CIRCULAR DEQ 8

MONTANA STANDARDS FOR

SUBDIVISION STORM DRAINAGE

2002 Edition

CHAPTER 1

SUBMISSION OF PLANS

1.0 GENERAL

All reports, final plans and specifications should be submitted to the Montana Department of Environmental Quality (MDEQ) or a delegated division of local government. No approval can be issued until final, complete detailed plans and specifications have been submitted to the reviewing authority and found to be satisfactory. Three copies of the final plans and specifications must be submitted. An approved set will be returned to the applicant, and one set to the local authority. Documents submitted for formal approval shall include, but not be limited to

a. A summary of the basic design,

b. General layout of drainage patterns and drainage structures,

c. Detailed plans and specifications,

d. Engineering report.

1.1 ENGINEERING REPORT

The engineering report for storm drains for subdivisions shall present the following information:

1.1.1 General information,

including

A. Identification of the subdivision, and

B. Name and mailing address of the owner.

1.1.2 Extent of the storm drainage,

including

A. Delineation of drainage areas within the subdivision, estimates of peak flows generated within these drainage areas, and estimates of flow volumes, if detention ponds or other storage facilities are included in the design,

B. Delineation of drainage areas outside the subdivision that flow through the subdivision, and estimates of peak flows generated within these drainage areas,
C. For flows that originate outside the subdivision, provisions for passing these flows through the subdivision without flooding home sites or drain field sites (at a recurrence interval of 100 years), and without overtopping of roadways (at a recurrence interval of 10 years),

D. For flows that originate within the subdivision, provisions for detaining or retaining these flows, so that the peak flow (e.g. from the 2-year, 1-hour event) that leaves the subdivision after development does not exceed the peak flow before development,

E. Where storm drainage is intended to be discharged into the ground, locations of nearby (within 200 feet) wells and drain fields that may be impacted, or a statement that there are no wells or drain fields nearby.

1.2 PLANS

Plans for storm drainage improvements shall provide for the following:

1.2.1 General layout

including

a. Suitable title,

b. Name of entity responsible for maintaining the storm drainage improvements,

c. Scale, in feet,

d. North point,

e. Date and name of the designer,

f. Legible prints, and

g. Location, nature and size of existing storm drainage facilities, if any, including drainage structures under existing roadways.

1.2.2 Detailed plans,

including

a. Location, size, type, slope and minimum cover of any proposed pipes,

b. Location and details of any proposed structures,

c. Direction of drainage flow along each street and at each intersection,

d. Location, size, length and slope of any proposed storm drain trunk lines,

e. Location and details of any proposed detention or retention ponds.

f. Location and details for erosion control (temporary and permanent) at each location where storm drainage leaves the subdivision, and at any other location where erosive velocities may occur. Information on soil types at these locations will be necessary to determine appropriate erosive velocities.

1.3 SPECIFICATIONS

Complete, detailed, technical specifications shall be supplied for the proposed drainage project.

CHAPTER 2

PEAK FLOW DETERMINATION

2.0 GENERAL

There are numerous methods available to determine the peak flow from a drainage basin, for different return periods. The peak flow is an instantaneous peak and is a function of the return period (such as 2-year) but not of the duration. All methods have limitations on the size of drainage basin for which the method is appropriate (some methods have an upper limit, while others have a lower limit). The designer may determine the appropriate method for peak flow determination. The reviewing authority may request documentation on the method used, in order to determine the applicability to the subdivision. Some commonly used methods are described in the Montana Department of Transportation=s Hydrology Chapter of the AASHTO Drainage Manual. This document is available on the Internet at www.mdt.state.mt.us, under Hydraulics Section.

2.1 RATIONAL METHOD

Use of the Rational Method is limited to drainage areas less than 200 acres. The rainfall intensity must be determined for a time period equal to the time of concentration of the drainage basin. Appendix A provides data which can be used to determine rainfall intensities for time periods as short as five minutes. The following minimum runoff coefficients shall be used for design: paved or other hard surface areas - 0.90, gravel areas - 0.80. The following maximum runoff coefficients shall be used for design of permeable surfaces: unimproved areas - 0.30, lawns or other landscaped areas - 0.10. Other values may be used if scientific and engineering data are submitted that indicate the values are appropriate.

