the pre-development peak flows and timing of those peaks at each tributary junction beginning at the pond outlet and ending at the next tributary junction beyond the 10% point. Two watershed land use conditions should be considered when developing the hydrologic model and assessing the potential for downstream impacts, existing and future land use. In addition, the designer should assess other watershed conditions such as sub-basins size, upstream attenuation, etc. so that the results are conservative.

3. Change the land use on the site to post-development and rerun the model.
4. Perform appropriate hydraulic computations for the downstream system to determine if the drainage systems meet or do not meet design standards. Hydraulic computations that may need to be considered include pipe capacity computations, culvert capacity analysis, floodplain backwater computations, channel stability assessment, etc. Storm and flood events that should be assessed include the 10-, 25, 50-, and 100-year events based on the downstream system design standards. For both models, the base survey will need to identify all features that could be affected by flooding to adequately assess impacts. The accuracy of the analysis shall meet current site plan design standards and be sealed by a North Carolina Professional Engineer or Landscape Architect. The field survey shall meet current site plan design standards and be sealed by a North Carolina registered land surveyor.
5. Design the structural BMP control facility such that the pre-developed conditions and post-developed conditions are the same at the downstream site property line and downstream receiving system meets all design standards.

Example

Figure 3.5.1 illustrates the concept of the ten-percent rule for two sites in a watershed.

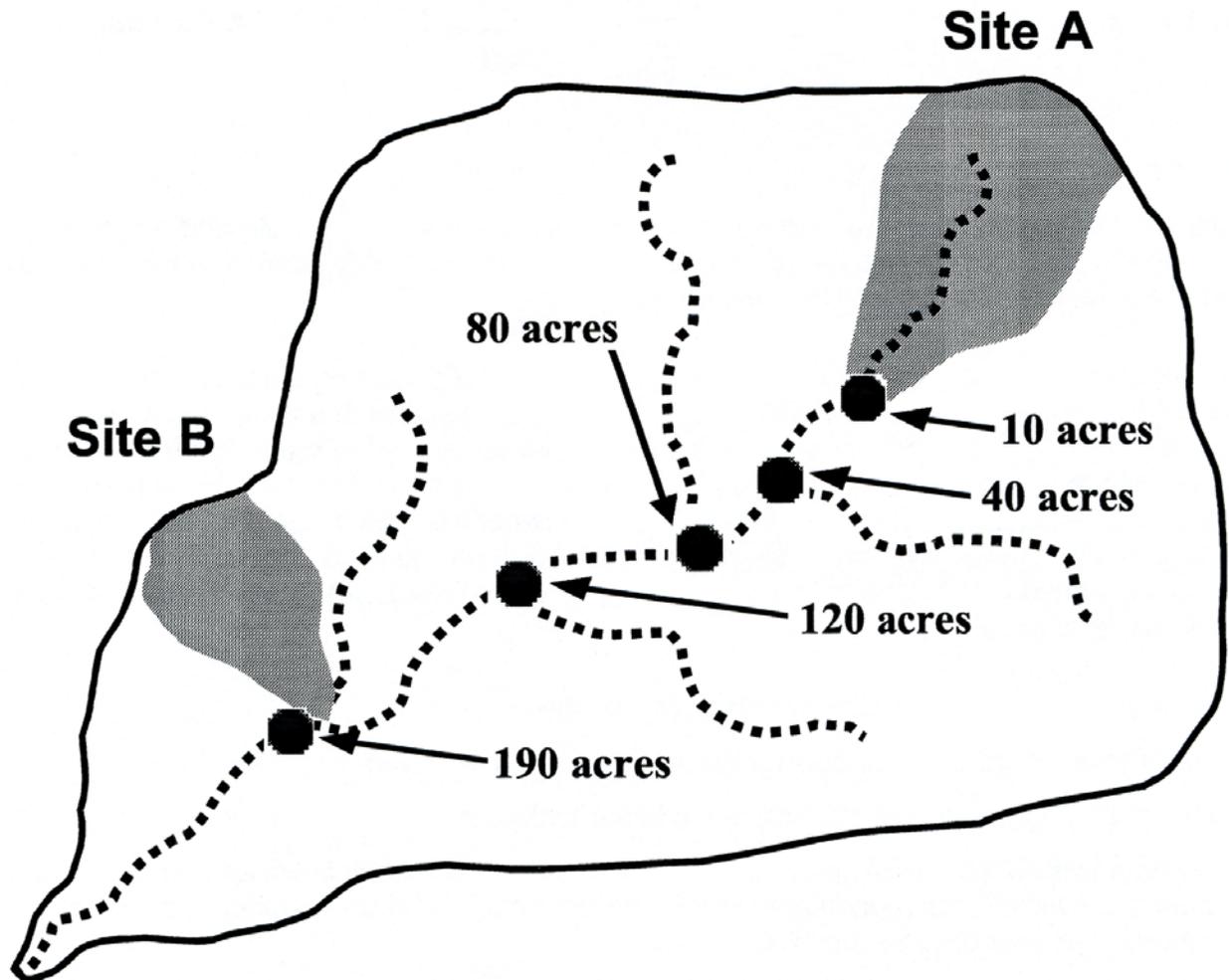


Figure 3.5.1 Example of the Ten-Percent Rule

Site A is a commercial development of 10 acres, all draining to a wet pond BMP. The design standards require the post-developed discharges for the 10- and 25-year storm events are controlled to pre-developed conditions at the property line unless a downstream analysis is performed that shows the receiving system can accept additional discharge. The receiving drainage system requires the assessment of the 10-, 25-, 50-, and 100-year storm events down to the ten-percent rule location (see Table 3-11 for appropriate design storms). Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked "120 acres". The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points. For simplicity, the analysis can be run down to the 120-acre point, or if the designer wishes, drainage areas can be iteratively drawn to find the exact 100-ac point.