2.2 USGS REGRESSION EQUATIONS

The U.S. Geological Survey has published regression equations which can be used to determine peak flows. These equations are generally valid only for drainage areas of about 1 square mile or larger. At the time of the preparation of this circular, the most current regression equations are published in USGS Water Resources Investigation Report 92-4048.

2.3 SCS CURVE NUMBER METHOD

The SCS Curve Number Method was developed by the Soil Conservation Service (now the Natural Resources Conservation Service). This method requires knowledge of the hydrologic group for each soil type, vegetation, and slope and its use is limited to drainage areas of 3 square miles or smaller. When two hydrologic groups are listed for a soil type (such as B/D), the CN for each soil type should be computed, and the average used. Note that in the use of this method, initial abstractions should never exceed more than 50% of the total precipitation (I_a/P # 0.50)

2.4 COMMERCIAL DEVELOPMENTS

Where a commercial lot is proposed, and the extent of proposed hard surface (buildings, concrete and paving) is known, the proposed development shall be used to compute the peak flow, and required detention. Where the extent of proposed hard surface is unknown, it shall be assumed that 80% of the lot area (excluding areas steeper than 15%) will be hard surface, and the peak flow and required detention computed based on this assumption. A location for a detention pond shall be selected and shown on the lot layout, and the required dimensions of the detention pond shall be determined. The location of the detention pond shall be at a location where the runoff will naturally accumulate, or at a location where the runoff can be directed.

2.5 MINIMUM FLOWS

In some developments, the soils are such that all of the rainfall from a design storm infiltrates completely, due to the porous nature of the soil. However, conditions such as snow melt or rain on frozen ground will produce some runoff. When more detailed information is unavailable to estimate the runoff produced by these events, a rainfall intensity equal to 20% of the 2-year, one-hour intensity may be used, with all of the rainfall assumed to run off.

In other developments, where the existing ground has very little vegetation, the installation of lawns and shrubs may decrease the runoff. In these areas, it may be appropriate to ignore runoff from areas that are not roadways, driveways, or buildings. However, runoff from these hard surfaces must still be accounted for, because the hard surfaces will tend to concentrate the runoff.

CHAPTER 3

ROADWAYS

3.1 GENERAL

All roadway drainage structures shall be designed to convey the 10-year peak flow without overtopping the roadway. They shall also be designed to convey the 100-year peak flow without inundating any home site or drain field, although overtopping the roadway is acceptable. Culvert computations shall be provided indicating the culvert inverts, roadway elevation, and flood elevations for both the 10-year and 100-year events.

Roadways within the subdivision shall be designed to accommodate runoff, either in curb and gutter sections or roadway ditch sections. The drainage design shall identify direction of flow for each side of each street, and at all intersections. Where sag vertical curves are included in the design, the design shall include provisions for disposal of water exceeding the capacity of the drainage structure without inundating homes or drain field locations, and these provisions shall be noted on the plans.

3.2 CULVERT TYPES

Culvert types (CSP, RCP, etc.) shall be shown on the plans. Culvert types shall be selected to ensure that the minimum cover recommended by the manufacturer is provided.

CHAPTER 4

RETENTION/DETENTION PONDS

4.1 GENERAL

The terms retention ponds and detention ponds are frequently used interchangeably. In order to avoid confusion in this circular, all ponds will be referred to as detention ponds, except those that drain a closed basin (i.e, there is no outlet for the pond, even when the water level exceeds the design water surface).

Detention ponds are intended to hold the peak flow, and release it at a slower rate, generally equal to the peak flow that would have occurred prior to development.