The designer constructs a HEC-1 model, or equivalent, of the 80-acre areas using existing and future land use conditions for each sub-watershed in each tributary. Major attenuation throughout the watershed that could impact the accuracy of the analysis should be included in the model in order to produce conservative results. Hydraulic computations are performed to assess the capacity of the receiving system. The downstream analysis indicates that the entire drainage system meets design

standards for all appropriate storm event; 10-, 25-, 50-, and 100-year. Therefore, control of only the 10-year storm event at the outfall of the development site is required. If the downstream analysis showed that downstream system did not meet design standards, then controlling both the appropriate design storms would be required.

Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10-percent point is the junction of the site outlet with the stream. Therefore, the limits of the downstream analysis are the downstream site boundary.

3.6 Combined Water Quality (WQ_v) and Channel Protection (CP_v) Design

3.6.1 Introduction

The post-construction ordinance requires treatment of the 1-inch, 6-hour storm event to meet water quality (WQ_v) goals and control of the 1-year, 24-hour storm event to meet channel protection (CP_v) goals. For some sites (relatively small and high impervious sites) the BMP storage volume needed to meet both of these goals may be similar in size. To facilitate a simpler and more straight-forward design process, a design target has been developed that meets both the water quality treatment (WQ_v) and channel protection control (CP_v) goals of the post-construction ordinance.

The simpler and most straight-forward design process is called the *Combined Water Quality and Channel Protection Design* and was developed by assessing numerous sites with various watershed sizes and imperviousness intensity. The results of the numerous site assessment indicates that holding the 1-year, 24-hour storm event for 24 hours longer than the detention time for the 1-inch, 6-hour storm event will provide the appropriate water quality treatment. Therefore, a site designer can elect to perform a *Combined Water Quality and Channel Protection Design* for the optimal efficiency wet pond by holding the 1-year, 24-hour storm event for 120 hours (4 days detention time for the optimal efficiency design plus 24 hours) after the center of rainfall (12 hours). Design computations showing that the 1-inch, 6-hour storm event will not needed to be performed.

The *Combined Water Quality and Channel Protection Design* applies to BMPs in this Manual that are designed by routing the design storms through the BMP storage volume which include wet ponds, wetlands, and bioretention. The design is met by controlling the channel protection volume (CP_v) for a duration that is 24 hours longer than the design duration for the water quality (WQ_v) design. For example, the optimal efficiency for a wet pond is met by controlling the 1-year, 24-hour storm event for 120 hours (4 days plus 24 hours). The standard efficiency for a wet pond is met by controlling the 1-year, 24-hour storm event for 72 hours (2 days plus 24 hours). These design durations are always measured after the center of rainfall.

3.6.2 Example

The following design example is a re-design of the wet pond design presented in Section 4.2 using the *Combined Water Quality and Channel Protection Design*. The site data is summarized as follows:

Site Area = Total Drainage Area (A) = 10.0 acre
Impervious Area – 3.4 ac; or $I=3.4/10=34.0\%$
Soils Types: 50% “C”, 50% “B”
Land Use: Residential (1/2 acre lots)

Figure 3.6.1 shows the site plan for the development and base hydrologic data that will be used in the design example.

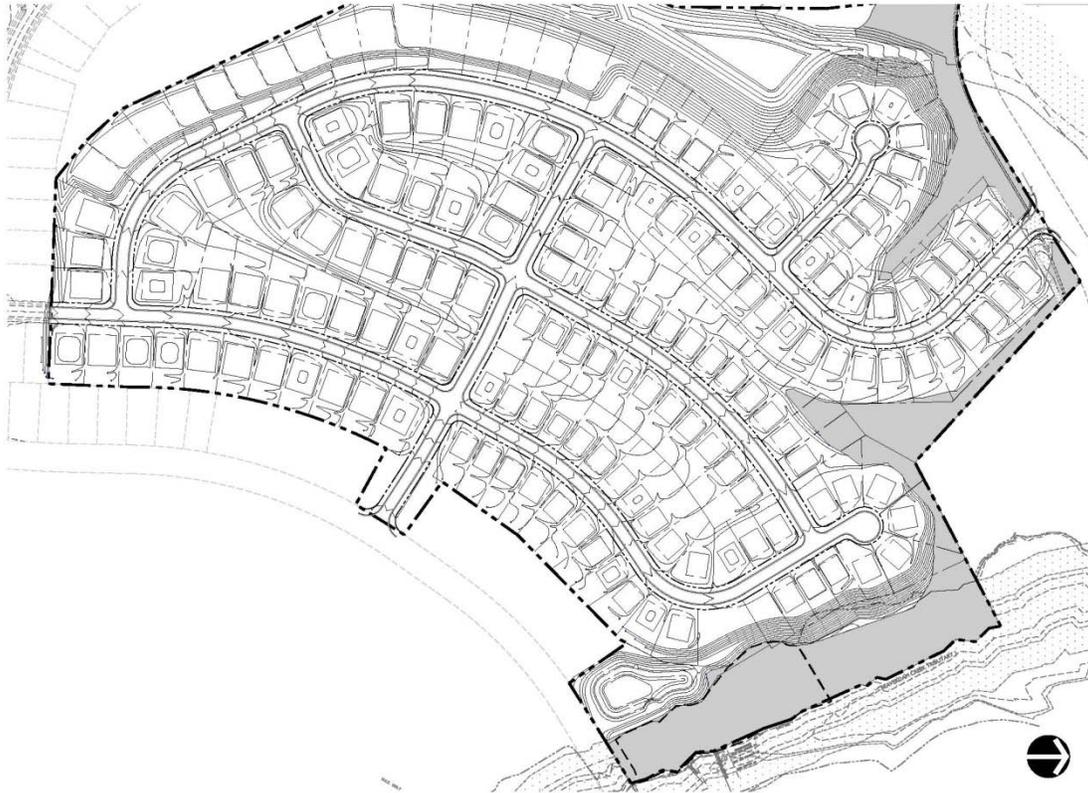


Figure 3.6.1 Example Site Plan for Combined Water Quality and Channel Protection Design

Step 1 Compute Channel Protection Volume (CP_v)

- Compute maximum soil retention using SCS methods shown in the Charlotte-Mecklenburg Storm Water Design Manual. Note that the CN value used is the original site CN value, not the adjusted CN value used during the water quality runoff volume computation.

$$\begin{aligned}
 S &= 1000/CN-10 \\
 &= 1000/77.8 - 10 \\
 &= 2.85 \text{ inches}
 \end{aligned}$$

Compute total runoff for the 1-year, 24-hour storm event. Total rainfall depth is 2.58 inches.

$$\begin{aligned}
 Q_d &= (P-0.2S)^2/(P+0.8S) \\
 &= [2.58 - (0.2)(2.85)]^2/(2.58 + 0.8*2.85) \\
 &= 0.83 \text{ inches}
 \end{aligned}$$

Compute watershed runoff

$$CP_v = (0.83 \text{ inches})(10 \text{ acres})(1 \text{ foot}/12 \text{ inches}) = 0.69 \text{ acre-feet}$$

The entire 1-year, 24-hour is required to be held within the wet pond dry storage volume above the permanent pool for a minimum of 120 hours. A design method that is called the “Static Method” sets the storage volume equal to the runoff volume, assumes that the storage volume fills instantaneously and empties through the outlet structure orifices and weirs. Using the Static Method, the facility would require 0.69 acre-feet for the Channel Protection Volume (CP_v). These values can be used as estimates to develop approximate storage volumes and grading plans, but routing computations must be performed to complete the design.

Step 2 Compute Release Rate for Combined Water Quality and Channel Protection Design

The following outlet hydraulic computations are performed using the Static Method. Routing computations must be performed to refine the design that show that a minimum of 5 percent of the runoff volume is held within the storage volume at the design duration time after the center of the design storm rainfall (12 hours for 1-year, 24-hour storm event).

- Compute the release rate for the 1-year, 24-hour storm event.

The 1-year, 24-hour storm event is to be released over a 120 hours beyond the center of rainfall (120 hours plus 12 hours) period.

$$\text{Release rate} = (0.69 \text{ ac-ft} \times 43560 \text{ ft}^2/\text{acre}) / (132 \text{ hrs} \times 3,600 \text{ sec/hr}) = 0.063 \text{ cfs}$$

Step 3 Compute Site Hydrologic Input Parameters

Using SCS hydrologic procedures and/or computer models the data shown in Table 3-12 and 3-13 can be determined for the example development site.

Table 3-12 Hydrologic Input Data

Condition	Area (acres)	CN	CN (adjusted) for 1-inch storm	t_c (hours)
Pre-developed	10	65	N/A	0.631
Post-developed	10	77.8	91.0	0.202

**Table 3-13 Results of Preliminary Hydrologic Calculations
(From Computer Model Results Using SCS Hydrologic Procedures)**

Condition	$Q_{1\text{-year}}$	$Q_{10\text{-year}}$	$Q_{25\text{-year}}$	$Q_{50\text{-year}}$
Runoff	cfs	cfs	cfs	cfs
Pre-developed	1.6	7.8	11.6	14.8
Post-developed	10.7	27.6	36.1	42.7

Step 4 Develop Storage-Elevation Table and Curve

Figure 3.6.1 shows the pond location on site, Figure 3.6.2 shows the plan view of the pond grading and Table 3-14 shows the storage-elevation data that was developed for this example.