4.2 CLOSED-BASIN PONDS

Where there is no outlet for a pond, the water must be contained until it evaporates or infiltrates. The design of a closed-basin pond must consider a relatively long-duration rainfall event. Closed-basin ponds shall therefore be designed for a 24-hour rainfall event. Infiltration shall generally be based on tests at the ground surface, not percolation tests conducted 18 to 36 inches below the ground. At least one test pit shall be provided within the boundaries of each proposed closed-basin pond. The test pit shall be excavated to a minimum depth of 5 feet below the bottom of the proposed pond. The soils shall be classified based on the U.S. Department of Agriculture=s system and shall include any indications of bedrock or seasonal high groundwater. The design storm shall be at least a 2-year event, but the pond must also be analyzed for a 100-year event, to ensure that no home sites or drainfields are inundated during this event. An example of a closed-basin pond computation is shown in Appendix B.

4.3 DETENTION PONDS

In order to detain the peak flow from a rainfall event, it is only necessary to slow down the runoff from high-intensity, short-duration events. For detention ponds, the design event should have a duration at least equal to twice the time of concentration, but never shorter than one hour. The detention pond may be sized using a simple analysis of volume and discharge capacity, or a more complete analysis that includes routing of the design event. Calculations for pond size and outfall pipe size must be provided. Detention ponds shall be sized for a 2-year event, but must also be analyzed for a 100-

year event, to ensure that no home sites or drainfields are inundated during this event. If this analysis shows that the capacity of the pond will not hold the entire event, the analysis must include a description of where the excess water will go, and what the potential downstream damages may be. In this case, the analysis must also compare the pre-development situation with the proposed development situation.

The simple analysis would consist of a determination of the total volume of runoff during the design event. The detention pond volume must be at least equal to the total volume of runoff. The outflow from the detention pond shall not exceed the peak flow from the drainage basin prior to development. The outfall pipe from the detention pond must be sized so that the discharge at maximum head (when the pond is full) does not exceed the peak flow from the drainage basin prior to development. An example of detention pond sizing using this simple analysis is shown in Appendix C.

As an alternative, a hydrograph from this storm may be developed and routed through the detention pond. The outflow from a detention pond shall not exceed the peak flow from the drainage basin prior to development. An example of hydrograph development and a detention pond computation is shown in Appendix C.

Use of variable side slopes in the design of these ponds is encouraged, to reduce their visual impact. In general, detention ponds are most effective in the middle 1/3 of the drainage basin. Ponds near the upper end of the drainage may be too small to be effective, unless numerous ponds are constructed, and ponds near the lower end of the drainage may actually function to increase the peak flow.

CHAPTER 5

INFILTRATION FACILITIES

5.1 GENERAL

One common approach to handle additional runoff created by a development is the installation of some type of facility that allows the runoff to infiltrate into the ground. These facilities are referred to by various names, including drainage sumps, french drains, boulder pits, catch basins, and dry wells. These facilities are now classified as injection wells by the US EPA, which should be contacted regarding any Federal rules that may apply. There may also be local rules regarding the use of this type of facility.

5.2 DESIGN

The design of infiltration facilities should include a means for sediment removal and oil separation. It should also be designed to provide for other maintenance as necessary. Appropriate filter fabric shall be included to keep adjacent soils out of the infiltration facility.

5.3 SIZING

The sizing of these facilities may be done using a simple analysis or may include development of a storm-inflow hydrograph (as noted in Chapter 4 for ponds) and an outflow hydrograph.

If a simple analysis is completed, the storage in the void spaces (assumed to be 30% unless test data for the specified material is provided) must be large enough to contain the entire volume of runoff from the design event (a 2-year, one-hour event). No credit is provided for the infiltration into the ground during this short time period, because this can lead to under-sizing of the infiltration facility. An example of this is included as example 2 in Appendix D.

If storm inflow and outflow hydrographs are developed, infiltration into the ground can be included in the computation. In this case, the inflow is accommodated in two ways - storage in the void spaces, and infiltration into the ground. Percolation tests (completed at the depth of the infiltration facility) or other appropriate testing shall be done to determine the appropriate infiltration rate to use in the design. Infiltration facilities should be located above the seasonally high ground water. An example of hydrograph development and a dry well computation is shown in Appendix D.

Appendix A - Precipitation Values for Design

The information in this appendix was taken from the modifications to the AASHTO Drainage Manual, completed by the Hydraulics Section of the Montana Department of Transportation. The data below are short-duration (less than one-hour) intensities for the seven first-order weather stations in Montana.