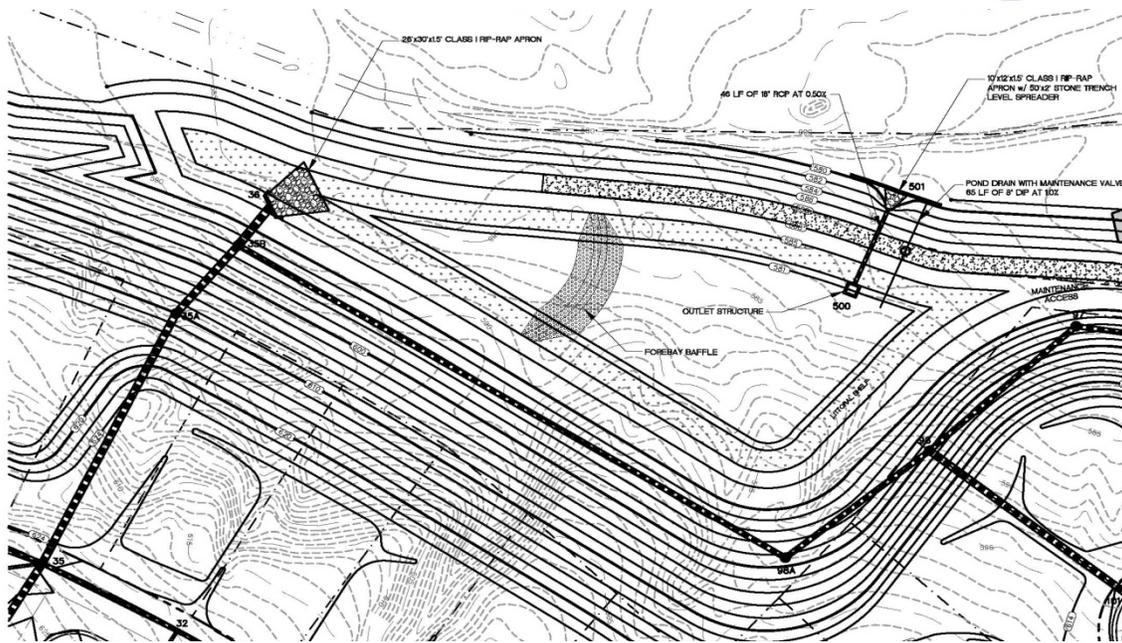


Figure 3.6.2 Plan View of Pond Grading (Not to Scale)

Table 3-14 Storage-Elevation Data

Elevation	Area (sf)	Area (ac)	Avg. Area (ac)	Height (ft)	Inc vol (ac-ft)	Acc vol (ac-ft)	Acc vol (ac-ft)
695	6400	0.147				0.000	
695.5	6889	0.158	0.153	0.5	0.076	0.076	
696	7396	0.170	0.164	0.5	0.082	0.158	
696.5	7921	0.182	0.176	0.5	0.088	0.246	
697.0	8464	0.194	0.188	0.5	0.094	0.340	
697.5	9025	0.207	0.201	0.5	0.100	0.441	
698.0	9604	0.220	0.214	0.5	0.107	0.547	
698.5	10201	0.234	0.227	0.5	0.114	0.661	
699.0	10816	0.248	0.241	0.5	0.121	0.782	
699.5	11449	0.263	0.256	0.5	0.128	0.910	
700.0	12100	0.278	0.270	0.5	0.135	1.045	0
700.5	12769	0.293	0.285	0.5	0.143	1.187	0.143
701.0	13456	0.309	0.301	0.5	0.151	1.338	0.293
701.5	14161	0.325	0.317	0.5	0.158	1.496	0.452
702.0	14884	0.342	0.333	0.5	0.167	1.663	0.618
702.5	15265	0.350	0.346	0.5	0.173	1.836	0.791
703.0	16384	0.376	0.363	0.5	0.182	2.018	0.973
703.5	17161	0.394	0.385	0.5	0.193	2.210	1.166
704.0	17956	0.412	0.403	0.5	0.202	2.412	1.367
704.5	18769	0.431	0.422	0.5	0.211	2.623	1.578
705.0	19600	0.450	0.440	0.5	0.220	2.843	1.798

Step 5 Set Basic Elevations for BMP Structure

Set basic elevations for pond structures.

- The pond bottom is set at elevation 695.0.
- Provide gravity flow to allow for pond drain, set riser invert at 694.5.
- Set barrel outlet elevation at 694.0.

Set water surface and other elevations.

- Required permanent pool volume = 1.00 ac-ft. From the elevation-storage table or curve, read the elevation that will have a cumulative volume of 1.00 ac-ft or greater = 700.0. This elevation has a cumulative volume of 1.045 ac-ft which is greater than 1.00 ac-ft to allow for a small safety factor.
- Forebay volume will be provided in two pools, each below the two major inflow location, with an average volume of 0.030 ac-ft in each. This will give the required forebay volume of 0.06 ac-ft.
- The required extended detention volume is the 1-year, 24-hour volume of 0.69 ac-ft held for 120 hours beyond the center of rainfall at 12 hours. From the elevation-storage table or curve (volume above permanent pool), read the elevation that will allow 0.69 ac-ft of storage above the permanent pool at elevation 700. The preliminary elevation to hold the Channel Protection Volume (CP_v) is 702.2.

Step 6 Compute Required Outlet Structure and Stage-Discharge for Combined Water Quality and Channel Protection Volume

Compute the required combined water quality and channel protection volume orifice diameter to release 0.69 acre-feet over 120 hours beyond 12 hours.