Precipitation Intensity Values, in inches/hour

Return Period and Duration	Billings	Glasgow	Great Falls	Havre
2 years				
5 minutes	3.08	3.91	3.26	2.51
10 minutes	2.26	2.78	2.29	1.93
15 minutes	1.82	2.18	1.84	1.59
30 minutes	1.08	1.40	1.22	1.03
60 minutes	0.62	0.86	0.72	0.64

Return Period and Duration	Helena	Kalispell	Missoula
2 years			
5 minutes	2.60	2.06	2.09
10 minutes	1.87	1.60	1.49
15 minutes	1.46	1.32	1.19
30 minutes	0.90	0.82	0.70
60 minutes	0.52	0.48	0.41

The data below are two-year, one-hour intensities for 105 weather service stations in Montana. These 105 stations include the seven first order stations. The one-hour values are different from the 60-minute values due to the different period of record, and the difference between the highest rainfall in any 60-minute increment (for example, between 2:17 p.m. and 3:17 p.m.) and the highest rainfall in a one-hour increment (for example, between 2 p.m. and 3 p.m.). If durations less than one-hour are required for stations where short-duration data is not available, the ratios from a nearby first order weather station can be used, or the one-hour intensity values can be multiplied by the statewide averages shown below to obtain the 5, 10, 15 and 30 minute values.

For example, the 2-year, 5-minute intensity for Alzada would be the 2-year, 1-hour intensity of 0.60 times 4.7 = 2.82 inches per hour. The 2-year, 15-minute intensity for Alzada would be 0.60 times 2.8 = 1.68 inches per hour.

STATEWIDE AVERAGES FOR
SHORT-DURATION INTENSITIES

Duration	Multiply 1-hour intensity by:
5 minutes	4.7
10 minutes	3.4
15 minutes	2.8
30 minutes	1.7

TWO-YEAR, ONE-HOUR PRECIPITATION

Station	
Alzada	0.60
Ashland Ranger Station	0.62
Augusta	0.59
Baylor	0.63
Belgrade Airport	0.37
Billings	0.54
Bloomfield	0.77
Boulder	0.41
Bozeman 6 miles W	0.37
Bredette	0.67
Bridger	0.33
Broadus	0.64
Browning	0.42
Butte 8 miles S	0.41
Cameron	0.38
Cardwell	0.40

Choteau	0.50
Clark Canyon Dam	0.35
Cohagen	0.53
Content	0.62
Cooke City	0.34
Corwin Springs	0.34
Custer	0.50
Cut Bank	0.35
Darby	0.36
Station	
Decker	0.67
Dillon Airport	0.35
Dillon 9 miles S	0.36
Divide	0.39
Dodson	0.53
Dovetail	0.44
Drummond	0.40
Ekalaka	0.67
Elkhorn Hot Springs	0.32
Essex	0.34
Eureka Ranger Station	0.39
Fort Peck	0.71
Froid	0.64
Gibbons Pass	0.39

Gibson Dam	0.40
Glasgow	0.69
Glendive	0.72
Great Falls	0.58
Haugan	0.37
Havre	0.46
Hays	0.40
Hebgen Dam	0.41
Helena	0.47
Highwood	0.46
Hilger	0.53

TWO-YEAR, ONE-HOUR PRECIPITATION

Station	
Holter Dam	0.43
Iliad	0.44
Ismay	0.73
Joplin	0.47
Kalispell Airport	0.40
Kings Hill	0.45
Lakeview	0.43
Lavina	0.53
Lewistown	0.57
Libby Ranger Station	0.36

Lima	0.38
Lincoln Ranger Station	0.42
Livingston	0.40
Lodge Grass	0.55
Logan	0.43
Lolo Hot Springs	0.39
Lonepine	0.26
Martinsdale	0.47
Miles City	0.61
Millegan	0.51
Missoula	0.38
Molt 6 miles SW	0.45
Niehart	0.50
Ovando	0.43
Station	
Philipsburg Ranger Station	0.41
Plains Ranger Station	0.34
Polebridge	0.34
Reedpoint	0.49
Reserve 14 miles W	0.63
Ridge	0.70
Round Butte	0.39
Russell	0.47