- Required 1-year, 24-hour runoff volume = 0.69 ac-ft
- From the elevation storage table or curve, the elevation at that storage volume = 702.2

Size the combined WQ_v and CP_v design orifice.

- Average extended detention release rate = $(0.69 \text{ ac-ft})(43,560 \text{ ft}^2/\text{ac}) / (132 \text{ hr})(3600 \text{ sec/hr}) = 0.063 \text{ cfs}$
- Average head = $(702.2 - 700.0) / 2 = 1.10 \text{ ft}$
- Use orifice equation to compute cross-sectional area and diameter of outlet.
 - $Q = CA(2gh)^{0.5}$, for $Q = 0.063 \text{ cfs}$, $h = 1.15 \text{ ft}$, and $C = \text{discharge coefficient} = 0.6$
 - Solve for A: $A = 0.063 \text{ cfs} / [0.6((2)(32.2 \text{ ft/s}^2)(1.10))^{0.5}] = 0.012 \text{ ft}^2$
 - With $A = \pi d^2 / 4$, $d = 0.125 \text{ ft} = 1.5 \text{ inches}$
 - Use 1.5 inch orifice plate.

Compute the stage-discharge equation for the 1.5 diameter combined WQ_v and CP_v orifice.

- $Q = CA(2gh)^{0.5} = 0.6(0.012 \text{ ft}^2)[2(32.2 \text{ ft/s}^2)]^{0.5}(h^{0.5})$
 $Q = 0.058h^{0.5}$ Where $h = \text{wsel} - 700.06$
 (Note: Account for one half of the orifice diameter when calculation head).

Step 7 Calculate Q_{10} and Q_{25} (if required) Release Rate(s) and Water Surface Elevation(s)

In order to calculate the 10-year and 25-year (if required) release rate(s) and water surface elevation(s), the designer must set up a stage-storage-discharge relationship for the control structure for the low flow release pipes (WQ_v and CP_v) plus the 10-year and 25-year (if required) storm(s).

The first step is to route the 1-year, 24-hour storm event through the facility using the orifice and stage-storage developed by the Static Method design. The following HEC-1 output file illustrates the results. Note that the peak stage elevation for the 1-year, 24-hour storm event is 701.99 which is less than 702.2,



the peak stage assumed during the Static Method design. In addition, export of the outflow hydrograph through the TAPE21 function indicates that 15% of the runoff volume remains within the basin at the time duration 132 hours, therefore, additional iterations of the 1-year, 24-hour routing are performed. The goal of the additional iterations is to increase the outlet size and reduce the storage volume so that only 5% of the runoff volume remains in the wet pond storage volume at the time duration of 132 hours.

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* FLOOD HYDROGRAPH PACKAGE (HEC-1)
*   JUN 1998
*   VERSION 4.1
*
* RUN DATE 14MAR07 TIME 10:46:00
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* U.S. ARMY CORPS OF ENGINEERS
* HYDROLOGIC ENGINEERING CENTER
* 609 SECOND STREET
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THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.

THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION

NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY, DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION

KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1 HEC-1 INPUT PAGE 1

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LINE ID.....1.....2.....3.....4.....5.....6.....7.....8.....9.....10
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2 ID ANALYZED BY ABC ENGINEERING
3 ID DATE: OCTOBER 2006
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* TIME SPECIFICATION CARD
* TOTAL COMPUTATIONAL DURATION 132 HOURS
4 IT 4 0 0 1980
* DIAGRAM
* OUTPUT CONTROL CARD
5 IO 5 0 0
*
6 KK PRE1
7 IN 6
8 PB 2.58
* ***** 1-YEAR, 24-HOUR STORM EVENT *****
* ***** 6 MINUTE TIME INCREMENT, 24-HOUR STORM EVENT *****
9 PI .0000 .0010 .0010 .0010 .0011 .0010 .0011 .0010 .0011 .0011
10 PI .0011 .0011 .0011 .0011 .0012 .0011 .0012 .0011 .0012 .0012
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18 PI .0032 .0032 .0032 .0032 .0032 .0032 .0033 .0034 .0036 .0038
19 PI .0039 .0041 .0044 .0046 .0048 .0051 .0054 .0058 .0062 .0066
20 PI .0070 .0077 .0086 .0096 .0106 .0115 .0238 .0476 .0764 .1371
21 PI .0951 .0190 .0166 .0144 .0122 .0098 .0084 .0080 .0074 .0068
22 PI .0064 .0060 .0056 .0054 .0052 .0048 .0046 .0044 .0042 .0040
23 PI .0038 .0037 .0036 .0035 .0034 .0034 .0033 .0033 .0032 .0031
24 PI .0030 .0030 .0029 .0028 .0027 .0027 .0026 .0026 .0025 .0024
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32 PI .0011 .0012 .0011 .0012 .0011 .0011 .0012 .0011 .0011 .0011
33 PI .0011
34 KM 10-ACRE PRE-DEVELOPED CONDITIONS
35 KO 5 0 0 0 21
36 BA .0156
37 LS 0 65.0 0
38 UD 0.378