St. Regis	0.41
Scobey	0.50
Seeley Lake Ranger Station	0.42
Shelby	0.47
Silver Star	0.33
Simms	0.51
Stanford	0.56
Summit	0.36
Swan Lake	0.38
Terry 25 miles NW	0.69
Townsend 12 miles ENE	0.42
Utica	0.64
Vananda	0.54
Westby	0.62
West Glacier	0.41

TWO-YEAR, ONE-HOUR PRECIPITATION

Station	
Whitehall	0.47
White Sulphur Springs	0.46
Willow Creek	0.39
Winnett 11 miles ESE	0.52
Wisdom	0.32

Wolf Point	0.60
Yellowtail Dam	0.55
Zortman	0.58

Appendix B - Example Retention Pond Calculations

This example is for a 3.2 acre development in Miles City. The 2-year, 24-hour rainfall event for Miles City is 1.41 inches, and the 100-year, 24-hour rainfall event is 3.75 inches, based on data obtained from the Montana Department of Transportation web site. Using the rational formula, a runoff coefficient of 0.95 assumes that 95% of the rainfall becomes runoff. Therefore, the total runoff from the 2-year, 24-hour event is 0.95 * 1.41 inches * 3.2 acres = 4.29 acre-inches. Dividing this value by 12 (to convert inches to feet) yields a required storage volume of 0.36 acre-feet. This would be the minimum size of the retention pond.

Using the same approach, the runoff from the 100-year, 24-hour event is 1.00 * 3.75 inches * 3.2 acres = 12.0 acre-inches, or 1.00 acre-feet. A runoff coefficient of 1.00 is used due to the correction for the 100-year event, based on $C*C_f = 0.95 * 1.25 = 1.19 > 1.00$. If the storage volume of the pond is equal to or greater than 1.00 acre-foot, then the pond would contain all of the runoff from the 100-year event, but it is still possible to have rainfall events greater than the 100-year event, and it may be appropriate to consider where the water will go during these very infrequent events. If the storage volume of the pond was between 0.36 and 1.00 acre-feet, some runoff would occur during events more frequent than the 100-year event, and an analysis should be done to determine where this runoff will go.

Appendix C - Example Detention Pond Calculations

This example is for a 3.2 acre development in Miles City. The undeveloped peak flow was 1.65 cfs for a 2-year design storm (using a C of 0.25, and a time of concentration of 10 minutes). This was established as the criteria to be used for allowable discharge from an on-site detention pond.

The first step is to make an estimate of the required pond size. This estimate is based on the total runoff for a one-hour period. The 2-year, one-hour rainfall for Miles City is 0.61 inches (see Appendix A). For a 3.2 acre site, with a runoff coefficient of 0.95, the flow rate would be 0.95 * 0.61 * 3.2 = 1.85 cfs, or 6,660 cubic feet. Site conditions restrict the pond to a normal maximum depth of about 2 feet, so a working depth of 1.5 feet will be used. The table below is used to determine the rainfall hyetograph. Time increments should be equal to (or shorter than) the time of concentration.

Time	Rainfall Intensity (in./hr.)	Total Rainfall Amount (in.)	Incremental Rainfall Amount (in.)	Incremental Rainfall Intensity (in./hr.)
10 min.	2.07	0.35	0.35	2.07
20 min.		0.44	0.09	0.54
30 min.	1.04	0.52	0.08	0.48
40 min.		0.55	0.03	0.18
50 min.		0.58	0.03	0.18
60 min.	0.61	0.61	0.03	0.18

Note: Rainfall intensity values determined as follows: two-year, 1 hour precipitation value of 0.61 determined from Appendix A; two-year, 10-minute intensity value determined by multiplying one-hour value by 3.4, in accordance with Appendix A; two-year 30-minute intensity value determined by multiplying one-hour value by 1.7, in accordance with Appendix A. Two-year, 20-minute, 40-minute and 50-minute total rainfall amounts were determined by straight-line interpolation.