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1 HEC-1 INPUT PAGE 2

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LINE ID.....1.....2.....3.....4.....5.....6.....7.....8.....9.....10
39 KK POST1
40 KM 10-ACRE POST-DEVELOPED CONDITIONS
41 KO 5 0 0 0 21
42 BA .0156
43 LS 0 77.8 0
44 UD 0.121

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45      KK DETE10
46      KO      5      0      0      0      21
47      KM DETENTION BASIN DESIGN FOR 1-YEAR, 24-HOUR STORM EVENT HELD FOR 132 HOURS
48      KM POST-DEVELOPED WITH CONTROLS
49      KM 1.50 INCH ORIFICE TO ELEVATION 700.0
50      KM TOP OF EXTENDED DETENTION STORAGE 702.2
51      KM BASED ON STATIC METHOD
52      RS      1      ELEV      700
53      SV      0      .143      .293      .452      .618      .690
54      SE      700      700.5      701      701.5      702      702.2
55      SQ      0.00      0.039      0.057      0.071      0.082      0.088
56      ZZ

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* VERSION 4.1
* RUN DATE 14MAR07 TIME 10:46:00
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MECKLENBURG COUNTY BMP DESIGN MANUAL
ANALYZED BY ABC ENGINEERING
DATE: OCTOBER 2006

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD			BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
				6-HOUR	24-HOUR	72-HOUR			
+	HYDROGRAPH AT								
	PRE1	1.	12.47	0.	0.	0.	.02		
+	HYDROGRAPH AT								
	POST1	10.	12.13	1.	0.	0.	.02		
+	ROUTED TO								
	DETE10	0.	22.93	0.	0.	0.	.02	701.99	24.27

*** NORMAL END OF HEC-1 ***

The following HEC-1 output illustrates that result of the additional iterations that were performed to refine the Static Method orifice size. The goal is to increase the outlet size and reduce the storage volume so that only 5% of the runoff volume remained in the basin at the time duration of 132 hours. The orifice size was increased from 1.5 inches to 1.7 inches by performing the additional iterations. The routed peak stage was decreased from 701.99 to 701.93 by the additional iterations. Export of the outflow hydrograph through the TAPE21 function indicates that 7% of the runoff volume remains within the basin at the time duration 132 hours instead of 15% that remained in the wetpond storage volume using the 1.5 inch orifice.

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* VERSION 4.1
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X X XXXXXXX XXXXX XXX

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THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE. THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE, SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY, DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1 HEC-1 INPUT PAGE 1

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LINE ID.....1.....2.....3.....4.....5.....6.....7.....8.....9.....10
1 ID MECKLENBURG COUNTY BMP DESIGN MANUAL
2 ID ANALYZED BY ABC ENGINEERING
3 ID DATE: OCTOBER 2006
* *****
*

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conditions. The example (the separate WQ_v and CP_v design shown in Chapter 4.2.9) for this site used a 15-inch orifice to control the 10- and 25-year, 6-hour storm events for flood control which is expected to be the approximate required orifice size for this example (*Combined Water Quality and Channel Protection design*).

The following HEC-1 output file illustrates the results of the iterative process for designing the flood control portion of the wet pond. Intermediate steps are not presented. The fourth step is to design the emergency spillway for the 50-year storm event. The elevation of the emergency spillway is set above the peak stage of the routed 25-year storm event. A freeboard of 6 inches above the 50-year peak stage to the top of embankment is required.

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* VERSION 4.1 *
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THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HECL (JAN 73), HECLGS, HECLDB, AND HECLKW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
 THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
 NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE, SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
 DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
 KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1 HEC-1 INPUT PAGE 1

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LINE ID.....1.....2.....3.....4.....5.....6.....7.....8.....9.....10
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2 ID ANALYZED BY ABC ENGINEERING
3 ID DATE: OCTOBER 2006
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*
* TIME SPECIFICATION CARD
* TOTAL COMPUTATIONAL DURATION 6 HOURS
4 IT 1 0 0 365
* DIAGRAM
* DATA INPUT CARD
5 IN 5
* OUTPUT CONTROL CARD
6 IO 5 0 0
*
7 KK PRE1
* *****
* ***** 10-YEAR, 6-HOUR STORM EVENT *****
* *****
8 PI .000 .011 .011 .012 .012 .012 .012 .013 .013 .013
9 PI .014 .014 .015 .015 .016 .016 .017 .018 .018 .023
10 PI .024 .025 .026 .027 .029 .036 .039 .042 .045 .049
11 PI .054 .079 .089 .103 .161 .201 .395 .590 .275 .177
12 PI .112 .095 .084 .057 .051 .047 .043 .040 .038 .030
13 PI .028 .027 .025 .024 .023 .019 .018 .017 .017 .016
14 PI .016 .015 .015 .014 .014 .013 .013 .013 .012 .012
15 PI .012 .011 .011 .000
16 KM 10-ACRE PRE-DEVELOPED CONDITIONS
17 KO 5 0 0 0 21
18 BA .0156
19 LS 0 65.0 0
20 UD 0.378
21 KK POST1
22 KM 10-ACRE POST-DEVELOPED CONDITIONS
23 KO 5 0 0 0 21
24 BA .0156
25 LS 0 77.8 0
26 UD 0.121
27 KK DETE10
28 KO 5 0 0 0 21
29 KM DETENTION BASIN DESIGN FOR 1-YEAR, 24-HOUR STORM EVENT HELD FOR 132 HOURS
30 KM POST-DEVELOPED WITH CONTROLS
31 KM 1.70 INCH ORIFICE TO ELEVATION 700.0
32 KM TOP OF EXTENDED DETENTION STORAGE 702
33 KM 15-INCH ORIFICE ACTIVATES FOR STAGES ABOVE 702
34 RS 1 ELEV 700
35 SV 0 .143 .293 .452 .618 .791 .973 1.166 1.367
36 SE 700 700.5 701 701.5 702 702.5 703 703.5 704
37 SQ 0.00 0.050 0.073 0.091 0.105 0.105 0.105 0.105 0.105 0.105

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38      ZZ
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*   JUN 1998
*   VERSION 4.1
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* RUN DATE 14MAR07 TIME 14:32:30
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MECKLENBURG COUNTY BMP DESIGN MANUAL
ANALYZED BY ABC ENGINEERING
DATE: OCTOBER 2006

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD			BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
				6-HOUR	24-HOUR	72-HOUR			
HYDROGRAPH AT	PRE1	8.	3.62	1.	1.	1.	.02		
HYDROGRAPH AT	POST1	28.	3.27	3.	3.	3.	.02		
ROUTED TO	DETE10	7.	3.68	1.	1.	1.	.02	702.45 3.68	

*** NORMAL END OF HEC-1 ***

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* FLOOD HYDROGRAPH PACKAGE (HEC-1)
*   JUN 1998
*   VERSION 4.1
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* RUN DATE 14MAR07 TIME 14:34:59
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* DAVIS, CALIFORNIA 95616
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THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HECL (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE. THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY, DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1 HEC-1 INPUT PAGE 1

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2	ID	ANALYZED BY ABC ENGINEERING									
3	ID	DATE: OCTOBER 2006									
4	IT	1	0	0	365	TIME SPECIFICATION CARD					
		TOTAL COMPUTATIONAL DURATION 6 HOURS									
5	IN	5	DIAGRAM DATA INPUT CARD								
6	IO	5	0	0	OUTPUT CONTROL CARD						
7	KK	PRE1	***** 25-YEAR, 6-HOUR STORM EVENT *****								
8	PI	.000	.014	.014	.014	.015	.015	.016	.016	.017	
9	PI	.017	.018	.018	.018	.019	.019	.020	.021	.022	.025
10	PI	.027	.028	.029	.031	.033	.043	.046	.049	.053	.058
11	PI	.064	.093	.104	.120	.189	.235	.466	.680	.324	.208
12	PI	.131	.111	.098	.067	.061	.055	.051	.048	.045	.034
13	PI	.032	.030	.029	.027	.026	.023	.022	.021	.021	.020
14	PI	.019	.019	.018	.017	.017	.016	.016	.016	.015	.015
15	PI	.014	.014	.014	.014	.000					
16	KM	10	ACRE	PRE-DEVELOPED	CONDITIONS						
17	KO	5	0	0	0	21					
18	BA	.0156									
19	LS	0	65.0	0							
20	UD	0.378									

```

21      KK  POST1
22      KM  10-ACRE POST-DEVELOPED CONDITIONS
23      KO   5   0   0   0   21
24      BA  .0156
25      LS   0  77.8   0
26      UD  0.121

27      KK  DETE10
28      KO   5   0   0   0   21
29      KM  DETENTION BASIN DESIGN FOR 1-YEAR, 24-HOUR STORM EVENT HELD FOR 132 HOURS
30      KM  POST-DEVELOPED WITH CONTROLS
31      KM  1.70 INCH ORIPICE TO ELEVATION 700.0
32      KM  TOP OF EXTENDED DETENTION STORAGE 702
33      KM  15-INCH ORIPICE ACTIVATES FOR STAGES ABOVE 702
34      RS   1  ELEV  700
35      SV   0  .143  .293  .452  .618  .791  .973  1.166  1.367
36      SE   700  700.5  701  701.5  702  702.5  703  703.5  704
37      SQ   0.00  0.050  0.073  0.091  0.105  8.091  9.106  10.019  10.855
38      ZZ

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* JUN 1998 *
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* RUN DATE 14MAR07 TIME 14:34:59 *
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MECKLENBURG COUNTY BMP DESIGN MANUAL
ANALYZED BY ABC ENGINEERING
DATE: OCTOBER 2006

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD			BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
				6-HOUR	24-HOUR	72-HOUR			
+	HYDROGRAPH AT								
+	PRE1	12.	3.60	2.	2.	2.	.02		
+	HYDROGRAPH AT								
+	POST1	36.	3.27	4.	4.	4.	.02		
+	ROUTED TO								
+	DETE10	9.	3.68	2.	2.	2.	.02	702.97	3.68

*** NORMAL END OF HEC-1 ***

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1*****
* FLOOD HYDROGRAPH PACKAGE (HEC-1) *
* JUN 1998 *
* VERSION 4.1 *
* RUN DATE 14MAR07 TIME 14:37:11 *
*****
* U.S. ARMY CORPS OF ENGINEERS *
* HYDROLOGIC ENGINEERING CENTER *
* 609 SECOND STREET *
* DAVIS, CALIFORNIA 95616 *
* (916) 756-1104 *
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THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE, SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1 HEC-1 INPUT PAGE 1

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LINE ID.....1.....2.....3.....4.....5.....6.....7.....8.....9.....10
1 ID MECKLENBURG COUNTY BMP DESIGN MANUAL
2 ID ANALYZED BY ABC ENGINEERING
3 ID DATE: OCTOBER 2006
*
* TIME SPECIFICATION CARD
* TOTAL COMPUTATIONAL DURATION 6 HOURS