The table below is used to determine the inflow hydrograph for the detention pond. The flows are computed using the rational equation, with a drainage area of 3.2 acres, a runoff coefficient of 0.95, and the rainfall intensity indicated.

Time	Incremental Rainfall Intensity (in./hr.)	Incremental Flow (cfs)	Incremental Volume (ft^3)	Total Volume (ft^3)
10 min.	2.07	6.29	3774	3774
20 min.	0.54	1.64	984	4758

30 min.	0.48	1.46	876	5634
40 min.	0.18	0.55	330	5964
50 min.	0.18	0.55	330	6294
60 min.	0.18	0.55	330	6624

The stage-storage-discharge relationship for the detention pond needs to be established. Using a total volume of 6600 cubic feet and a working depth of 1.5 feet, an approximate stage storage relationship is shown below. This relationship is exclusively a function of pond shape. The example is for a pond with vertical sides, for computational simplicity.

Depth	Storage Volume, ft^3
0.0	0
0.5	2200
1.0	4400
1.5	6600
2.0	8800
2.5	11,000
3.0	13,200

To achieve a discharge of 1.7 cfs with a head of 1.5 feet, an 8 inch pipe will be required. Using a simple culvert analysis (FHWA HY-8 program), the stage discharge relationship for this outlet is shown below.

Depth	Discharge, cfs
0.0	0.0
0.5	0.6
1.0	1.3
1.5	1.8
2.0	2.3
2.5	2.8
3.0	3.3

A simple routing procedure then determines the maximum pond size:

Time	Inflow Volume (ft^3)	Inflow + Storage (ft^3)	Depth (ft)	Outflow (cfs)	Outflow Volume (ft^3)	Storage Volume (ft^3)
10 min.	3774		0.86	1.10	660	3114
20 min.	984	4098	0.93	1.20	720	3378

30 min.	876	4254	0.97	1.26	756	3498
40 min.	330	3828	0.87	1.12	672	3156
50 min.	330	3486	0.79	1.01	606	2880
60 min.	330	3210	0.73	0.92	552	2658

The peak outflow is 1.26 cfs, below the pre-development flow of 1.65 cfs. The maximum storage volume is 3498 cubic feet. This is the required pond size, at a depth of 1.5 feet, for the 2-year event. The computation could be repeated with a modified stage storage relationship, to provide a final analysis. This method does make some simplifying assumptions, but they are generally not significant. The table above also indicates that the peak storage occurs very early in the rainfall event, so a longer duration event would not increase the required storage volume. A detention pond must also be analyzed to determine the possible impacts of the 100-year event. The following computation is for this event.

Time	Rainfall Intensity (in./hr.)	Total Rainfall Amount (in.)	Incremental Rainfall Amount (in.)	Incremental Rainfall Intensity (in./hr.)
10 min.	5.58	0.93	0.93	5.58
20 min.		1.12	0.29	1.74
30 min.	2.79	1.40	0.28	1.68
40 min.		1.48	0.08	0.48
50 min.		1.56	0.08	0.48
60 min.	1.64	1.64	0.08	0.48

Time	Incremental Rainfall Intensity (in./hr.)	Incremental Flow (cfs)	Incremental Volume (ft^3)	Total Volume (ft^3)
10 min.	5.54	16.84	10,104	10,104
20 min.	1.74	5.29	3,174	13,278
30 min.	1.68	5.11	3,066	16,344
40 min.	0.48	1.46	876	17,220
50 min.	0.48	1.46	876	18,096
60 min.	0.48	1.46	876	18,972

Time	Inflow Volume (ft^3)	Inflow + Storage (ft^3)	Depth (ft)	Outflow (cfs)	Outflow Volume (ft^3)	Storage Volume (ft^3)
10 min.	10,104		2.30	2.60	1560	8544
20 min.	3,174	11,718	2.66	2.96	1776	9942
30 min.	3,066	13,008	2.96	3.26	1956	11,052
40 min.	876	11,928	2.71	3.01	1806	10,122
50 min.	876	10,998	2.50	2.80	1680	9318
60 min.	876	10,194	2.32	2.62	1572	8622

The pre-development peak flow is 4.43 cfs. The peak flow after development is 2.96 cfs, so this meets the requirement that the peak flow not be greater than the pre-development peak. The maximum storage volume is 11,052 cubic feet for the 100-year event. This is the required pond size, at a depth of 3.0 feet. If this volume of water cannot be stored within the pond, then an analysis must be done to identify where the excess water will go, and at what rate.

Discharge Structure Considerations

There are a number of considerations in design of a discharge structure. A pipe sized to carry the design discharge at the design stage is the simplest form of structure. In some cases, an orifice plate may be installed in a discharge structure to limit the flow, but this is not as reliable as a pipe, and may cause more maintenance problems. All discharge structures should be reviewed to determine how they will function during larger rainfall events. Control of pollutants (oil, grease and sediments) should also be considered, both during the design event and during larger events.

Appendix D - Infiltration Facility Calculations

Example 1

This example is for a 0.2 acre development in Miles City, with a depth to groundwater of six feet and a percolation rate of 45 minutes per inch (at a depth of five feet). The 2-year, one-hour rainfall for Miles City is 0.61 inches (see Appendix A).

The first analysis is the simple analysis. For an intensity of 0.61 inches per hour, a runoff coefficient of 0.95 and an area of 0.2 acre, the Rational equation yields a flow of 0.116 cubic feet per second (0.95*0.61*0.2). For the 3600 seconds in an hour, this is a total volume of 418 cubic feet, which is the required volume of storage in the infiltration facility. Based on the depth to groundwater of six feet, the effective depth of the dry well will be five feet, allowing one foot for cover over the facility. Assuming a void ratio of 30% for gravels in the dry well, and a depth of 5 feet, the area of the dry well would need to be 418 / (5*0.30) = 279 square feet to hold the entire flow, without any percolation into the groundwater. This simple analysis therefore indicates that the required area of the infiltration facility is 279 square feet, with a depth of 5 feet.

If infiltration were considered in this example on a simple basis, the infiltration volume could be computed as follows. A percolation rate of 45 minutes per inch corresponds to an infiltration rate of 1.33 inches per hour or 0.11 feet per hour. For an area of 279 square feet, this would be a volume of about 31 cubic feet, which would reduce the size of the drywell by about 31/(5*0.30) = 20 square feet. An area of about 260 square feet would therefore be required.

The second analysis is a more detailed analysis including inflow and outflow hydrographs. For a 0.2 acre site, the estimated time of concentration is 5 minutes (although in fact it is probably less). The table below is used to determine the rainfall hydrograph. Time increments should be equal to (or shorter than) the time of concentration.

Time	Rainfall Intensity (in./hr.)	Total Rainfall Amount (in.)	Incremental Rainfall Amount (in.)	Incremental Rainfall Intensity (in./hr.)
5 min.	2.87	0.24	0.24	2.87
10 min.	2.07	0.35	0.11	1.32
15 min.	1.71	0.43	0.08	0.96
20 min.		0.46	0.03	0.36
25 min.		0.49	0.03	0.36
30 min.	1.04	0.52	0.03	0.36
35 min.		0.54	0.02	0.24
40 min.		0.56	0.02	0.24

45 min.		0.58	0.02	0.24
50 min.		0.59	0.01	0.12
55 min.		0.60	0.01	0.12
60 min.	0.61	0.61	0.01	0.12

Note: Rainfall intensity values determined as follows: two-year, 1 hour precipitation value of 0.61 determined from Appendix A; two-year, 5-minute intensity value determined by multiplying one-hour value by 4.7, in accordance with Appendix A; two-year, 10-minute intensity value determined by multiplying one-hour value by 3.4, in accordance with Appendix A; two-year, 15-minute intensity value determined by multiplying one-hour value by 2.8, in accordance with Appendix A; two-year 30-minute intensity value determined by multiplying one-hour value by 1.7, in accordance with Appendix A. All other total rainfall amounts were determined by straight-line interpolation.

The table below is used to determine the inflow hydrograph for the infiltration facility. The flows are computed using the rational equation, with a drainage area of 0.2 acres, a runoff coefficient of 0.95, and the rainfall intensity indicated.