4 IT 1 0 0 365
* DIAGRAM
* DATA INPUT CARD
5 IN 5
* OUTPUT CONTROL CARD
6 IO 5 0 0
*
7 KK PRE1
*
* ***** 50-YEAR, 6-HOUR STORM EVENT *****
*

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8      PI      .000      .016      .016      .016      .017      .017      .018      .018      .019      .019
9      PI      .020      .020      .021      .022      .022      .023      .024      .025      .026      .031
10     PI      .032      .033      .035      .037      .039      .049      .053      .056      .061      .066
11     PI      .073      .103      .116      .133      .209      .260      .513      .749      .356      .231
12     PI      .145      .124      .109      .077      .069      .063      .058      .054      .051      .040
13     PI      .038      .036      .034      .033      .031      .026      .025      .024      .023      .023
14     PI      .022      .021      .021      .020      .019      .019      .018      .018      .017      .017
15     PI      .017      .016      .016      .000
16     KM      10-ACRE PRE-DEVELOPED CONDITIONS
17     KO      5      0      0      0      21
18     BA      .0156
19     LS      0      65.0      0
20     UD      0.378

21     KK      POST1
22     KM      10-ACRE POST-DEVELOPED CONDITIONS
23     KO      5      0      0      0      21
24     BA      .0156
25     LS      0      77.8      0
26     UD      0.121

27     KK      DETE10
28     KO      5      0      0      0      21
29     KM      DETENTION BASIN DESIGN FOR 1-YEAR, 24-HOUR STORM EVENT HELD FOR 132 HOURS
30     KM      POST-DEVELOPED WITH CONTROLS
31     KM      1.70 INCH ORIFICE TO ELEVATION 700.0
32     KM      TOP OF EXTENDED DETENTION STORAGE 702
33     KM      15-INCH ORIFICE ACTIVATES FOR STAGES ABOVE 702
34     KM      20 FOOT EMERGENCY SPILLWAY SET AT 703.0
35     RS      1      ELEV      700
36     SV      0      .143      .293      .452      .618      .791      .973      1.166      1.367
37     SE      700      700.5      701      701.5      702      702.5      703      703.5      704
38     SQ      0.00      0.050      0.073      0.091      0.105      8.091      9.106      28.404      62.855
39     ZZ

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* FLOOD HYDROGRAPH PACKAGE (HEC-1) *
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MECKLENBURG COUNTY BMP DESIGN MANUAL
ANALYZED BY ABC ENGINEERING
DATE: OCTOBER 2006

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD			BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
				6-HOUR	24-HOUR	72-HOUR			
+									
+	HYDROGRAPH AT PRE1	15.	3.58	3.	3.	3.	.02		
+	HYDROGRAPH AT POST1	43.	3.25	4.	4.	4.	.02		
+	ROUTED TO DETE10	17.	3.52	3.	3.	3.	.02	703.21	3.52
+									

*** NORMAL END OF HEC-1 ***

Table 3-15 Summary of Controls Provided

Control Element	Type/Size of Control	Stor. (ac-ft)	Peak Elev. (MSL)	Disc. (cfs)	Remarks
Permanent Pool	N/A	1.00	700.00	N/A	3.33 times 1-inch storm event volume
Forebay	Riprap weir wall	0.06	700.00	N/A	Two forebays, one at each major inflow point
Water Quality Extended Detention (WQ _{v-ed})	1.7-inch orifice at 700.0 and 2.0-foot tall weir	N/A	N/A	N/A	Not modeled
Channel Protection (CP _v)	1.7-inch orifice at 700.0 and 2.0-foot tall weir	0.60	701.93	0	7% of runoff volume remains in basin at 132 hours (120 hours after center of rainfall)
Overbank Flood Protection Q ₁₀	15-inch orifice at 700.0	0.77	702.45	8	Same orifice control was designed for the 10- and 25-year storm events
Overbank Flood Protection Q ₂₅	15-inch orifice at 700.0	0.96	702.97	9	Same orifice control was designed for the 10- and 25-year storm events
Extreme Flood Protection Q ₅₀	20-foot weir at 703.00	1.05	703.21	17	Top of embankment is set at 704.

Comparison of the results of the wet pond design using the *Combined Water Quality and Channel Protection* approach with the design of the wet pond using a standard separate Water Quality (WQ_v) and Channel Protection (CP_v) design approach indicates that the BMP storage volume is very close to the same size. In the case of this example, the additional storage volume needed to comply with the combined design approach was offset by the benefits of routing the design storm events. These differences may be greater for other site types, sizes, imperviousness, etc.

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