Time	Incremental Rainfall Intensity (in./hr.)	Incremental Flow (cfs)	Incremental Volume (ft^3)	Total Volume (ft^3)
5 min.	2.87	0.55	165	165
10 min.	1.32	0.25	75	240
15 min.	0.96	0.18	54	294
20 min.	0.36	0.07	21	315
25 min.	0.36	0.07	21	336
30 min.	0.36	0.07	21	357
35 min.	0.24	0.05	15	372
40 min.	0.24	0.05	15	387
45 min.	0.24	0.05	15	402
50 min.	0.12	0.02	6	408
55 min.	0.12	0.02	6	414
60 min.	0.12	0.02	6	420

Based on the depth to groundwater of six feet, the effective depth of the dry well will be five feet, allowing one foot for cover over the facility. Assuming a void ratio of 30% for gravels in the dry well, and a depth of 5 feet, the area of the dry well would need to be 420 / (5*0.30) = 280 square feet to hold the entire flow, without any percolation into the groundwater. For a first design attempt, try a 10' x 10' x 5' deep dry well, to determine if this size is acceptable. Assuming that the percolation rate does not change with water depth (an incorrect, but somewhat conservative assumption), the infiltration for each 5 minute time period would be (5 minutes / 45 minutes per inch) * 100 square feet / 12 inches per foot = 1 cubic foot. Based on this calculation, it is apparent that the water lost to infiltration is minor compared to the inflow, and the selected dry well size is too small. For a second design attempt, try a 20' x 12' x 5' dry well. For a dry well area of 240 square feet, the infiltration for each 5 minute time period would be about 2 cubic feet. Routing the hydrograph through this dry well would yield a storage volume at the end of one hour of 420 cubic feet - (12 time increments * 2 cubic feet per increment) = 396 cubic feet. The storage volume of the dry well is 20 * 12 * 5 *0.30 = 360 cubic feet. The dry well is still slightly too small. A dry well 20' x 14' x 5' would be adequate, and computations should be done to show this, although this provides 280 square feet of area, which the initial computation above indicaed was adequate to hold the entire flow. (Actually, a dry well about 20' x 13' x 5' would be adequate.) This is a relatively large dry well, for a relatively small drainage area, and is only adequate to contain the 2-year runoff event. For a dry well sized for this frequent event, an analysis must be done to determine where runoff from larger events would be diverted.

<u>Example 2</u>

For a second example, the conditions are all the same as the first example, except the percolation rate is 5 minutes per inch.

The first analysis is the simple analysis. The analysis is the same as that in example 1, which indicates that the required area of the infiltration facility is 279 square feet, with a depth of 5 feet.

If infiltration were considered in this example on a simple basis, the infiltration volume could be computed as follows. A percolation rate of 5 minutes per inch corresponds to an infiltration rate of 12 inches per hour or 1.0 feet per hour. For an area of 180 square feet, this would be a volume of about 180 cubic feet, which would indicate that the required storage is only 238 cubic feet (418-180). As noted in the more detailed analysis below, this is too small.

The second analysis is a more detailed analysis including inflow and outflow hydrographs. For this condition and an 18' x 10' dry well, the infiltration for each 5 minute time period would be 15 cubic feet. The infiltration is now significant enough to look at a routing computation, shown below.

Time	Inflow Volume (ft^3)	Outflow Volume (ft^3)	Net Change (ft^3)	Storage Volume (ft^3)
5 min.	165	15	150	150

10 min.	75	15	60	210
15 min.	54	15	39	249
20 min.	21	15	6	255
25 min.	21	15	6	261
30 min.	21	15	6	267
35 min.	15	15	0	267
40 min.	15	15	0	267
45 min.	15	15	0	267
50 min.	6	15	-9	258
55 min.	6	15	-9	249
60 min.	6	15	-9	240

The maximum required storage volume is 267 cubic feet. The volume available in the selected dry well is 18 * 10 * 5 * 0.3 = 270 cubic feet. The selected dry well will be adequate to hold the runoff from the 2-year event, but not from any larger event. For a dry well sized for this frequent event, an analysis must be done to determine where runoff from larger events would be diverted.